California High-Speed Train Project

TECHNICAL MEMORANDUM

Interim Seismic Design Criteria

Bridges and Aerial Structures, Tunnels and Underground Structures, Passenger Stations and Building Structures TM 2.10.4

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Prepared by

for the California High-Speed Rail Authority

System Level Technical and Integration Reviews

The purpose of the review is to ensure:

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

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

System Level Technical Reviews by Subsystem:

Systems: _Not required___________________ ___________
Print Name: Date

Infrastructure: *Signed document on file*________ 28 May 09 _
Tom Jackson Date

Operations: Not required___________________ ___________
Print Name: Date

Maintenance: Not required___________________ ___________
Print Name: Date

Rolling Stock: Not required___________________ ___________
Print Name: Date

TABLE OF CONTENTS

ABSTRACT

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

This technical memorandum provides guidance for the seismic design for high-speed train bridges and aerial structures, tunnels and underground structures, passenger stations and buildings. Seismic design criteria for miscellaneous structures such as earth retaining structures, culverts, sound walls, and equipment supports and communication, mechanical and electrical system enclosures is either done by reference to existing documents or pending documents. The memorandum discusses the general system seismic performance objectives, applicable design codes and standards, and classification of high-speed train structures.

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

This seismic design criteria are intended to provide guidance for the seismic design process, including methodologies, analytical procedures, and assumptions; and to establish acceptable standards in terms of structural performance and integrity of the high-speed train system design.

The seismic design criteria are to be used in combination with other applicable technical memoranda and design guidelines documents.

6.0 DESIGN CRITERIA

6.1 DESIGN CLASSIFICATIONS

6.1.1 Structural Classifications

HST facility structures 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. Different facilities have varying design objectives and the design criteria should recognize that. Structural classification provides a method to differentiate between the various design objectives for the different structural types.

6.1.1.1 General Classifications

CHST facility structures are classified as:

- Bridges – high-speed train track structures spanning rivers, lakes, canals, and canyons
- Aerial Structures – elevated high-speed train trackway structures that cross highways and railroads.
- Earth Retaining Structures – including U-walls and retaining walls
- Cut-and-Cover Underground Structures – including cut-and-cover underground stations or track structures
- Bored Tunnels
- Mined Tunnels
- Buildings and All Other Above-ground Structures – including station buildings, parking structures, secondary and ancillary buildings, sound walls, and miscellaneous structures
- Underground Ventilation Structures
- Underground Passenger Stations
- Equipment and Equipment Supports

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

- **Primary Structures:** Primary Structures are those that directly support track and running trains, including bridges, aerial structures, stations, tunnels and underground structures, and earth retaining structures. Primary Structures also include other facilities and systems essential to train service including, tracks, rail fasteners, earth embankments and fills, train control, operation, and communication facilities, traction power facilities, power distribution network facilities, and equipment facilities.
- **Secondary Structures:** Secondary Structures are those that are not necessary for immediate resumption of train service including, administrative buildings, shop buildings, storage facilities, cash handling buildings, parking structures and training facilities.

This document is related to seismic design of Primary Structures. The seismic design criteria for Secondary Structures are developed in other documents.

6.1.1.2 Importance Classification

Primary HST facility 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. Important Structures shall be designed to meet all three performance levels defined in Section 6.2.2
- **Ordinary Structures:** Any structures that are not designated as Important will be considered Ordinary Structures. Ordinary Structures shall be designed to meet the No Collapse and Operability Performance levels defined in the Section 6.2.2.

Designers shall make a formal written request to the Authority or delegate, justifying each structure's Importance Classification as either Important or Ordinary. The Authority or delegate shall determine the Importance Classification of a structure.

6.1.1.3 Technical Classification

Primary HST facility structures shall be further be classified according to their technical complexity as it relates to design. This classification will dictate the analytical requirements that must be met during the design. See Table 6-1 for the performance criteria and analytical requirements, based upon Importance and Technical Classification.

- **Complex Structure:** 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, highly skewed supports, or support columns of drastically varying height.
    - **Unusual Framing -** Structures that include outrigger or C-bent supports, unbalanced mass and/or stiffness distribution, extremely tall support columns, or multiple superstructure types.
    - **Long Aerial Structure Spans -** Aerial structures that have spans greater than 300 ft.
    - **Unusual Geologic Conditions -** Structures that are founded on soft soil, soil having moderate to high liquefaction potential, soil of significantly varying type over the length of the structure, or structures located in close proximity to earthquake faults. Unusual geologic conditions and near source seismic effects will be defined by the Geotechnical Report.
- **Standard Structure:** Structures that are not Complex Structures and comply with the CHSTP Design Guidelines for Standard Aerial Structures.
- **Non-Standard Structure:** Structures that do not meet the requirements for Complex Structures or the CHSTP Design Guidelines for Aerial Structures. The same design and analysis requirements used for a Standard structure must be met.

Designers shall make a formal written request to the Authority or delegate justifying each structure's Technical Classification as Complex, Standard, or Non-Standard. The Authority or delegate shall make the final ruling on the Technical Classification.

Table 6-1 Performance Criteria Requirements per Structure Classification

<table>
<tr><th colspan="2" rowspan="2"></th><th colspan="2">Importance Classification</th></tr>
<tr><th>Important</th><th>Ordinary</th></tr>
<tr><th rowspan="6">Technical Classification</th><th rowspan="2">Complex</th><td>Three Criteria: NCL, SPL, OPL</td><td>Two Criteria: NCL, OPL</td></tr>
<tr><td>Nonlinear Time-History Analysis (Section 6.5.4.4)</td><td>Linear Response Spectrum Analysis (Section 6.5.4.3</td></tr>
<tr><th rowspan="2">Standard</th><td>Three Criteria: NCL, SPL, OPL</td><td>Two Criteria: NCL, OPL</td></tr>
<tr><td>Linear Response Spectrum Analysis (Section 6.5.4.3)</td><td>Equivalent Static Analysis (Section 6.5.4.2) or Linear Response Spectrum Analysis (Section 6.5.4.3)</td></tr>
<tr><th rowspan="2">Non-Standard</th><td>Three Criteria: NCL, SPL, OPL</td><td>Two Criteria: NCL, OPL</td></tr>
<tr><td>Linear Response Spectrum Analysis (Section 6.5.4.3)</td><td>Equivalent Static Analysis (Section 6.5.4.2) or Linear Response Spectrum Analysis (Section 6.5.4.3)</td></tr>
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6.2 SEISMIC DESIGN PERFORMANCE CRITERIA

6.2.1 General

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

6.2.2 Seismic Performance Criteria

Following an earthquake, the functionality of HST systems will vary based on the actual ground motions and the design based performance criteria. For HST facilities and structures, there are three levels of Seismic Performance Criteria:

- **No Collapse Performance Level (NCL):** HST facilities are able to undergo the effects of the Maximum Considered Earthquake (MCE) with no collapse. Significant damage may occur that requires extensive repair or complete replacement, yet passengers and personnel are able to evacuate safely.
- **Safety Performance Level (SPL):** HST facilities are able to undergo the effects of the Design Basis Earthquake (DBE) with repairable damage and temporary service suspension. However, normal service can resume within a reasonable time frame, and passengers and personnel can safely evacuate. Only short term repairs to structural and track components are expected.
- **Operability Performance Level (OPL):** HST system will be able to operate at maximum design speed and safely brake to a stop during a Lower-level Design Basis Earthquake (LDBE). Normal service will resume when track alignments have been inspected and any necessary short term track repairs, such as minor realignment and grade-adjustment, are made. No structural damage is expected.

In general, an individual structure may need to comply with multiple performance levels.

See Table 6-1 for the performance criteria requirements, based upon Importance and Technical Classification.

See Table 2-2, Table 2-3, and Table 6-4Table 2-4 for performance objectives and acceptable damage for No Collapse Performance Level (NCL), Safety Performance Level (SPL), and Operability Performance Level (OPL), respectively.

Table 6-2: 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 under all dead load and live load during and after a Maximum Considered Earthquake (MCE). The performance objectives are: 1. No collapse. 2. Safe evacuation of passengers and personnel. 3. For underground structures, no flooding or mud inflow.	Significant yielding of reinforcement steel or structural steel, however, fracture is not permitted
		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 6-3: Performance Objectives/Acceptable Damage for Safety Performance Level (SPL)

Performance Level	Performance Objectives	Acceptable Damage
Safety Performance Level (SPL) Design Basis Earthquake (DBE)	**Safety Performance Level (SPL):** The main objective is to limit structural damage to be repairable such that normal train operations can resume within a reasonable amount of time following the Design Basis Earthquake (DBE). The performance objectives are: 1. Limited structural and track damage, requiring short term repairs. 2. Safe evacuation of passengers and personnel. 3. Resumption of normal service within a reasonable amount of time. 4. Restore operation of all equipment within reasonable amount of time. 5. Provide safe performance in aftershocks 6. Bridge piles shall not experience significant damage, no rocking of pile caps on top of piles is permitted. Limited rocking of structures supported on spread footings. 7. For underground structures, no flooding or mud inflow.	Yielding of reinforcement steel or structural steel, although replacement should not be necessary and serviceability should be maintained
		Spalling of concrete cover where access permits repair
		Small permanent offsets, not permanently interfering with functionality or serviceability
		Flexural plastic hinging of the columns should be used as the fusing mechanism where rocking is not allowed or economically viable.

Table 6-4: Performance Objectives/Acceptable Damage for Operability Performance Level (OPL)

Performance Level	Performance Objectives	Acceptable Damage
Operability Performance Level (OPL) Lower-level Design Basis Earthquake (LDBE)	**Operability Performance Level (OPL):** The main objective is to limit structural damage to be minimal such that trains can safely operate at maximum design speed and safely brake to a stop during a Lower-level Design Basis Earthquake (LDBE). The performance objectives are: 1. Essentially elastic structural response, minor structural damage. 2. Normal train breaking operations. 3. Safe evacuation of passengers and personnel. 4. Resumption of normal service within a reasonable amount of time, limited to inspection and minor repair of track due to minor realignment and grade-adjustment. 5. Provide safe performance in aftershocks 6. No rocking of bridge foundations 7. For underground structures, no flooding or mud inflow.	Minor inelastic response
		Narrow cracking in concrete and no yielding in either reinforcement steel or structural steel
		No measurable permanent deformations

6.2.3 Design Earthquakes

The system performance criteria approach uses design earthquakes to which CHST facilities are to be designed. As more devastating earthquakes have a lower probability of occurrence, a probabilistic approach to defining earthquake hazard is used in engineering design. A "return period" identifies the expected rate of occurrence of a level of earthquake. Additionally, deterministic methods evaluate earthquakes that are estimated to produce the most severe ground motion.

For the purpose of this document, three levels of design earthquakes (Maximum Considered Earthquake (MCE), the Design Basis Earthquake (DBE) and the Lower-level Design Basis Earthquake (LDBE)) are defined as follows:

- **Maximum Considered Earthquake (MCE):** This level of earthquake is taken as consistent with the MCE as defined in ASCE 7-05. In general, this level of earthquake corresponds to the probabilistic ground motions having a return period of about 2,475 years with the deterministic limits provided in the ASCE 7-05. This event corresponds to ground motions having about 2% probability of exceedance in 50 years or about 4% probability of exceedance within the design life of 100 years.
- **Design Basis Earthquake (DBE):** The greater of the deterministic event with median plus one-half standard deviation or the probabilistic event having a return period of about 950 years, about 10% probability of exceedance within the design life of 100 years.
- **Lower-level Design Basis Earthquake (LDBE):** The probabilistic event with a return period of about 100 years, about 63% probability of exceedance within the design life of 100 years.

For more information see the TM 2.9.3 - Geologic and Seismic Hazard Evaluation Guidelines.

6.2.4 Fault Crossings

It is recognized that where the alignment crosses active faults, system seismic performance criteria may be impractical due to expected large offset displacements each side of the fault. Specifically, track damage may exceed acceptable operating criteria, even after the LDBE event.

Therefore, it is desirable that all identified major fault zones be crossed at-grade without any aerial structures, so damage can be quickly repaired and service resumed after the LDBE event.

Policies and criteria will be given in separate technical memoranda for fault crossing design.

6.2.5 Seismic Design of Earth Retaining Structures

Seismic design of earth retaining structures, including U-walls and retaining walls, shall conform to Caltrans Bridge Design Specifications (CBDS), and any additional requirements set forth in the site specific Geotechnical Data Report.

6.2.6 Seismic Design Benchmarks for 15% & 30% Design

All structure classifications shall meet basic benchmark seismic design requirements for 15% and 30% design. These benchmarks are more stringent for Important and Complex structures.

These benchmarks will be given in future technical memoranda for the 15% and 30% Design.

6.3 DESIGN CODES AND REFERENCE DESIGN COMMENTS

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

1. AASHTO LRFD: AASHTO LRFD Bridge Design Specifications 4th Edition, 2007 published by the American Association of State Highway and Transportation Officials
2. AREMA: American Railway Engineering and Maintenance-of-Way Association, Manual for Railway Engineering
3. American Concrete Institute, Building Code Requirements for Reinforced Concrete, ACI 318
4. American Institute of Steel Construction, Steel Construction Manual, Thirteenth Edition

5. ASCE 7-05: Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineering
6. AWS: Structural Welding Code, Steel, 1996 ANSI/AWS D1.1-96
7. AWS: Bridge Welding Code ANSI/AASHTO/AWSD1.5-95
8. The California Building Code
9. California Department of Transportation (Caltrans) Bridge Design Manuals, latest edition
   - Bridge Design Specification (CBDS) - AASHTO LRFD Bridge Design Specification, 2005, with Caltrans Interim Revisions
   - Bridge Memo to Designers Manual (CMTD)
   - Bridge Design Practices Manual (CBDP)
   - Bridge Design Aids Manual (CBDA)
   - Bridge Design Details Manual (CBDD)
   - Bridge Memo to Designers Manual (CMTD)
   - Standard Specifications
   - Standard Plans
   - Seismic Design Memorandum
   - Caltrans Seismic Design Criteria ver. 1.4 (CSDC)
10. European Standard EN 1991-2:2003 Traffic Loads on Bridges
11. European Standard EN 1990/A1:2006-07 Annex 2
12. FEMA 356 - Prestandard and Commentary for the Seismic Rehabilitation of Buildings, November 2000
13. NEHRP – Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2000 Edition

The edition of each standard used shall be current. Later editions may be used subject to approval of the Authority.

In the event of conflicting requirements between the Design Criteria and the standards and codes or local regulations, referenced above, the Design Criteria shall take precedence and the Designer shall advise the Authority or designee in writing.

The Design Criteria makes reference to, or incorporates, portions of the following documents:

- Bay Area Rapid Transit (BART) – Track Safety Standards, 1995
- BART – Earthquake Safety Program Design Criteria, 2006
- THSR – Taiwan High Speed Rail Corporation, Taiwan High Speed Rail Design Specifications, Volume 9, 1999
- FHA – Seismic Retrofitting Manual for Highway Structures: Part 1-Bridges and Part 2-Retaining Structures, Slopes, Tunnels, Culverts, and Roadways, 2006
- ATC-32 – Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, 1996

6.4 SEISMIC DESIGN

For MCE and DBE events, a performance (i.e., strain and deformation based) design approach is used for CHSTP structures. For LDBE events, force based design is applicable, since all structures are to essentially respond elastically.

For bridges and aerial structures, Caltrans Bridge Design Manuals (CBDM) form the basis of the seismic design philosophy, with project specific project amendments.

For cut-and-cover structures, CBDM also form the basis of design, with project specific amendments.

For portals and U-sections, mined and bored tunnels, and ventilation and access shafts, seismic design code criteria are pending. However, should concrete structural elements be mainly used, then CBDM shall apply

For passenger stations and building structures, CBC (i.e., force-based design) methodology will be used for all non-seismic related design. FEMA 356, with project specific amendments, is referenced for seismic design, since it contains appropriate performance based methodologies.

Although FEMA 356 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.

6.5 BRIDGES AND AERIAL STRUCTURES

6.5.1 General

All bridges and aerial structures are categorized as Primary Structures.

Based upon the bridge and aerial structure's Importance and Technical Classification, Table 6-1 presents the required performance criteria and analytical effort.

6.5.2 Seismic Design Philosophy

For bridges and aerial structures, the intended structural action under seismic loading is that of a Limited Ductility Structure, whereby:

- The bridge or aerial structure shall have a clearly defined mechanism for response to seismic loads.
- Inelastic behavior shall be limited to columns, piers, footing foundations and abutments.
- The detailing and proportioning requirements for full-ductility structures shall be satisfied.

In general, the designer allows specified structural components to undergo inelastic behavior under Maximum Considered Earthquake (MCE) and Design Basis Earthquake (DBE), providing a fusing mechanism, while force-protecting other components. The two main allowable fusing mechanisms for HST bridges and aerial structures are member flexural plastic hinging and foundation rocking. In either 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, negative moment in footings with no top mat of reinforcing, and unseated girders.

An adequate margin of strength shall be provided between the designated load-resistance fusing mode and non-ductile failure modes. Sufficient over-strength shall be provided to assure the desired fusing mechanism occurs 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 not extensive 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 applied.

6.5.2.1 Pre-Determined Locations of Damage

The designer should pre-determine the location of inelastic behavior, either rocking or plastic hinging, for the structure.

Rocking response is limited to the spread footing foundations. No rocking allowed for LDBE event.

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. No plastic hinge formation allowed for piles below the ground surface. The bridge deck shall remain essentially elastic.

6.5.3 Design Codes

Current Caltrans seismic analysis and design philosophies as stated in Caltrans Bridge Design Manuals (CBDM) form the basis of bridge and aerial structure seismic design. However, certain criteria herein exceed those of CBDM.

For items not specifically addressed in this or other sections of the CHST Project Design Criteria (CDC), CBDM shall be used.

6.5.4 Seismic Analysis and Demand Considerations

In increasing order of complexity, analysis techniques include equivalent static analysis, linear response spectrum analysis, and non-linear time history analysis. See Table 6-1Table 2-1 for the required performance criteria and analytical effort, based upon the bridge and aerial structure's Importance and Technical Classification.

6.5.4.1 Displacement Demands (Δ_D)

The displacement demand, Δ_D, shall be determined using equivalent static response spectrum or time history analysis. Modeling and analysis shall conform to CBDM, and in particular to Caltrans Bridge Design Aids Manual (CBDA), Bridge Memo to Designers Manual (CMTD), and Seismic Design Criteria (CSDC).

6.5.4.1.1 Displacement Demand Amplification Factor

The displacement demand, Δ_D, obtained from equivalent static analysis or linear response spectrum analysis, shall be multiplied by an amplification factor, C, as follows:

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

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

where,

T = fundamental period of structure (including foundation flexibility)

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

6.5.4.2 Equivalent Static Analysis

Equivalent static analysis may be used to determine the Displacement Demand, Δ_D, when the structure can be characterized as a simple single-degree-of-freedom (SDOF) system. Typically, this is the case where a bridge or aerial structure has single column piers or multiple column bents, and where most of the structural mass is concentrated at a single level. For these structures, displacement demand and capacity shall be expressed in terms of a generalized, controlling deflection of the structure at the center of mass of the superstructure.

The total applied force shall be equal to the product of the S_a (Acceleration Response Spectral value) and the Weight (W), but not less than 0.4g, or as defined the Geotechnical Data Report. Displacement demand, Δ_D, obtained from the equivalent static analysis, shall be multiplied by the adjustment factor given in Section 6.5.4.1.1, to account for the uncertainty associated with calculation of structural period.

6.5.4.3 Linear Response Spectrum Analysis

Linear response spectrum analysis involves creating a three dimensional analytical 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 fundamental structural mode shapes for use in analysis.

A sufficient number of modes should be included to account for a minimum of 90% of the total structural mass. It should be noted that 90% mass participation may not be sufficient for long viaduct models. The designer should examine the modes for such structures to ensure that they are sufficient to capture the behavior of the structure. The modal response contributions shall be combined using the complete quadratic combination (CQC) method. Displacement demand, Δ_D, obtained from the equivalent linear response spectrum analysis, shall be multiplied by the adjustment factor given in Section 6.5.4.1.1, to account for the uncertainty associated with calculation of structural period.

To account for effects of earthquake loadings in mutually orthogonal three directions, the maximum response for a single component quantity shall be obtained by combining the responses from three directional global response spectrum analyses, using SRSS method. The three directions of earthquake input are the longitudinal (L), transverse (T) and vertical (V) directions. The maximum earthquake response (E) of a particular scalar component quantity shall be calculated from:

$$E = (E_L^2 + E_T^2 + E_V^2)^{1/2}$$

where E_L, E_T and E_V are the responses due to longitudinal, transverse and vertical direction earthquake inputs, respectively.

Alternatively, the 100%-40% combination rule can be used. In this method, the maximum earthquake response (E) of a particular scalar component quantity can also be calculated from the larger of:

$$E= \quad 1.0E_L + 0.4E_T + 0.4E_V$$

$$0.4E_L + 1.0E_T + 0.4E_V$$

$$0.4E_L + 0.4E_T + 1.0E_V$$

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.

Where there is a change in soil type along the bridge alignment or the bridge is irregular, as defined in CBDM, consideration shall given 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 the response spectrum analysis.

Dead and live loads shall be added to the computed demands or applied as an initial condition. Live loads shall be applied to the structure system such that to produce the maximum effects in accordance with Section 6.5.4.13.

Appropriate linear stiffness shall be assumed for abutments and expansion hinges. Analyses shall be performed for compression models (abutments active, 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 CBDA and CMTD recommendations.

6.5.4.4 Nonlinear Time History Analysis

Nonlinear time history analysis involves creating a three dimensional analytical 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, and may include nonlinear representations of structural and foundation elements.

The time histories shall reflect the characteristics (magnitude, fault distances, site condition, spectral content, and source mechanisms) of the controlling design earthquake ground motion. The motions shall be three-component (two horizontal components and one vertical component) ground motion time histories, selected and properly scaled or spectrally matched from no fewer than three recorded events. 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 foundation elements.

For each data set, the square root of the sum of the squares (SRSS) of the 5%-damped site-specific spectrum of the scaled horizontal components shall be constructed. These data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the

5%-damped spectrum for the design earthquake for periods between 0.2T seconds and 1.5T seconds (where T is the fundamental period of the structure).

When time history analysis is used, the analysis shall be performed under multiple sets of ground motions, there are two options:

1. Use three sets of ground motions. The envelope maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used for design.
2. Use seven sets of ground motions. The average value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used for design.

Nonlinear time history analysis may also be used for structures where response spectrum analysis is deemed overly conservative.

In cases when a nonlinear time history analysis is used to calculate a displacement demand, Δ_D, the analysis shall conform to the following:

1. The designer shall develop a nonlinear time history analysis plan to be submitted to CHSTP for review and comment. The plan should discuss in detail the proposed analysis, indicating the analysis software to be used as well as the modeling assumptions made and the various modeling techniques to be employed.
2. At a minimum, the nonlinear time history analysis shall comply with the following guidelines:

   - Dead and live loads shall be applied as an initial condition. Train loads and mass shall be included in the “dynamic” mass as required by Section 6.5.4.13.
   - 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.

6.5.4.5 Rocking

Where rocking of the footings is used as the primary seismic response mechanism of the structure, non-linear analysis methods are required. One acceptable method for such analysis is the most current Caltrans rocking analysis procedure which is based on work by Priestley and Seible (Ref. 5.1) and 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, if the designer wishes to use the displacement response spectrum provided they should reduce the displacement values to account for greater damping associated with rocking behavior as recommended in the Caltrans procedures).
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 should 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 nonlinear time history analysis.

6.5.4.6 Flexural Plastic Hinging

Where flexural plastic hinging is used as the primary seismic response mechanism of the structure, the analysis shall conform to the most current Caltrans procedures.

6.5.4.7 Assessment of Relative Longitudinal Displacements between Decks

For computing the relative displacements between decks under combinations of loads including LDBE, use EN 1991-2:2003 Subsection 6.4 and EN 1990/A1:2006-07 Annex A2. The detailed modeling requirements for such an analysis are defined in Track/Structure Performance Criteria and Riding Comfort Criteria.

6.5.4.8 Element Cross Section Analysis

Concrete element cross sectional properties, including the effective area moment of inertia, shall be determined from moment-curvature analyses that consider the effects of concrete cracking, confinement and strain hardening of the reinforcement in accordance with CMTD 20-4 and Section 5.6 of CSDC.

For built-up structural steel sections, effective section properties presented in Appendix B of CSDC may be used in a seismic analysis in lieu of properties computed using more refined section analysis procedures.

6.5.4.9 Material Properties

Material properties used in calculating the demands of structural components due to all seismic loadings shall conform to Section 3.2 of CSDC for reinforced and/or prestressed concrete components and Sections 3.2.2 and 4.2 of CSDC for structural steel components.

For seismic design purposes, the expected material properties shall be used in determining the demands.

6.5.4.10 Foundation Flexibility

For pile foundations, soil-foundation-structure interaction effects shall be considered. Liquefaction, lateral spreading and other seismic phenomena as specified in Section 6.5.4.16 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 rocking) is large relative to the column or pier stiffness, then the foundation may be modeled as fixed.

For spread footing foundations, liquefaction, lateral spreading and other seismic phenomena as specified in Section 6.5.7 shall be considered.

6.5.4.11 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, it should be checked that the boundary conditions and element properties remain consistent with initial modeling assumptions.

6.5.4.12 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 6.5.4.4, of the structural system should be performed that considers rail-structure interaction.

This rail-structure interaction shall include, at a minimum, the rails and fastening system, modeled in a manner that considers the slippage of the fasteners and the stiffness of the rails. When rail-structure interaction is included in the model, consideration shall be given to the capacity of the rail fasteners and connections in both shear and tension. Otherwise, any benefits to the structure performance provided by the continuous welded rail shall be ignored.

Note that that the rail and fastening system design will need to comply with the SPL and OPL performance criteria.

6.5.4.13 Effects of Train Mass and Live Load on Displacement Demand

Train live loads with impact factor shall be applied to the structural system, per TM 2.3.2 - Structure Design Loads, as to produce the maximum effect. It should be noted that 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, HST train loads may be modeled as equivalent static distributed loads. Where equivalent distributed loads are used in the analysis, the Design Engineer shall account for any local or global effects to the structure due to actual concentrated axle loads.

The mass associated with one track of train live load shall be included in the models. This mass should be applied at the center of mass of the train.

When checking for loading combinations including seismic effects, the following live loads shall be considered simultaneously:

1. One train live load
2. Longitudinal force from one train live load (braking or acceleration)

6.5.4.14 P-Δ Effects

For flexural plastic hinging, P-Δ effects shall conform to the requirements in Section 4.2 of CSDC.

6.5.4.15 Vertical Seismic Loads

Vertical seismic loads should be considered when designing the HST aerial structures. In general, the vertical acceleration can be approximated by an equivalent static vertical force applied to the superstructure. The static force can be determined by multiplying the dead load by the peak vertical response spectrum value. Alternatively, the designer may determine the fundamental period of the superstructure girder and use this to determine the corresponding vertical response spectrum value. The uniform seismic force may be reduced to account for the mass participation factor of the fundamental period. See Section 6.5.4.3 for more information on combining vertical and horizontal seismic demands.

6.5.4.16 Soil Structure Interaction

Soil-structure interaction (SSI) effects shall be considered for all structures that are not supported by rock or rock-like soil foundation material having a minimum shear wave velocity of 2500 ft/sec, or as determined in the Geotechnical Data Report. In performing SSI modeling and analysis of soil-structure systems, the following criteria should be considered.

6.5.4.16.1 SSI Analysis Method

Both direct and substructuring analysis methods are acceptable for SSI analysis. Direct methods involve analyzing the total soil-structure system in one step and can be applied to both linear and nonlinear systems. In the substructuring method, the total SSI system is partitioned into two substructures, namely the structure and foundation. The foundation is analyzed first to develop the foundation impedance properties and scattered motions, which are then specified as boundary condition in dynamic analysis of the structure. The substructuring methods, in general, are applicable only to linear systems. Nonetheless, nonlinearity of foundation soils may be accounted for by using equivalent linear method.

6.5.4.16.2 Seismic Wave Field

In the SSI analysis, the seismic wave field may be assumed to consist of vertically propagating shear and compression waves if the torsional effects due to inclined propagating waves are considered. The effects of wave incoherence on the torsional and/or rocking response of long span structures or structures with large footprints should also be considered.

6.5.4.16.3 Foundation Soil Layering and 3-D Effects

The effects of foundation soil layering and 3-D geometry on the dynamic stiffness and radiation damping of the foundation should be accounted for in the SSI analysis.

6.5.4.16.4 Nonlinear Soil Behavior

The nonlinear soil behavior should be considered in the SSI analysis. The soil nonlinearity may be assumed to consist of two parts: primary and secondary nonlinearities. The primary soil nonlinearity denotes the nonlinear soil behavior due to ground excitation in the absence of the structure. The secondary nonlinearity denotes the material nonlinearity induced in the soil due to SSI effects. The primary nonlinearity shall be accounted for in the SSI analysis. The secondary nonlinearity need not be considered in the SSI analysis if 1) provision for uncertainties in the soil material properties as stated in Section 6.5.4.16.9 is included in the SSI analysis; and, 2) there is no potential for soil yielding in the vicinity of the structure that can result in foundation sliding, uplift, separation from side soils, etc.

6.5.4.16.5 Structure-Structure Interaction

In general, structure-structure interaction effects may be ignored in the SSI analysis unless the response of one structure can be significantly impacted by that of the other structure through foundation coupling.

6.5.4.16.6 Foundation Basemat and Wall Flexibility

The foundation basemat and wall flexibility for the embedded structures shall be considered in the SSI analysis unless it can be shown that ignoring such effects will not increase the dynamic response of the structure.

6.5.4.16.7 Embedment Effects

The embedment effects for foundations having a width/depth ratio of 10:1 or larger may be ignored in the SSI analysis. In these cases, the structure may be considered as surface supported.

6.5.4.16.8 Control Motion

The control motion for the SSI analysis shall be specified as stiff soil or rock out crop motion corresponding to the level of bottom of the structure base slab for the mat-supported structures or the point of pile fixity for pile-supported structures.

6.5.4.16.9 Uncertainty in SSI Analysis

For MCE and DBE analyses, soil-structure interaction shall consider the best estimate of the soil properties for developing the soil-structural models. For LDBE analysis, the uncertainties in the SSI analysis shall be considered. An acceptable method to account for these uncertainties is to vary the low-strain soil shear modulus. Low strain soil shear modulus shall be varied between the best estimate value times $(1 + C_v)$ and the best estimate value divided by $(1 + C_v)$, where C_v is a factor that accounts for uncertainties in the SSI analysis and soil properties. If sufficient, adequate soil investigation data are available, the mean and standard deviation of the low strain shear modulus shall be established for every soil layer. The C_v shall then be established so that it will cover the mean plus or minus one standard deviation for every layer. The minimum value of C_v shall be 0.5. When insufficient data are available to address uncertainties in soil properties, C_v shall be taken as no less than 1.0.

6.5.4.16.10Strain-Compatible Shear Soil Shear Modulus and Damping

The SSI analysis should consider the soil shear modulus and damping properties that are compatible with the level of effective soil shear strain seismic shaking. These properties shall be developed from acceptable ground response analysis that incorporates hysteretic soil material behavior. The use of equivalent linear method in computer program SHAKE to develop strain-compatible shear modulus and damping for the SSI analysis is considered acceptable unless for special soil conditions where there is potential for soil liquefaction or other extreme nonlinear soil behavior. In such cases, in addition to SHAKE, truly nonlinear analysis programs with proper nonlinear soil models and capability to account for pore-water pressure generation and dissipation shall also be used. Other simplified methods that use residual soil strengths and/or special P-Y curves that are calibrated against actual field performance data may be also be acceptable for special soil conditions in the SSI analysis.

6.5.5 Seismic Capacity of Structural Components

6.5.5.1 Displacement Capacity (Δ_C)

The displacement capacity, Δ_C, shall be determined by nonlinear static displacement capacity or "pushover analysis". The displacement capacity shall be defined as the controlling structure displacement that occurs when any primary element reaches its specified capacity in the pushover analysis. Specified capacity shall be considered to be reached when the concrete or steel strains of any primary element meets the limits specified in Section 6.5.5.1.2.

The displacement capacity, Δ_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, Δ_C, shall be consistent with those used to determine the displacement demand, Δ_D.

All structural members and connections shall also satisfy the capacity based performance requirements in Section 6.5.7.

6.5.5.1.1 Nonlinear Static Analysis

In determining the displacement capacity, Δ_C, using nonlinear static pushover analysis; the following procedure shall be followed:

Dead load shall be applied as an initial step. Live load with impact shall be applied to the structural system to produce the maximum effect.

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 6.5.4.6. 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 Section 3.2 of CSDC.

6.5.5.1.2 Plastic Hinge Rotational Capacity

Plastic moment capacity of ductile flexural members shall be calculated by moment-curvature (M-φ) analysis and shall conform to Section 3.3 of CSDC for concrete members and Section 6.4 of CSDC 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 either of the following allowable strain limits:

6.5.5.1.2.1 Strain Limits for Reinforced Concrete Element

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

No-Collapse Level: $\varepsilon_{su}^{a} \leq 2/3\ \varepsilon_{su}$

Safety Level: $\varepsilon_{su}^{a} \leq 0.015$

Operability Level: $\varepsilon_{su}^{a} \leq \varepsilon_{sy}$

Where: ε_{su} is the ultimate tensile strain of reinforcing steel,

ε_{sy} is the nominal yield tensile strain of reinforcing steel.

Concrete confined compressive allowable strain limit (ε_{cu}^{a}):

No-Collapse Level: $\varepsilon_{cu}^{a} \leq 2/3\ \varepsilon_{cu}$

Safety Level: $\varepsilon_{cu}^{a} \leq$ lesser of $1/3\ \varepsilon_{cu}$ $1.5\ \varepsilon_{co}$

Operability Level: $\varepsilon_{cu}^{a} \leq \varepsilon_{co}$

Where: ε_{cu} is the ultimate confined compressive strain as computed by Mander's model for confined concrete,

ε_{co} is the strain at maximum concrete compressive stress as computed by Mander's model for confined concrete.

Concrete unconfined compressive allowable strain limits (ε_{cu}^{a}):

No-Collapse Level: $\varepsilon_{cu}^{a} = 0.004$

Safety Level: $\varepsilon_{cu}^{a} = 0.003$

Operability Level: $\varepsilon_{cu}^{a} = 0.002$

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

6.5.5.1.2.2 Strain Limits for Structural Steel Elements

No-Collapse Level: $\varepsilon_{su}^{a} \leq 2/3\ \varepsilon_{su}$

Safety Level: $\varepsilon_{su}^{a} \leq 0.01$

Operability Level: $\varepsilon_{su}^{a} \leq \varepsilon_{sy}$

Where ε_{sy} = yield strain of steel, and ε_{su} = ultimate strain of steel.

6.5.5.2 Rocking

The rocking capacity of the bridge and aerial structure piers shall be determined as per Section 3.2.4.5.

6.5.5.3 Material Properties

Material properties used in calculating the capacity of structural components to resist all seismic demands shall conform to Section 3.2 of CSDC for reinforced and/or prestressed concrete components and Sections 3.2.2 and 4.2 of CSDC for structural steel components.

For seismic design purposes, the expected material properties shall be used in determining the capacity of the components. Expected material properties of structural components may also be used for the design of capacity protected members, but only when expected material properties were used to calculate the design demands.

6.5.5.4 Shear Capacity

Shear capacity of ductile components shall conform to Section 3.6 of CSDC for concrete members and Article 10.48.8 of CBDS for structural steel members.

6.5.5.5 Joint Internal Forces

Continuous force transfer through the column/superstructure and column/footing joints shall be provided for. These joint forces require that the joint have sufficient strength to ensure elastic behavior in the joint regions under the effects of the DBE determined based on the capacity of the adjacent members. This will automatically satisfy the LDBE requirements.

Joint design shall conform to Section 7.4 of CSDC.

6.5.6 Seismic Performance Evaluation

6.5.6.1 Rocking

For the No Collapse and Safety Performance Levels, when rocking is the primary seismic response mechanism, a stable rocking response must be provided, see Section 6.5.4.5.

For the Operability Performance Level, rocking of structures is not allowed.

6.5.6.2 Displacement Limits Under Earthquake

6.5.6.2.1 No Collapse Level

The maximum displacement Demand/Capacity Ratio is as follows:

$$\Delta_D / \Delta_C \leq 1.0$$

Where:

Δ_D = the displacement demand based on the frame model as defined in Section 6.5.4.1.

Δ_C = the displacement capacity based on the nonlinear static analysis model as defined in Section 6.5.5.1.

6.5.6.2.2 Safety Level

Under Design Basis Earthquake (DBE) the bridge or aerial structure shall be designed to provide for the track support by the bridge, and satisfy the requirements specified in Section 6.2.2.

6.5.6.2.3 Operability Level

Under Lower-level Design Basis Earthquake (LDBE) the displacement of the bridge shall be designed to allow the train to brake safely to a full stop from the maximum design speed and satisfy the following requirements:

- At structural expansion joints where there is a rail expansion joint, the maximum relative longitudinal deck movement under earthquake and braking or acceleration shall not exceed 1.2 inches.
- At structural expansion joints where there is no rail expansion joint, the maximum relative longitudinal deck movement under earthquake and braking or acceleration shall not exceed 1.0 inches.

6.5.6.3 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, 0.4, 0.4 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, vertical and horizontal seismic loads need not be combined. However, the designer shall evaluate the piles with the column plastic moment in the principal axes as well as along diagonal axes to determine the critical loading on the piles.

6.5.7 Seismic Design

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

6.5.7.1 Capacity 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 design factors for the evaluation of capacity-protected elements shall be applied as specified in Section 4.4 of CSDC for concrete members and Section 4.3 of CSDC for structural steel members.

The nominal moment strength of reinforced concrete capacity-protected elements shall be derived from M-φ analysis where ε_c=0.004 or ε_s=0.015, whichever is reached first. Loads shall be combined as specified under the “Extreme” load combinations specified in TM 2.3.2 - Structure Design Loads.

6.5.7.2 Soil Improvement

For foundations in soft or liquefiable soils, foundation soil improvement may be considered in the new design. Acceptable methods of foundation improvement include soil surcharge with wick drains, soil grouting (such as jet grouting, compaction grouting, chemical grouting, etc.), vibro-compaction and stone columns, displacement piles, dynamic compaction, deep soil mixing, cutter soil mixing, etc. The Geotechnical Report shall provide information relative to foundation materials and other conditions encountered in the field and provide recommendations for alternative types and methods of foundation soil improvement.

6.5.7.3 Design of Shallow Foundations

Shallow foundations shall be designed as capacity protected structural elements under any loading or combination of loadings (including seismic loads). The Geotechnical Report shall provide information and design parameters regarding shallow foundations.

When designing for footing shear, column-to-footing joint shear, and moments in footings, the column plastic moment and shear should be used with the appropriate over-strength factors applied.

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

6.5.7.4 Design of Pile Foundations

Pile foundations shall be designed as capacity protected structural elements under any loading or combination of loadings (including seismic loads). The Geotechnical Report shall provide information and design parameters regarding pile foundations.

When designing for pile cap shear, column-to-pile cap joint shear, and moments in pile cap, the column plastic moment and shear should be used with the appropriate over-strength factors applied.

Pile foundations shall be designed such that plastic hinging does not occur in the piles below ground surface.

6.5.7.4.1 Design Codes

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. The designer is encouraged to use innovative piling schemes (pile types, details, construction methodologies) where cost savings can be realized.

The Geotechnical Report shall provide information relative to foundation materials and other conditions encountered in the field in connection with recommendations for the types and lengths of piles that will be most suitable for use under the existing conditions, as appropriate. 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.

6.5.7.4.2 Ultimate Pile Load Capacity in Compression

The ultimate pile load capacity in compression shall be determined on the basis of appropriate values of skin friction plus end bearing developed from the existing or new site-specific geotechnical investigations, and shall take into consideration the tolerable total and differential structure settlement. In developing axial load capacity under seismic loading, the resistance of potentially liquefiable layers shall be ignored.

6.5.7.4.3 Negative Skin Friction

Pile load demand in compression shall be increased as appropriate to reflect down drag forces which may result from seismically induced settlement or liquefaction, embankment construction, construction dewatering or pile installation methods. When negative skin friction is considered, it shall be treated as an addition to the working load. If measures are proposed for reducing the effect of negative skin friction, these methods shall be approved by the geotechnical engineer

6.5.7.4.4 Uplift

Uplift shall not be allowed in any pile under any loading or combination of loadings under static loading condition. Piles shall be allowed to resist an intermittent uplift load due to the design earthquake. In calculating the ultimate uplift capacity of the piles, the strength of any potentially liquefiable layers resisting uplift should be limited to their residual strength. The pile-to-pile cap connection shall be designed as a rigid connection. This rigid connection shall be such that it will resist various forces acting on the head of pile, including axial compressive force, pulling force, horizontal force and moment.

6.5.7.4.5 Lateral Loads

Piles shall be designed to adequately resist the lateral loads transferred to them from the supported structure and/or from lateral soil displacement against piles. The lateral capacity of the piles shall be determined from lateral pile analysis that establishes the lateral load versus pile head deflection (P-Y curve). In performing lateral load analysis under seismic loading, appropriate p-y curves associated with potentially liquefiable layers presented in recent literature should be used. In addition, for piles founded in slopes that are susceptible to slope deformation due to liquefaction or other slope stability conditions, the effect of slope movement against the piles shall be properly considered in developing the driving and resisting forces on the piles.

When the lateral resistance of the soil surrounding the piles due to passive pressure of soil is inadequate to counteract the horizontal forces transmitted to the foundation, or when increased rigidity of the entire structure is required, battered piles may be used in conjunction with vertical piles in a pile foundation. Battered piles shall be designed to safely resist imposed loading, 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 tension demands on these piles and potential shear failure of the piles under these tension demands. Battered piles should be avoided where down drag loads are 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 lines, or interfering with existing structures or pile foundations.

6.5.7.4.6 Group Effects

Generally for piles constructed in groups, the spacing of pile centers shall be no less than 3 times the pile diameter. All piles in one group shall be the same diameter. Where pile centers are less than 3 times the pile diameter, the design of piles shall make adequate allowance for group effects. Group pile capacity shall be determined as the product of the group efficiency, number of piles in the group, and the capacity of a single pile.

For axial and lateral loading, group efficiency values of less than 1.0 may be required depending upon the type of soil, the loading condition and the center-to-center spacing of the piles. Group efficiency values for bearing capacity, settlement and/or axial and lateral loading shall be provided in the Geotechnical Report.

6.5.7.4.7 Design Load Capacity of Piles

The allowable axial load capacity of a pile for service loads shall be based on a minimum factor of safety of 2.0 relative to the ultimate pile capacity when pile load tests are performed. The allowable load capacity of a pile under the “Extreme” load combinations (per TM 2.3.2 - Structure Design Loads) shall be based on a minimum factor of safety of 1.25 relative to the ultimate pile capacity when pile load tests are performed.

6.5.7.4.8 Differential Settlement

Due to site-specific ground conditions, the foundation system may be susceptible to differential settlement. Where the potential for such a condition exists, the loads resulting from the estimated amounts of the differential settlement shall be taken into consideration if such loads result in a more critical design condition. Consideration of such loads within specific loading combinations shall be the same as for loads resulting from dead load. In all cases, the foundations of bridges or aerial structures shall be designed for settlement not to exceed that represented by:

- For simply supported multi-spans, a change in slope of 1 in 1000.
- For continuous spans, a change in slope of 1 in 1500.
- Limits required to comply with Track-Structure and Passenger Safety Criteria.

6.5.7.4.9 Horizontal Displacement

Under Design Basis Earthquake (DBE), the maximum relative horizontal displacement between pile head and pile toe shall not exceed 2.0 inches.

6.5.7.5 Expansion Joint and Hinge / Seat Capacity

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

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

In designing the expansion joints, the designer shall verify and ensure the actual displacement required is within the allowable displacement for the type of structural expansion joints selected.

Adequate seat length shall be provided to accommodate anticipated seismic displacements and prevent unseating of the structure. Seat width requirements are specified in Caltrans CSDC Section 7.2.5 and 7.8.3 for hinges and abutments respectively. Hinge restrainers shall be designed as a secondary line of defense against unseating of girders in accordance with Article 7.2.6 of 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.

6.5.7.6 Columns

In general, columns are expected to deform into the inelastic range with repairable damage. Although the displacement ductility demand on columns will be limited, they shall be designed to satisfy the detailing requirements for full-ductility structural elements as specified in CSDC Sections 7.6 and 8.

6.5.7.7 Superstructures

Capacity protected superstructure element shall remain essentially elastic.

6.5.7.8 Structural Joints

Superstructure and the bent cap joints and footing joints shall be designed to conform to the requirements of CSDC Section 7.4 and Section 7.7.1.4, respectively.

6.6 TUNNELS AND UNDERGROUND STRUCTURES

6.6.1 General

Cut-and-cover tunnels, portals and U-sections, bored and mined tunnels, and ventilation and access shafts are categorized as Primary facilities.

This document addresses preliminary seismic criteria for tunnels and underground structures for Preliminary Design. Further detailed and specific criteria are under development and will be included as future technical memoranda.

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.

6.6.2 Seismic Design Philosophy

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

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

In general, the designer allows specified structural components to undergo inelastic behavior under Maximum Considered Earthquake and Design Basis Earthquake (MCE and DBE), while force-protecting other components. The structure shall remain linear elastic under Lower-level Design Basis Earthquake (LDBE). An adequate margin of strength shall be provided between the designated load-resistance ductile mode and non-ductile failure modes. Sufficient over-strength capacity shall be provided to assure the desired ductile mechanism occurs and that the undesirable non-ductile failure mechanisms are prevented from forming.

6.6.3 Design Codes

For cut-and-cover structures, 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 sections of the CHST Project Design Criteria (CDC), CBDM shall be used.

For portals and U-sections, mined and bored tunnels, and ventilation and access shafts, seismic design code criteria are pending. However, should concrete structural elements be mainly used, then CBDM shall apply.

6.6.4 Seismic Analysis and Demand Considerations

6.6.4.1 Input Ground Displacement

Ground displacement is the primary consideration for the seismic design of underground structures. To assess the ground displacements 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.

6.6.4.2 Load and Load Combinations

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

6.6.4.3 Capacity Reduction Factors

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

6.6.4.4 Analysis Techniques

The general procedure for seismic design of underground structures shall be based primarily on the ground deformation approach specified herein. 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 of the underground structure and soil is important and shall be considered, and therefore, the effects of soil-structure interaction shall be taken into consideration.

Underground tunnel structures undergo three primary modes of deformation during seismic shaking: racking/ovaling, axial, and curvature deformations. The racking/ovaling deformation is caused primarily by seismic waves propagating perpendicular to the tunnel's longitudinal axis. Vertically propagating shear waves are generally considered the most critical type of waves for this mode of deformation. The axial and curvature deformations are induced by components of seismic waves that propagate along the longitudinal axis of the structure.

Appropriate modeling and analysis methods shall be used for static and seismic analyses of the tunnel lining and portal structures. Static analyses shall utilize the numerical models to determine member forces in the tunnel final lining, cut-and-cover structures, and portal structures for design due to self-weight, rock loads, and live loads. Two- or three-dimensional numerical models shall be used to represent the tunnel final lining ground interaction for static and seismic demands.

6.6.4.4.1 Earth Tunnel Liners - General

Earth tunnel liners shall be designed to sustain all the loads to which they will be subjected with minimum factor of safety of two. Such loads shall include:

1. Handling loads as determined by the transport and handling system.
2. Shield thrust ram loads as determined by the shield propulsion system.
3. Erection loads including external grouting loads.
4. Earth pressure shall be calculated using 2D finite element analysis methods based on the best available geotechnical data. In lieu of this computer analysis, no less than full overburden shall be used.
5. Hydrostatic pressure.
6. Self-weight of the tunnel structure.
7. Loads due to imperfect liner erection, but not less than 0.5 percent diametrical distortion.
8. Additional loads due to the driving of adjacent tunnels.
9. Effects of tunnel breakouts at cross-passages, portals, and shafts.
10. Live loads of vehicles moving in the tunnel or on the surface above it
11. Surcharge loads due to adjacent buildings.
12. Seismic loads as indicated in this document.

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

6.6.4.4.2 Rock Tunnel Liners

Temporary Support System

The temporary support systems shall be designed to sustain all the loads to which they will be subjected with minimum factor of safety of two. Such loads shall include:

1. Rock load shall be calculated using 2D finite element analysis methods based on best available geotechnical data. In lieu of this computer analysis, no less than the weight of two diameters of rock overburden shall be used.

2. Self-weight.
3. Additional loads due to the driving of adjacent tunnels.

Cast-in-Place Liners

The cast-in-place liners shall be designed to sustain all the loads to which they will be subjected with minimum factor of safety of two without beneficial effects from the initial support system. Such loads shall include:

1. Rock load shall be calculated using 2D finite element analysis methods based on best available geotechnical data. In lieu of this computer analysis, no less than the weight of two diameters of rock overburden shall be used.
2. Hydrostatic pressure either total or residual.
3. Additional loads due to the driving of adjacent tunnels (if applicable).
4. Live loads of vehicles moving in the tunnel.
5. Seismic loads as indicated in this document.

Precast Segmental Liners

The precast segmental liners shall be designed to sustain all the loads to which they will be subjected with adequate factors of safety. Such loads shall include:

1. Handling loads as determined by the transport and handling system.
2. Shield thrust ram loads if applicable as determined by the shield propulsion system.
3. Erection loads including external grouting loads.
4. Rock loads based on considerations of rock condition.
5. Hydrostatic pressure either total or residual.
6. Self-weight of the tunnel structure.
7. Loads due to imperfect liner erection.
8. Additional loads due to the driving of adjacent tunnels.
9. Live loads of vehicles moving in the tunnel.
10. Seismic loads.

6.6.4.4.3 Construction Sequence

Construction sequence including dead loads, surcharge, and potential soil arching effects shall be included prior to seismic analysis.

6.6.4.4.4 Proximity Analysis

If a tunnel is built in the vicinity of another tunnel, or underground structure, a proximity study shall be performed. With this analysis, the designer shall decide whether the interaction of the two structures needs to be considered.

6.6.4.4.5 Racking/Ovaling Analysis

The effect of shear waves propagating normal or nearly normal to the tunnel axis, resulting in a distortion of the cross-sectional shape of the tunnel lining shall be analyzed using a racking/ovaling analysis. In this analysis, soil, liner, and interface of soil and liner shall be modeled appropriately.

6.6.4.4.6 Seismic Loads due to Axial and Curvature Deformations

A global 3D model of the tunnel shall be developed using nonlinear beam elements representing the cross section of the tunnel. The model of the tunnel shall be supported by nonlinear soil springs in three orthogonal directions. The ground motions shall be applied to the ground nodes of the springs including the wave passage effect and soil properties.

6.6.4.4.7 Cross Passages and Connection Joints

The stress concentration at the connection of the cross passage and the main tunnel shall be obtained using a detailed 3D tunnel/soil continuum model.

6.6.4.4.8 Stability

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

6.6.4.4.9 Interface Joints

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

6.6.5 Cut-and-Cover Tunnels

For cut-and-cover structures, CBDM forms the basis of design.

For earth surcharge, the unit weight of earth, both above and below the groundwater table shall not be less than 130 pcf, unless specified otherwise by the Geotechnical Engineer. However, in making calculations with regard to surcharge resisting flotation of the structure, the actual unit weight of backfill placed over the structure may be used, but in no case shall be taken as greater than 120 pcf. Where full hydrostatic pressure below the groundwater table is used as a design load, a submerged design unit weight of not less than 68 pcf shall be used for earth below the groundwater table.

Prior to performing seismic analysis, the construction sequence for the tunnel dead load and surcharge shall be realistically represented.

6.6.6 Portals and U-sections

Design criteria for portals and U-sections is pending. However, should these mainly consist of reinforced concrete structures, then they shall be in accordance with CBDM, as amended by requirements in this document.

6.6.7 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. Design criteria for the seismic design of bored tunnels is pending. However, should the bored tunnels have reinforced concrete lining, then it shall be in accordance with CBDM, as amended by requirements in this document.

6.6.8 Mined Tunnels

Mined tunnels include rock tunnel sections, using either the precast concrete segmental lining or cast-in-place concrete lining. Design criteria for the seismic design of mined tunnels are pending. However, should the mined tunnels have reinforced concrete lining, then it shall be in accordance with CBDM, as amended by requirements in this document.

6.6.9 Ventilation and Access Shafts

Design criteria for ventilation and access shafts is pending. However, should the shafts have reinforced concrete lining, then they shall be in accordance with CBDM, as amended by requirements in this document.

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. Considerations 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 deposits. 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 drastically different stiffness/mass in poor ground conditions.

6.7 PASSENGER STATIONS AND BUILDING STRUCTURES

6.7.1 General

The design criteria set forth in this document govern the seismic analyses and design of the building structures within the HST system. Building structures include passenger stations and buildings below ground, at the ground level and above ground.

6.7.2 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 allows specified structural components to undergo inelastic behavior under Maximum Considered Earthquake (MCE) and Design Basis Earthquake (DBE), 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 linear elastic under LDBE. Active, semi-active and passive energy dissipation devices or base isolation systems are permitted. If employed, these devices and systems are another source of nonlinear mechanism in the structure.

An adequate margin of strength shall be provided for nonlinear elements. Enough over-strength 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 applied.

6.7.3 Design Codes

CBC methodology will be used for all non-seismic related design. However, since the CBC primarily uses force-based seismic design, FEMA 356 is referenced for the performance (i.e., strain and deformation) based seismic design methodology proposed for the CHSTP.

Although the basis of the following criteria relies heavily on FEMA 356, certain criteria might exceed those of FEMA 356. If items are not specifically addressed in this or any other section of the criteria, FEMA 356 is to be used.

6.7.4 Seismic Analysis and Demand Considerations

6.7.4.1 Analysis Techniques - General

A 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) are satisfied, all structures shall be analyzed using Nonlinear Dynamic Procedure (NDP).

6.7.4.2 Linear Dynamic Procedure (LDP)

Linear dynamic procedure shall be used in accordance with the requirements of FEMA 356. 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 FEMA 356.

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 linear time history analysis is used, the analysis shall be performed under multiple sets of ground motions, there are two options:

1. Use three sets of ground motions. The envelope maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used for design.
2. Use seven sets of ground motions. The average value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used for design.

The ground motion sets shall meet the requirements of Section 6.2.3.

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 of FEMA 356.
- Out-of-Plane Discontinuity Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1 of FEMA 356.
- Severe Weak Story Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1 of FEMA 356.
- Severe Torsional Strength Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1 of FEMA 356.
- 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 an active fault, or for ground motions with near-field pulse-type characteristics, the response spectrum method shall not be used.

6.7.4.3 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 FEMA 356. The target displacement shall be calculated by the procedure described in FEMA 356. At least two types of lateral load pattern shall be considered as described in FEMA 356. 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 should 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 that 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.

6.7.4.4 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.

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. Three sets of input ground motions shall be used for three sets of analyses using a different set of horizontal and vertical components of input ground motion. Where the relative orientation of the ground motions is not determinant, 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 multiple sets of ground motions, there are two options:

1. Use three sets of ground motions. The envelope maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used for design.
2. Use seven sets of ground motions. The average value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used for design.

The ground motion sets shall meet the requirements of Section 6.2.3.

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. Alternatively, deformation limits specified in FEMA 356 shall be used.
- For force-controlled action members the force demand shall be compared with the capacities as per FEMA 356, ACI and AISC.

6.7.4.5 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. Developing and analyzing local detailed finite element models is permitted and encouraged.

6.7.4.6 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.

6.7.4.7 Building Separation

Buildings shall be separated from adjacent structures to prevent pounding as per requirements of FEMA 356 Section 2.6.10.

6.7.4.8 Material Properties

Material properties of steel and concrete components shall be obtained from testing a number of specimens and samples. If such testing is not available, material properties used in calculating the capacity of structural components to resist all seismic demands shall conform to Sections 5 and 6 of FEMA 356 for steel and concrete structures, respectively. Expected material properties shall be based on mean values of tested material properties. Lower bound material properties shall be based on mean values of tested material properties minus one standard deviation (σ). Nominal material properties, or properties specified in construction documents, shall be taken as lower bound material properties unless otherwise specified in Chapters 5 through 8 of FEMA 356. Corresponding expected material properties shall be calculated by multiplying lower bound values by appropriate factors specified in Chapters 5 through 8 of FEMA 356 to translate from lower bound to expected values.

For seismic design purposes, the expected material properties shall be used in determining the capacity of the components. Expected material properties of structural components may also be used for the design of capacity protected members, but only when expected material properties were used to calculate the design demands.

6.7.4.9 Element Cross Section Analysis

Cross sectional properties of concrete and steel elements with nonlinear behavior, may be represented by moment-curvature curves. When developing moment-curvature curves for concrete, the effects of concrete cracking, confinement and strain hardening of the reinforcement shall be considered in accordance with CMTD 20-4 and Section 5.6 of CSDC.

Moment curvature analysis shall also be used to determine concrete and steel element cross sectional properties, including the effective area and moment of inertia, for elements that are to remain elastic.

For built-up structural steel sections, effective section properties shall be computed from stiffness analysis using local detailed finite element models. Alternatively, section properties shall be obtained from what is presented in Appendix B of CSDC.

6.7.4.10 Foundation Flexibility

The foundation flexibility reflecting the soil-foundation-structure interaction effects, including liquefaction, lateral spreading and other seismic phenomena, shall be considered as per Section 6.7.4.18. Pile 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, then the foundation may be modeled as fixed.

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 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.

6.7.4.11 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, it should be checked that the boundary conditions and element properties remain consistent with initial modeling assumptions.

6.7.4.12 Multidirectional Seismic Effects

The ground motions shall be applied concurrently in two horizontal directions and vertical direction. In the capacity/demand assessment of deformation-controlled actions, it should be considered that orthogonality effects exist and that the building is expected to sustain deformations simultaneously in the orthogonal directions. When response spectrum analysis is used, modal combination shall be performed using the CQC approach. Spatial combination shall be performed using the SRSS technique.

6.7.4.13 Load and Load Combinations

Loads and load combinations shall comply with the requirements of FEMA 356, ACI and AISC LRFD. 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.

6.7.4.14 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.

6.7.4.15 P-Δ Effects

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

6.7.4.16 Minimum Separation

Buildings shall be separated from adjacent structures to prevent pounding as per requirements specified in Section 2.6.10.1 of FEMA 356. Exempt conditions described in Section 2.6.10.2 of FEMA 356 shall not be permitted.

6.7.4.17 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 FEMA 356. The effects of overturning on foundations and geotechnical components shall be considered in the evaluation of foundation strength and stiffness as specified in FEMA 356.

6.7.4.18 Soil-Structure Interaction

Soil-structure interaction (SSI) effects shall be considered for all structures that are not supported by rock or rock-like soil foundation material having a minimum shear wave velocity of 2500 ft/sec, or as determined in the Geotechnical Data Report. In performing SSI modeling and analysis of soil-structure systems, the following criteria should be considered.

6.7.4.18.1 SSI Analysis Method

Both direct and substructuring analysis methods are acceptable for SSI analysis. Direct methods involve analyzing the total soil-structure system in one step and can be applied to both linear and nonlinear systems. In substructuring methods, the total SSI system is partitioned into two substructures, the structure and foundation. First, the foundation is analyzed to develop the foundation impedance properties and scattered motions. Then they are specified as boundary conditions in dynamic analysis of the structure. The substructuring methods, in general, are applicable only to linear systems. Nonetheless, nonlinearity of foundation soils may be accounted for by using equivalent linear method.

6.7.4.18.2 Seismic Wave Field

In the SSI analysis, the seismic wave field may be assumed to consist of vertically propagating shear and compression waves if the torsional effects due to inclined propagating waves are considered. In addition, analysis should address the effects of wave incoherence on the torsional and/or rocking response of long span structures or structures with large footprints.

6.7.4.18.3 Foundation Soil Layering and 3-D Effects

The effects of foundation soil layering and 3-D geometry on the dynamic stiffness and radiation damping of the foundation should be accounted for in the SSI analysis.

6.7.4.18.4 Nonlinear Soil Behavior

Nonlinear soil behavior should be considered in the SSI analysis. Soil nonlinearity may be assumed to consist of two parts: primary (far-field) and secondary (near-field) nonlinearities. The primary soil nonlinearity denotes the nonlinear soil behavior due to ground excitation in the absence of the structure. The secondary nonlinearity denotes the material nonlinearity induced in the soil due to SSI effects. The primary nonlinearity shall be accounted for in the SSI analysis. The secondary nonlinearity need not be considered in the SSI analysis if 1) provision for uncertainties in the soil material properties as stated in Section 6.7.4.18.9 is included in the SSI analysis, and 2) there is no potential for soil yielding in the vicinity of the structure that can result in foundation sliding, uplift, separation from side soils, etc.

6.7.4.18.5 Structure-Structure Interaction

In general, structure-structure interaction effects may be ignored in the SSI analysis unless the response of one structure can be significantly impacted by that of the other structure through foundation coupling.

6.7.4.18.6 Foundation Basemat and Wall Flexibility

The foundation basemat and wall flexibility for the embedded structures shall be considered in the SSI analysis unless it can be shown that ignoring such effects will not increase the dynamic response of the structure.

6.7.4.18.7 Embedment Effects

The embedment effects for foundations having a width/depth ratio of 10:1 or larger may be ignored in the SSI analysis. In these cases, the structure may be considered as surface supported.

6.7.4.18.8 Control Motion

The control motion for the SSI analysis shall be specified as stiff soil or rock out crop motion corresponding to the bottom level of the SSI model.

6.7.4.18.9 Uncertainty in SSI Analysis

For MCE and DBE analyses, soil-structure interaction shall consider the best estimate of the soil properties for developing the soil-structural models. For LDBE analysis, the uncertainties in the SSI analysis shall be considered. An acceptable method to account for these uncertainties is to vary the low-strain soil shear modulus per ASCE-4 recommendations:

Low strain soil shear modulus shall be varied between the best estimate value times (1 + Cv) and the best estimate value divided by (1 + Cv), where Cv is a factor that accounts for uncertainties in the SSI analysis and soil properties. If sufficient, adequate soil investigation data are available, the mean and standard deviation of the low strain shear modulus shall be established for every soil layer. The Cv shall then be established so that it will cover the mean plus or minus one standard deviation for every layer. The minimum value of Cv shall be 0.5. When insufficient data are available to address uncertainties in soil properties, Cv shall be taken as no less than 1.0.

6.7.4.18.10 Strain-Compatible Shear Soil Shear Modulus and Damping

The SSI analysis should consider the soil shear modulus and damping properties that are compatible with the level of effective soil shear strain from seismic shaking. These properties shall be developed from acceptable ground response analysis that incorporates hysteretic soil material behavior. The use of equivalent linear method in computer program SHAKE to develop strain-compatible shear modulus and damping values for the SSI analysis is considered acceptable unless for special soil conditions where there is potential for soil liquefaction or other extreme nonlinear soil behavior. In such cases, in addition to equivalent linear method, truly nonlinear analysis programs with verified nonlinear soil models and capability to account for pore-water pressure generation and dissipation shall also be used. Other simplified methods that use residual soil strengths and/or special P-Y curves that are calibrated against actual field performance data is acceptable for special soil conditions in the SSI analysis.

6.7.5 Seismic Capacity of Structural Components

The component capacities shall be computed based on methods given in Chapters 5 and 6 of FEMA 356 for steel and concrete structures, respectively. However, strain limits described in the following sections shall be used.

6.7.5.1 Strain Limits for Reinforced Concrete Elements

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

No Collapse Level: $\varepsilon_{su}^{a} \le 2/3\ \varepsilon_{su}$

Safety Level: $\varepsilon_{su}^{a} \le 0.015$

Operability Level: $\varepsilon_{su}^{a} \le \varepsilon_{sy}$

Where:

ε_{su} is the ultimate tensile strain of reinforcing steel,

ε_{sy} is the nominal yield tensile strain of reinforcing steel

Concrete confined compressive allowable strain limit (ε_{cu}^{a}):

No Collapse Level: $\varepsilon_{cu}^{a} \le 2/3\ \varepsilon_{cu}$

Safety Level: $\varepsilon_{cu}^{a} \le$ lesser of $1/3\ \varepsilon_{cu}$ $1.5\ \varepsilon_{co}$

Operability Level: $\varepsilon_{cu}^{a} \le \varepsilon_{co}$

Where:

ε_{cu} is the ultimate confined compressive strain as computed by Mander's model for confined concrete

ε_{co} is the strain at maximum concrete compressive stress as computed by Mander's model for confined concrete

Concrete unconfined compressive allowable strain limit (ε_{cu}^{a}):

No Collapse Level: $\varepsilon_{cu}^{a} = 0.004$

Safety Level: $\varepsilon_{cu}^{a} = 0.003$

Operability Level: $\varepsilon_{cu}^{a} = 0.002$

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

6.7.5.2 Strain Limits for Structural Steel Elements

Structural steel allowable strain limit (ε_{su}^{a}):

No Collapse Level: $\varepsilon_{su}^{a} \le 2/3\ \varepsilon_{su}$

Safety Level: $\varepsilon_{su}^{a} \le 0.01$

Operability Level: $\varepsilon_{su}^{a} \le \varepsilon_{sy}$

Where:

ε_{sy} = yield strain of steel
ε_{su} = ultimate strain of steel

6.7.5.3 Capacity of Members with Force-Controlled Action

Axial force, bending moment and shear capacities shall be computed in accordance with the requirement of FEMA 356.

6.7.5.4 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 design factors for the evaluation of capacity protected elements shall be applied as specified in Section 4.4 of CSDC for concrete members and Section 4.3 of CSDC for structural steel members. For No Collapse Level performance, the nominal moment strength of reinforced concrete capacity-protected elements shall be derived from M-φ analysis where ε_c=0.004 or ε_s=0.015, whichever is reached first.