Chapter 11

Seismic

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Acronyms

ASCE	American Society of Civil Engineers
Caltrans	California Department of Transportation
CBC	California Building Code
CBDM	Caltrans Bridge Design Manual
CQC	Complete quadratic combination
CSDC	Caltrans Seismic Design Criteria
D/C	Demand/Capacity
ELTHA	Equivalent linear time history analysis
ESA	Equivalent static analysis
HST	High-Speed Train
LDP	Linear Dynamic Procedure
MCE	Maximum Considered Earthquake
NCL	No Collapse Performance Level
NDP	Non-linear Dynamic Procedure
NSP	Non-linear Static Procedure
NLTHA	Non-linear time history analysis
OBE	Operating Basis Earthquake
OPL	Operability Performance Level
RSA	Response Spectrum Analysis
SDAP	Seismic Design and Analysis Plan
SRSS	Square root of the sum of the squares
SSI	Soil-structure interaction

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11 Seismic

11.1 Scope

This chapter provides seismic design criteria for Primary Type 1, Primary Type 2, and Secondary elements of infrastructure, as defined in Section 11.4.1.

For seismic design criteria for tunnels and underground structures, see the *Geotechnical* chapter.

For seismic design criteria for earthen or soil supporting structures, such as retaining walls, embankments, cut and existing slopes, and reinforced earth structure, see the *Geotechnical* chapter.

11.2 Regulations, Codes, Standards, and Guidelines

Refer to the *General* chapter for requirements pertaining to regulations, codes, and standards. The provisions within this chapter shall govern the design. Provisions in the following current document versions shall also be considered as guidelines when sufficient criteria are not provided by this chapter.

- American Concrete Institute (ACI), Building Code Requirements for Reinforced Concrete, ACI 318
- American Welding Society (AWS) Codes
  - AWS D1.1/D1.1M: Structural Welding Code-Steel
  - AWS D1.8/D1.8M: Structural Welding Code-Seismic Supplement
- American Association of State Highway and Transportation Officials (AASHTO)/AWS D1.5M/D1.5: Bridge Welding Code
- American Association of State Highway and Transportation Officials (AASHTO), Guide Specifications for Seismic Isolation Design
- American Association of State Highway and Transportation Officials (AASHTO), Guide Specifications for LRFD Seismic Bridge Design
- California Building Code (CBC)
- American Institute of Steel Construction (AISC), Manual of Steel Construction
- American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering
- American Society of Civil Engineers (ASCE) 41: Seismic Rehabilitation of Existing Structures
- California Department of Transportation (Caltrans) Bridge Design Manuals (CDBM)

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- Caltrans Bridge Design Specification (CBDS): American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, with California Amendments.
- Caltrans Bridge Memo to Designers Manual (CMTD)
- Caltrans Bridge Design Practices Manual (CBPD)
- Caltrans Bridge Design Aids Manual (CBDA)
- Caltrans Bridge Design Details Manual (CBDD)
- Caltrans Standard Specifications
- Caltrans Standard Plans
- Caltrans Seismic Design Criteria (CSDC)

11.3 Seismic Design and Analysis Plan

The Designer shall develop a Seismic Design and Analysis Plan (SDAP) for the element of infrastructure.

The SDAP shall define:

- the General Classification as Primary (Type 1 or Type 2), or Secondary, as defined in Section 11.4.1
- the Technical Classification as Complex, Standard, or Non-Standard, as defined in Section 11.4.2

The SDAP shall contain detailed commentary on seismic analysis for each design earthquake, indicating the analysis software to be used, and the modeling assumptions and techniques to be employed.

The SDAP shall contain commentary as to the suitability of linear versus nonlinear analysis, considering geo-hazards, the severity of design ground motions, induced strains in the soil and structure, expected nonlinearities, and expected inelastic behavior.

The SDAP shall define the pre-determined mechanism for seismic response (i.e.: plastic hinging, foundation rocking, etc.) and the regions of targeted inelastic response.

Seismic related design variances shall be submitted per the *General* chapter. The SDAP shall justify all seismic related design variances. For retrofit of existing structures, the SDAP shall provide a detailed discussion of the extent of retrofit and the proposed methodology to verify seismic performance. The required amount of retrofit will be determined by the Contracting Officer on a case by case basis.

If energy dissipation, seismic response modification, or base isolation systems are proposed, then a design variance shall be submitted per the *General* chapter. The SDAP shall discuss in detail the proposed use of any such system, including the nonlinear response, and the capacity

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under service (i.e., braking and acceleration, wind, etc.) loads and OBE events in order to meet criteria in the Track-Structure Interaction section of the *Structures* chapter.

11.4 Design Classifications

Elements of HST infrastructure will provide a broad range of functions for the HST system.

General and technical classification provides a method to differentiate between seismic design objectives for the various elements of HST infrastructure.

11.4.1 General Classifications

Elements of HST infrastructure, based on their importance to HST service, shall be generally classified as Primary Type 1, Primary Type 2, or Secondary.

Primary Type 1 – Primary Type 1 elements are those that directly support HST track, including, but not limited, to:

- Bridges, aerial structures, and grade separations that directly support HST track.
- Tunnels and underground structures that directly support HST track.
- Passenger stations and building structures that directly support HST track.
- Earthen or soil supporting structures, such as retaining walls, embankments, cut and existing slopes, and reinforced earth structure, that directly support HST track.

For additional seismic design criteria for Primary Type 1 elements, see the Track-Structure Interaction section of the *Structures* chapter.

Primary Type 2 – Primary Type 2 elements are those that do not directly support HST track, but have the potential to affect HST service, including, but not limited, to:

- Highway, roadway, freight, and pedestrian structures that span over HST track.
- Train control, traction power, communication, operation, control, and equipment facilities essential for HST service.
- Tunnels and underground structures near HST track, where potential damage could affect HST service
- Building structures near HST track, where potential damage could affect HST service.
- Earthen or soil supporting structures near HST track or facilities, such as retaining walls, embankments, cut and existing slopes, and reinforced earth structure, where potential damage could affect HST track or service.
- Structures or improvements not supporting HST track, but essential for HST service, including, but not limited, to:
  - Train control, communication and operation facilities

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  - Traction power facilities
  - Other equipment facilities essential for HST service

Secondary – Secondary elements are those not designated as Primary Type 1 or Type 2, including, but not limited, to:

- Highway, roadway, freight, and pedestrian structures that do not support or span over HST track, where potential damage would not affect HST service.
- Tunnels and underground structures removed from HST track, where potential damage would not affect HST service.
- Building structures removed from HST track, where potential damage would not affect HST service, including, but not limited to:
  - Administrative buildings
  - Shop and maintenance buildings
  - Storage facilities
  - Parking structures
  - Training facilities
  - Other buildings or facilities not essential for HST service.
- Earthen or soil supporting structures removed from HST track, such as retaining walls, embankments, cut and existing slopes, and reinforced earth structure, where potential damage would not affect HST track.

Secondary structures owned by the Authority shall be subject to the seismic criteria in this chapter.

Secondary structures owned by Third Parties outside of HST right-of-way shall be subject to the applicable Third Party seismic criteria. Secondary structures owned by Third Parties within HST right-of-way shall be subject to the applicable Third Party seismic criteria and the seismic criteria in this chapter, whichever governs.

11.4.2 Technical Classification

Structures shall be technically classified, in order to determine the scope of seismic design requirements.

Complex Structures – Structures that have complex response during seismic events are considered Complex. Complex structural features include:

- Irregular Geometry – Structures that include multiple superstructure levels, variable width or bifurcating superstructures, tight horizontal curves (inside radius of curvature < 400 ft),

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large subtended horizontal angles (angle > 30°), or adjacent frames with corresponding transverse or longitudinal fundamental periods of vibration varying by greater than 25%.

- Unusual Framing – Structures with straddle, outrigger, or C-bent supports, or unbalanced mass and/or stiffness distribution.
- Short Columns – Structures with concrete columns having a ratio of height to least cross sectional dimension (H/D) less than 2.5.
- Tall Columns – Structures with concrete columns having a ratio of height to least cross sectional dimension (H/D) > 10 in single curvature, or > 15 in double curvature.
- Long Span Structures – Structures that have spans greater than 300 feet.
- Skewed Structures – Structures with skewed bents or abutments > 15 degrees.
- Lightweight Concrete – Structures that consist of lightweight concrete. Lightweight concrete shall not be used for ductile earthquake resisting elements.
- Energy Dissipation, Seismic Modification, or Base Isolation Devices – Structures using energy dissipation, seismic response modification, or base isolation devices.
- Unusual Foundation Systems – Structures with foundations other than spread footings, caissons, piles, or drilled shafts.
- Unusual Geologic Conditions – Structures that are subject to unusual geologic conditions, including geologic hazards as defined within the Geotechnical Reports required by the *Geotechnical* chapter, including, but not limited, to:
  - Soft, collapsible, or expansive soils
  - Soil having moderate to high liquefaction and other seismically induced ground deformation potential
  - Soil of significantly varying type over the length of the structure
- Tunnels or Underground Structures
- Structures at or in Close Proximity to Active Faults – See the *Geotechnical* chapter for Active Fault definition.

<u>Standard Structures</u> – Structures that are not Complex Structures and comply with pending CHSTP Design Guidelines for Standard Aerial Structures.

<u>Non-Standard Structures</u> – Structures that are not Complex or Standard Structures.

11.5 Seismic Design Approach

The goal of these criteria is to safeguard against loss of life and major failures due to Maximum Considered Earthquake (MCE), and interruption of HST operations due to structural or track damage and derailment caused by Operating Basis Earthquake (OBE).

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11.5.1 Seismic Performance Criteria

Two levels of seismic performance criteria are defined:

- No Collapse Performance Level (NCL) – For the NCL, Table 11-1 states the performance objectives and acceptable damage for Maximum Considered Earthquake (MCE) design.

Table 11-1: Performance Objectives/Acceptable Damage for NCL

Performance Level & Design Earthquake	Performance Objectives	Acceptable Damage
No Collapse Performance Level (NCL) Maximum Considered Earthquake (MCE)	<u>Primary Type 1</u> • No structural collapse. • Occupants not on trains shall be able to evacuate the structure safely. • Damage and collapse due to train derailment shall be mitigated through containment design. • If derailment occurs, train passengers and operators shall be able to be evacuated from derailed trains safely. • Reliable access for post-earthquake emergency services shall be provided for within design. • Extensive repairs or complete replacement of some system components may be required before train operation may resume. • For underground structures, no flooding or mud inflow. <u>Primary Type 2</u> • No structural collapse. • Occupants shall be able to evacuate the structure safely. • Damage and collapse due to train derailment shall be mitigated through containment design. • Reliable access for post-earthquake emergency services shall be provided for within design. • Extensive repairs or complete replacement of some system components may be required before train operation may resume. • For underground structures, no flooding or mud inflow. <u>Secondary</u> • No structural collapse. • Occupants shall be able to evacuate the structure safely. • Reliable access for post-earthquake emergency services shall be provided for within design. • Extensive repairs or complete replacement may be required. • For underground structures, no flooding or mud inflow.	Significant yielding of reinforcement steel or structural steel. Minor fracturing of secondary and redundant steel members or rebar, with no collapse. Extensive cracking and spalling of concrete, with minimal loss of vertical load carrying capability. Large permanent offsets, without collapse Extensive damage to HST track, track support, and rail fasteners

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- Operability Performance Level (OPL) – For the OPL, Table 11-2 states the performance objectives and acceptable damage for Operating Basis Earthquake (OBE) design.

Table 11-2: Performance Objectives/Acceptable Damage for OPL

Performance Level	Performance Objectives	Acceptable Damage
Operability Performance Level (OPL) Operating Basis Earthquake (OBE)	<u>Primary Type 1</u> • Essentially elastic structural response • Occupants not on trains shall be able to evacuate the structure safely. • No derailment, trains shall be able to safely brake from the maximum design speed to a safe stop. • Train passengers and operators shall be able to evacuate stopped trains safely. • Primary (Type 1) structure and track designed to comply with Track-Structure Interaction section of the *Structures* chapter. • Minimal disruption of service for all systems supporting HST operations. • Resumption of HST operations within a few hours with the possibility of reduced speeds. • For underground structures, no flooding or mud inflow <u>Primary Type 2</u> • Essentially elastic structural response • Occupants shall be able to evacuate the structure safely. • Minimal disruption of service for all systems supporting HST operations. • Resumption of HST operations within a few hours with the possibility of reduced speeds. • For underground structures, no flooding or mud inflow <u>Secondary</u> • OPL does not apply	Minor inelastic behavior, no spalling. No damage to HST track, track support, and rail fasteners Negligible permanent deformation of substructure and superstructure components.

11.5.2 Design Earthquakes

For design, two design earthquakes, the Maximum Considered Earthquake (MCE) and the Operating Basis Earthquake (OBE), are defined:

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

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11.5.3 Retrofit of Existing Structures

See the *Structures* chapter for requirements for Primary Type 2 or Secondary structures not supporting HST, which addresses retrofit of existing structures.

Existing Primary Type 2 structures shall be subject to both NCL/MCE and OPL/OBE design. Existing Secondary structures shall be subject to NCL/MCE design only.

A detailed discussion on the retrofit of existing structures shall be defined in the SDAP, as discussed in Section 11.3.

11.5.4 Seismic Requirements for Temporary Construction Structures

Temporary construction structures include new temporary structures and the temporary shoring and underpinning of existing structures.

For seismic requirements for temporary construction structures, the following design spectra and motions:

- 125% of the OBE design spectra and motions, or
- a scaled OBE spectra where the peak ground acceleration (PGA) equals 0.1g

whichever governs, shall apply.

For temporary construction structures, the performance requirements for NCL as given in Table 11-1 for Secondary Structures shall apply.

11.6 Seismic Design Requirements

For each general classification, Table 11-3 defines seismic design requirements for each seismic performance level and design earthquake.

Note that for Primary Type 1 generally classified structures, TSI/OBE refers to track and structure seismic performance during OBE events as defined in Track-Structure Interaction section of the *Structures* chapter.

Table 11-3: Seismic Design Requirements

General Classification		
Primary Type 1	**Primary Type 2**	**Secondary**
NCL/MCE	NCL/MCE	NCL/MCE
OPL/OBE	OPL/OBE	--
TSI/OBE	--	--

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11.7 Bridges, Aerial Structures, and Grade Separations

All Primary Type 1 or Type 2 bridges, aerial structures, and grade separations shall be subject to both NCL/MCE and OPL/OBE seismic criteria herein.

All Secondary bridges, aerial structures, and grade separations owned by the Authority shall be subject to the NCL/MCE seismic criteria herein.

All Secondary bridges, aerial structures, and grade separations owned by Third Parties shall be subject to the applicable Third Party seismic criteria.

11.7.1 Design Codes

For NCL/MCE design, current Caltrans performance based design methods as given in CBDM form the basis of design. Certain criteria herein exceed those of CBDM. For items not specifically addressed in this or other Design Criteria chapters, CBDM shall be used. See the *Structures* chapter for the load combination including MCE events.

For OPL/OBE design, current Caltrans force based design methods as given in AASHTO LRFD with Caltrans Amendments form the basis of design. Certain criteria herein exceed those of AASHTO LRFD with Caltrans Amendments. See the *Structures* chapter for the load combinations including OBE events.

Table 11-4 summarizes the applicable seismic design code for each General Classification.

Table 11-4: Applicable Bridge, Aerial Structure and Grade Separation Design Codes

Performance/ Design Earthquake	General Classification		
	Primary Type 1	**Primary Type 2**	**Secondary**
NCL/MCE	CBDM	CBDM	CBDM
OPL/OBE	AASHTO/Caltrans	AASHTO/Caltrans	--
TSI/OBE	*Structures* chapter	--	--

11.7.2 Seismic Design Approach

The seismic design approach differs depending upon the design earthquake.

11.7.2.1 NCL/MCE Design Approach

For NCL/MCE design, the approach shall be:

- The structure shall have a clearly defined and pre-determined mechanism for seismic response, with targeted regions for inelastic response.
- Inelastic behavior shall be limited to columns, piers, and abutments, at above soil or water surface locations.

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- Regions adjacent to inelastic behavior shall be capacity protected and perform as essentially elastic.
- Seismic design and detailing requirements per CSDC shall be satisfied.

Allowable pre-determined mechanisms for NCL/MCE design include:

- Flexural Plastic Hinging
- Foundation Rocking (see Section 11.7.2.1)
- Energy dissipation, seismic response modification, or base isolation systems

A. Flexural Plastic Hinging

Flexural plastic hinging shall be limited to the columns or piers. The location of plastic hinges shall be at above soil or water surface locations accessible for inspection and repair.

Non-fusing or capacity protected members shall be designed to prevent brittle failure mechanisms, such as footing shear, joint shear, column shear, tensile failure at the top of concrete footings, and unseating of girders. Non-fusing or capacity protected members shall be designed as essentially elastic, with 120% over-strength factor on the column plastic moment and shear applied.

Modeling, analysis and design shall conform to CBDM and CSDC.

For exceptionally soft soil sites, if below ground plastic hinging is unavoidable in caissons, piles or drilled shafts, then a design variance shall be submitted per the *General* chapter. All seismic related design variances shall be identified and justified in the SDAP, as required in Section 11.3.

B. Foundation Rocking

Foundation rocking shall not be allowed for Primary Type 1, or Complex structures.

Stable foundation rocking shall be allowed for NCL/MCE and OPL/OBE design for:

- Primary Type 2 or Secondary, and Standard or Non-Standard structures
- No skew
- Single column piers or multiple column bents, where most of the structural mass is concentrated at a single level.

In absence of rocking, Equivalent Static Analysis (ESA) as defined in Section 11.7.3.15 would apply to the structure.

See Section 11.7.3.16 for foundation rocking methodology.

11.7.2.2 OPL/OBE Design Approach

For OPL/OBE design, the approach shall be:

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- The structure shall respond as essentially elastic.
- For Primary Type 1 structures, the structure and track seismic performance during OBE events shall comply with the Track-Structure Interaction section of the Structures chapter.
- Foundation Rocking (see Section 11.7.2.1)

OPL/OBE demands shall be compared versus force-based capacities calculated per AASHTO LRFD with Caltrans Amendments per Section 11.7.5.3.

11.7.2.3 Energy Dissipation, Seismic Response Modification, or Base Isolation Systems

Energy dissipation, seismic response modification, or base isolation systems can be used to minimize damage, reduce seismic demands on substructures, and reduce foundation costs.

For seismic isolation design, CBDS implemented AASHTO's Guide Specifications for Seismic Isolation Design shall apply.

Energy dissipation, seismic response modification, or base isolation systems shall contain sufficient capacity under service (i.e., braking and acceleration, wind, etc.) loads and OBE events, in order to meet criteria in the Track-Structure Interaction section of the *Structures* chapter.

If energy dissipation, seismic response modification, or base isolation systems are proposed, then the use of such systems shall be identified in the SDAP, as required in Section 11.3.

11.7.3 Seismic Demands on Structural Components

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

For NCL/MCE design of Complex structures, NLTHA shall apply. For NCL/MCE design of Standard or Non-Standard structures, the appropriate analysis technique will depend upon the site specifics and structure.

For OPL/OBE design of the structure, a linear elastic analysis technique may be used. For Primary Type 1 structures, due to non-linear rail fastener slippage during OBE events (see the Track-Structure Interaction section of the *Structures* chapter), NLTHA shall apply.

11.7.3.1 Force Demands (F_U) for OPL/OBE

For OPL/OBE design, the ultimate force demand, F_u, shall be determined for all structural components.

For the structure, the loading combination shall be as specified in the *Structures* chapter.

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For Primary Type 1 structures, the loading combinations for structure and track seismic demands during OBE events shall be as specified in the Track-Structure Interaction section of the *Structures* chapter

11.7.3.2 Displacement Demands (Δ_D) for NCL/MCE

For NCL/MCE design, the global or local displacement demand, Δ_D, at the center of mass of the superstructure for each bent shall be determined, and compared versus the displacement capacity, Δ_C.

The loading combination shall be as specified in the *Structures* chapter.

11.7.3.3 Vertical Earthquake Motions

Where the MCE peak rock acceleration is 0.6g or greater, an equivalent static vertical load per CSDC shall be applied to the superstructure for design in order to estimate the effects of vertical acceleration.

For structures at or in close proximity to hazardous earthquake faults, as defined in the *Geotechnical* chapter, vertical motions shall be considered. This applies to the structural loading combinations as specified in the *Structures* chapter, and the loading combinations for structure and track seismic demands during OBE events as specified in the Track-Structure Interaction section of the *Structures* chapter.

11.7.3.4 Effective Sectional Properties

For NCL/MCE design, cracked bending and torsional moments of inertia for ductile substructure, and superstructure concrete members shall be per CSDC.

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

For structural steel sections, either moment-curvature analysis shall be performed which considers the stress-strain relationship of the structural steel, or effective section properties derived based upon the degree of nonlinearity shall be used. Effective section properties for structural steel components shall be consistent with CBDS and AISC Manual of Steel Construction.

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

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11.7.3.5 Mass

Both elemental and lumped mass shall be used in analysis.

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

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

Where applicable, train mass shall be considered per Section 11.7.3.12.

11.7.3.6 Material Properties for Demands

For NCL/MCE design, expected material properties shall be used in calculating the structural seismic demands in conformance with CSDC for concrete members and AASHTO LRFD with Caltrans Amendments for structural steel members.

For OPL/OBE design, nominal material properties shall be used in calculating the structural seismic demands.

11.7.3.7 Flexural Plastic Hinging

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

11.7.3.8 Assessment of Track-Structure Interaction

For assessment of train and track-structure interaction, including requirements and load combinations which include OBE events, see the Track-Structure Interaction section of the *Structures* chapter.

11.7.3.9 Foundation Stiffness

Foundation stiffness, including the effects of soil structure interaction if applicable, shall be considered for seismic analyses per the *Geotechnical* chapter.

11.7.3.10 Boundary Conditions

In cases where the structure is adjacent to or connected to other structures which are not included in the model (e.g.: adjacent spans or abutments), the model shall also contain appropriate elements at its boundaries to capture mass and stiffness effects of the adjacent structures.

For NCL/MCE abutment design, longitudinal and transverse response shall be per CSDC.

For OPL/OBE design of Primary Type 1 structures, abutment response shall be elastic.

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

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11.7.3.11 Continuous Welded Rail

For structures that carry continuously welded rail, there may be benefits to structural performance during a seismic event provided by the rail system. The rails may serve as restrainers at the expansion joints tying adjacent frames together under seismic loading. However, this is complex behavior, which shall be substantiated and validated through analysis.

Since the rail system seismic response at the expansion joints is highly nonlinear, linear elastic analysis is not appropriate. Instead a nonlinear time-history analysis, in accordance with Section 11.7.3.19, shall be performed which considers track-structure interaction.

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

11.7.3.12 Train Mass and Live Load

For NCL/MCE design, train mass and live load shall not be considered.

For OPL/OBE design, train mass and live load shall be considered per load combinations defined in the *Structures* chapter. For all OBE load combinations, both strength and track-structure interaction per the *Structures* chapter, train loads may be modeled as equivalent static distributed loads, and train mass as stationary mass. Although equivalent distributed loads are used in the analysis, local design shall account for any effects due to actual concentrated axle loads.

For OBE strength load combinations, the following train effects shall be considered simultaneously:

<u>Single track structures</u>:

- One train vertical live load + impact
- One train longitudinal braking force
- Mass of 1 train, applied at the center of train mass

<u>Multiple track structures:</u>

1/2 of trains potentially occupying the structure shall be considered. Where an odd number of trains potentially occupy the structure, round down to the nearest whole number of trains (example: for 3 trains, use 1/2(3) = 1.5 and round down to 1).

- 1/2 of the trains live load + impact
- 1/2 of trains longitudinal braking force

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- Mass of 1/2 of the trains, applied at the center of train mass

For OBE track-structure interaction load groups per the *Structures* chapter, the following train effects shall be considered simultaneously:

- One train vertical live load + impact
- One train longitudinal braking force, where applicable (Rail-Structure Interaction Only)
- Mass of 1 train, applied at the center of train mass

11.7.3.13 P-Δ Effects

For flexural plastic hinging, P-Δ effects shall be considered and conform to the requirements in CSDC.

For foundation rocking response, P-Δ effects shall be considered as per Section 11.7.3.16.

11.7.3.14 Displacement Demand Amplification Factor

When ESA or RSA is used for NCL/MCE or OPL/OBE design, the displacement demand, Δ_D, obtained shall be multiplied by an amplification factor, C, as follows:

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

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

Where:

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

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

This amplification factor is used in order to account for the uncertainty associated with the calculation of structural period for stiff structures. This amplification factor shall not apply to foundation rocking analysis.

11.7.3.15 Equivalent Static Analysis

Equivalent Static Analysis (ESA) may be used to determine earthquake demands, E:

- For NCL/MCE design, the Displacement Demand, Δ_D, at the center of mass of the superstructure.
- For OPL/OBE design, the Force Demands, F_u

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

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For NCL/MCE and OPL/OBE design, ESA shall apply to Standard or Non-Standard structures with the following characteristics:

- No skew.
- Single column piers or multiple column bents where most of the structural mass is concentrated at a single level.
- The fundamental mode of vibration is uniform translation.

A simply defined lateral force distribution due to balanced spans, and approximately equal bents.ESA shall not apply to Complex structures.

ESA earthquake demands shall be determined from horizontal spectra by either of 2 methods:

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

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

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

For calculation of ESA earthquake demands for both Methods 1 and 2:

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

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

Where:

C	=	the amplification factor, C, given in Section 11.7.3.14
S_a^L	=	longitudinal acceleration response spectral value at period T_L
T_L	=	fundamental period of structure in the longitudinal direction (including foundation stiffness)
S_a^T	=	transverse acceleration response spectral value at period T_T
T_T	=	fundamental period of structure in the transverse direction (including foundation stiffness)
W	=	tributary dead load + superimposed dead load for NCL/MCE design
W	=	tributary dead load + superimposed dead load + train mass and live load per Section 11.7.3.12 for OPL/OBE design.

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Effective sectional properties shall be used per Section 11.7.3.4. Material properties shall be used per Section 11.7.3.6.

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

For both NCL/MCE and OPL/OBE design, 5% damped response spectra, as given in the Geotechnical Reports required by the *Geotechnical* chapter, shall be used to determine S_a.

11.7.3.16 Foundation Rocking

For NCL/MCE and OPL/OBE design, where foundation rocking is allowed per Section 11.7.2.1, the procedure presented in AASHTO's Guide Specifications for LRFD Seismic Bridge Design, Appendix A: Foundation-Rocking Analysis shall be used for design. Within this reference, design for P-Δ effects and column plastic hinging requirements are outlined.

For NCL/MCE design, should column plastic hinging occur concurrently with foundation rocking response, then all non-fusing or capacity protected members including the foundation, if applicable, shall be designed as essentially elastic, with 120% over-strength factor on the column plastic moment and shear applied.

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

The Geotechnical Reports required by the *Geotechnical* chapter shall provide information and design parameters regarding foundation rocking.

11.7.3.17 Response Spectrum Analysis

Response Spectrum Analysis (RSA) shall be used to determine earthquake demands, E:

- For NCL/MCE design, the Displacement Demand, Δ_D, at the center of mass of the superstructure
- For OPL/OBE design, the Force Demands, F_u

when ESA provides an unrealistic estimate of the dynamic behavior.

For NCL/MCE and OPL/OBE design, RSA shall apply to Standard or Non-Standard structures with the following characteristics:

- Skewed bents or abutments ≤ 15 degrees
- Single column pier or multiple column bents
- Response primarily captured by the fundamental structural mode shapes containing a minimum of 90% mass participation in the longitudinal and transverse directions

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For NCL/MCE design, RSA shall not apply to Complex structures.

For OPL/OBE design, RSA may apply to Complex structures, upon approval of the SDAP per Section 11.3.

RSA involves creating a linear, three dimensional dynamic model of the structure, with appropriate representation of all material properties, structural stiffness, mass, boundary conditions, and foundation characteristics. The dynamic model is used to determine the fundamental structural mode shapes. A sufficient number of mode shapes shall be included to account for a minimum of 90% mass participation in the longitudinal and transverse directions.

Care shall be taken to ensure 90% mass participation for long aerial structure models. The Designer shall examine the mode shapes to ensure that they sufficiently capture the behavior of the structure.

A linear elastic multi-modal spectral analysis shall be performed using the 5% damped response spectra, as given in the Geotechnical Reports required by the *Geotechnical* chapter. The modal response contributions shall be combined using the complete quadratic combination (CQC) method.

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

Where there is a change in soil type along the bridge alignment, consideration shall be made to the possibility that out-of-phase ground displacements at 2 adjacent piers may increase the computed demand on expansion joints, rails or columns. This effect is not explicitly considered in RSA. In such cases, more sophisticated time history analyses shall be used.

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

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

RSA demands shall be determined from horizontal spectra by either of the 2 following methods:

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

$$\text{Case 1 : } E = 1.0E_L + 0.3E_T$$

$$\text{Case 2 : } E = 0.3E_L + 1.0E_T$$

For calculation of RSA earthquake demands:

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

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

Where:

C = the amplification factor, C, given in Section 11.7.3.14

Effective sectional properties shall be used per Section 11.7.3.4. Material properties shall be used per Section 11.7.3.6.

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

For NCL/MCE design, dead and superimposed dead loads shall be applied as an initial condition.

For OPL/OBE design, in addition to dead and superimposed dead loads, train mass and live load shall be considered per Section 11.7.3.12.

For NCL/MCE and OPL/OBE design, modal damping shall be 5%.

11.7.3.18 Equivalent Linear Time History Analysis

Equivalent Linear Time History Analysis (ELTHA) shall be used to determine earthquake demands, E:

- For NCL/MCE design, the Displacement Demand, Δ_D, at the center of mass of the superstructure
- For OPL/OBE design, the Force Demands, F_u

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

For NCL/MCE and OPL/OBE design, ELTHA shall apply to Standard or Non-Standard structures with the following characteristics:

- Skewed bents or abutments ≤ 15 degrees

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- Single column pier or multiple column bents

For NCL/MCE design, ELTHA shall not apply to Complex structures.

For OPL/OBE design, ELTHA may apply to Complex structures, upon approval of the SDAP per Section 11.3.

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

For NCL/MCE and OPL/OBE design, motions consistent with the 5% damped response spectra shall be used, as given in the Geotechnical Reports required by the *Geotechnical* chapter.

For NCL/MCE and OPL/OBE design, damping shall be 5%.

Should Rayleigh damping be used for ELTHA, it requires the calculation of both stiffness and mass proportional coefficients anchored at 2 structural frequencies, which shall envelope all important modes of structural response. The lower anchoring frequency (i.e., longest period) shall be determined using effective section properties per Section 11.7.3.4. and by reducing the resulting lowest natural frequency by 10%. The higher anchoring frequency (i.e., shortest period) shall be chosen such that a minimum of 90% mass participation in the longitudinal, transverse directions are mobilized.

Effective sectional properties shall be used per Section 11.7.3.4. Material properties shall be used per Section 11.7.3.6.

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

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

For NCL/MCE design, dead and superimposed dead loads shall be applied as an initial condition.

For OPL/OBE design, in addition to dead and superimposed dead loads, train mass and live load shall be considered per Section 11.7.3.12.

The time histories shall reflect the characteristics (fault distance, site class, moment magnitude, spectral shape, rupture directivity, rupture mechanisms, and other factors) of the controlling

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design earthquake ground motions, as given in the Geotechnical Reports required by the *Geotechnical* chapter.

The motions shall consist of 2 horizontal ground motion time histories, selected, scaled, and spectrally matched. The 2 horizontal components of the design ground motions shall be representative of the fault-normal and fault-parallel motions at the site, as appropriate, and transformed considering the orientation of the motions relative to the local or global coordinate systems of the structural model.

Consideration of vertical earthquake motions shall be considered per Section 11.7.3.3.

When ELTHA is used, seven (7) sets of ground motions shall be used, and the average maximum values of each response parameter (e.g., force or strain in a member, displacement or rotation at a particular location) shall be used for design.

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

11.7.3.19 Nonlinear Time History Analysis

Nonlinear Time History Analysis (NLTHA) shall be used to determine earthquake demands, E:

- For NCL/MCE design, the Displacement Demand, Δ_D, at the center of mass of the superstructure
- For OPL/OBE design, the Force Demands, F_u

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

For NCL/MCE design, NLTHA shall apply to Complex structures.

For OPL/OBE design, NLTHA, ELTHA, or RSA may apply to Complex structures, upon approval of the SDAP per Section 11.3.

For TSI/OBE design of Primary Type 1 structures, due to required track and structure seismic performance during OBE events per the Track-Structure Interaction section of the *Structures* chapter, NLTHA shall be used.

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

For NCL/MCE and OPL/OBE design, motions consistent with the 5% damped response spectra shall be used, as given in the Geotechnical Reports required by the *Geotechnical* chapter.

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For NCL/MCE and OPL/OBE design, damping shall be 5%.

Should Rayleigh damping be used for NLTHA, it requires the calculation of both stiffness and mass proportional coefficients anchored at 2 structural frequencies, which shall envelope all important modes of structural response. The lower anchoring frequency (i.e., longest period) shall be determined using effective section properties per Section 11.7.3.4 and by reducing the resulting lowest natural frequency by 10%. The higher anchoring frequency (i.e., shortest period) shall be chosen such that a minimum of 90% mass participation in the longitudinal, transverse directions are mobilized.

Effective sectional properties or moment-curvature analysis shall be used per Section 11.7.3.4. Material properties shall be used per Section 11.7.3.6.

If applicable, appropriate linear stiffness may be assumed for abutments and expansion joints per CSDC. Analyses shall be performed for compression models (abutments engaged, gaps between frames closed) and for tension models (abutments inactive, gaps between frames open), to obtain a maximum response envelope. If analysis results show that soil capacities are exceeded at an abutment, iterations shall be performed with decreasing soil spring constants at the abutment per CSDC recommendations. Otherwise, nonlinear representations of abutment and expansion joint characteristics shall be used.

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

For NCL/MCE design, dead and superimposed dead loads shall be applied as an initial condition.

For OPL/OBE design, in addition to dead and superimposed dead loads, train mass and live load shall be considered per Section 11.7.3.12.

The time histories shall reflect the characteristics (fault distance, site class, moment magnitude, spectral shape, rupture directivity, rupture mechanisms, and other factors) of the controlling design earthquake ground motions, as given in the Geotechnical Reports required by the *Geotechnical* chapter.

The motions shall consist of 2 horizontal ground motion time histories, selected, scaled, and spectrally matched. The 2 horizontal components of the design ground motions shall be representative of the fault-normal and fault-parallel motions at the site, as appropriate, and transformed considering the orientation of the motion relative to the local or global coordinate systems of the structural model.

Consideration of vertical earthquake motions shall be considered per Section 11.7.3.3.

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When NLTHA is used, seven (7) sets of ground motions shall be used, and the average maximum values of each response parameter (e.g., force or strain in a member, displacement or rotation at a particular location) shall be used for design.

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

11.7.4 Seismic Capacities of Structural Components

11.7.4.1 Force Capacities (ΦF_N) for OPL/OBE

For OPL/OBE design, LRFD force capacities, ΦF_N, for all structural components shall be found in accordance with AASHTO LRFD with Caltrans Amendments. Nominal material properties shall be used when determining OBE capacities.

11.7.4.2 Displacement Capacity (Δ_C) for NCL/MCE

For NCL/MCE design employing flexural plastic hinging using ESA, RSA, and ELTHA demands, the displacement capacity, Δ_C, shall be determined by nonlinear static pushover analysis, as described in Section 11.7.4.3. The displacement capacity shall be defined as the controlling structure displacement that occurs when any element of targeted inelastic response reaches its allowable capacity in the pushover analysis. The allowable capacity is reached when the concrete or steel strain of any element of targeted inelastic response meets the allowable strains specified in Sections 11.7.4.5 to 11.7.4.8.

If moment curvature representation of plastic hinging is used for NLTHA, then the curvature demands shall be converted to concrete or steel strains, and verified versus allowable strains specified in Sections 11.7.4.5 to 11.7.4.8.

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 capacity protected structural members and connections shall satisfy requirements in Section 11.7.5.5.

11.7.4.3 Nonlinear Static Pushover Analysis

For NCL/MCE design employing flexural plastic hinging, the following procedure shall be followed to determine the displacement capacity, Δ_C, using nonlinear static pushover analysis.

Dead and superimposed dead load shall be applied as an initial step.

Incremental lateral displacements shall be applied to the system. A plastic hinge shall be assumed to form in an element when the internal moment reaches the idealized yield limit in accordance with Section 11.7.3.7. The sequence of plastic hinging through the frame system

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shall be tracked until an ultimate failure mode is reached. The system capacity shall then be determined in accordance with CSDC.

11.7.4.4 Plastic Hinge Rotational Capacity

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

The rotational capacity of any plastic hinge is defined as the product of the plastic hinge length, as defined by CSDC, and the curvature (from M-ϕ analysis) when the element of targeted inelastic response first reaches the allowable strains in Sections 11.7.4.5 to 11.7.4.8.

11.7.4.5 Strain Limits for Ductile Reinforced Concrete Members

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

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

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

For NCL/MCE design, the following allowable confined concrete compressive strain limits (ε_{cu}^{a}) shall apply for ductile reinforced concrete members:

MCE: $\varepsilon_{cu}^{a} \leq \varepsilon_{cu}$

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

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

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

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

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

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

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

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

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

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11.7.4.7 Strain Limits for Unconfined Concrete

Unconfined compressive strain limits shall be applied to concrete members without sufficient lateral reinforcement to be considered confined. If the lateral reinforcement does not meet the requirements of CBDM for confinement, the section shall be considered unconfined.

For NCL/MCE design, the following allowable concrete unconfined compressive strain limit (ε_{cu}^{a}) applies:

$$\varepsilon_{cu}^{a} = 0.005, \text{ for above and below ground concrete}$$

For NCL/MCE design, there are no allowable strain requirements for unconfined cover concrete.

11.7.4.8 Strain Limits for Structural Steel Elements

For NCL/MCE design, the following structural steel allowable tensile strain limits (ε_{su}^{a}) apply:

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

where: ε_{su} = ultimate tensile strain

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

11.7.4.9 Foundation Rocking Capacity

For NCL/MCE and OPL/OBE design, where foundation rocking is allowed per Section 11.7.2.1, the foundation rocking capacity shall be determined as per Section 11.7.3.16.

11.7.4.10 Material Properties for Capacities

For NCL/MCE design, expected material properties shall be used in calculating capacities, except shear. For seismic shear capacities, nominal material properties shall be used. Expected material properties shall conform to CSDC for concrete members and AASHTO LRFD with Caltrans Amendments for structural steel members.

For OPL/OBE design, nominal material properties shall be used in calculating capacities.

11.7.4.11 Shear Capacity

For NCL/MCE design, the shear capacity of ductile components shall conform to CSDC for concrete members and AASHTO LRFD with Caltrans Amendments for structural steel members.

11.7.4.12 Joint Internal Forces

Continuous force transfer through the column/superstructure and column/footing joints shall conform to CSDC. These joint forces require that the joint have sufficient over-strength to

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ensure essentially elastic behavior in the joint regions based on the capacity of the adjacent members.

11.7.5 Seismic Performance Evaluation

11.7.5.1 Definition of Essentially Elastic

For both NCL/MCE over-strength and OPL/OBE general design, "essentially elastic" is defined as when the LRFD force capacities (ΦF_N) exceed the over-strength or factored demands.

11.7.5.2 Foundation Rocking

For NCL/MCE and OPL/OBE design, where foundation rocking is allowed per Section 11.7.2.1, evaluation shall be per Section 11.7.3.16.

11.7.5.3 Force Based Design for OPL/OBE

For OPL/OBE design, the maximum force based Demand/Capacity (D/C) Ratio shall be:

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

Where:

F_U = the force demand, as defined in Section 11.7.3.1.

ΦF_N = the LRFD force capacity, as defined in Section 11.7.4.1.

in order to satisfy the OPL performance objectives specified in Section 11.5.1.

11.7.5.4 Displacement Based Design for NCL/MCE

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

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

Where:

Δ_D = the displacement demand, as defined in Section 11.7.3.2.

Δ_C = the displacement capacity, based on strain limits, as defined in Section 11.7.4.2.

in order to satisfy the NCL performance objectives specified in Section 11.5.1.

11.7.5.5 Capacity Protected Element Design

In order to limit the inelastic deformations to the prescribed element of targeted inelastic response, the plastic moments and shears of the element of targeted inelastic response shall be used in the demand/capacity analysis of any adjacent capacity-protected elements of the structure.

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Component 120% over-strength factors for the evaluation of capacity-protected elements shall be applied as specified in CSDC for concrete members and AASHTO LRFD with Caltrans Amendments for structural steel members.

11.7.5.6 Soil Improvement

The Geotechnical Reports required by the *Geotechnical* chapter shall provide information and design parameters regarding soil improvement.

11.7.5.7 Design of Shallow Foundations

The Geotechnical Reports required by the *Geotechnical* chapter shall provide information and design parameters regarding the design of shallow foundations.

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

11.7.5.8 Design of Caissons, Pile, and Drilled Shaft Foundations

The Geotechnical Reports required by the *Geotechnical* chapter shall provide information and design parameters regarding the design of caissons, piles, and drilled shaft foundations.

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

For exceptionally soft soil sites, if below ground plastic hinging is unavoidable in caissons, piles or drilled shafts, then a design variance shall be submitted per the *General* chapter. All seismic related design variances shall be identified and justified in the SDAP, as required in Section 11.3.

The design of caissons, piles, and drill shaft foundations shall be in accordance with the CBDM.

11.7.5.9 Expansion Joint and Hinge/Seat Capacity

Structural expansion joint design shall provide free movement for service loads, and OPL/OBE design. However, hinge restrainers may be designed to limit relative longitudinal expansion joint displacements for OPL/OBE design.

Relative expansion joint displacements (longitudinal, transverse, and vertical) for load cases including OBE shall comply with limits contained within the Track-Structure Interaction section of the *Structures* chapter.

Under NCL/MCE design, local damage to structural expansion joints is allowed, but the damage shall not induce changes to important structural behavior.

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

Sacrificial components, such as seat type abutment shear keys, are not subject to capacity protection under NCL/MCE design. However, for Primary Type 1 structures, seat type abutment shear keys shall be designed as essentially elastic for OPL/OBE design, and have sufficient stiffness to criteria in the Track-Structure Interaction section of the *Structures* chapter.

For NCL/MCE design, when excessive longitudinal or transverse seismic displacement must be prevented, non-sacrificial shear keys shall be provided and designed as capacity-protected elements.

11.7.5.10 Columns

Columns shall satisfy the detailing requirements for ductile structural elements as specified in CSDC. Ductile detailing requirements apply to all columns, even those designed to be essentially elastic due to foundation rocking or energy dissipation, seismic response modification, or base isolation systems.

The use of lightweight concrete is not allowed in columns.

The column reinforcement ration shall be kept below 4% to reduce congestion due to added joint reinforcement. Column reinforcement shall not be adjusted for drain pipes or other utilities in potential plastic hinge zones. For column flare design and detailing, CSDC shall apply.

11.7.5.11 Superstructures and Bent Caps

Superstructures and bent caps shall be designed as capacity protected elements and shall conform to the requirements of CSDC.

11.7.5.12 Structural Joints

Structural joints (e.g.: column / superstructure, column / bent cap, or column / footing) shall conform to the requirements of CSDC.

11.8 Passenger Stations and Building Structures

All Primary Type 1 passenger stations and building structures shall be subject to the seismic criteria for Bridges, Aerial Structures, and Grade Separations per Section 11.7.

All Primary Type 2 passenger stations and building structures shall be subject to both NCL/MCE and OPL/OBE seismic criteria herein.

All Secondary passenger stations and building structures owned by the Authority shall be subject to the NCL/MCE seismic criteria herein.

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All Secondary passenger stations and building structures owned by Third Parties shall be subject to the applicable Third Party seismic criteria.

11.8.1 Design Codes

For NCL/MCE design of Primary Type 2 structures, ASCE 41 shall apply. Although ASCE 41 is a document originally issued for seismic rehabilitation of existing structures, it is applicable in absence of a similar performance based code for the seismic design of new building structures. Certain criteria herein might exceed those of ASCE 41. If items are not specifically addressed in this or other chapters, ASCE 41 shall be used.

For OPL/OBE design of Primary Type 2 structures, current CBC force based design methods shall apply. Note that the OPL/OBE load combination, as given in the *Structures* chapter, is a strength load combination. No seismic response modification factors shall apply to the OBE demands.

For NCL/MCE design of Secondary structures owned by the Authority, current CBC seismic design shall apply, including applicable use of seismic response modification factors.

Table 11-6 summarizes the applicable seismic design code for each General Classification.

Table 11-5: Applicable Passenger Station and Building Structure Design Codes

Performance/ Design Earthquake	General Classification		
	Primary Type 1	**Primary Type 2**	**Secondary**
NCL/MCE	CBDM [1]	ASCE 41	CBC (Seismic Design)
OPL/OBE	AASHTO/Caltrans [1]	CBC (Strength Design)	--
TSI/OBE	*Structures* chapter	--	--

Notes:
[1] as amended by Section 11.7

11.8.2 Seismic Design Approach

The seismic design approach differs depending upon the design earthquake.

11.8.2.1 NCL/MCE Design Approach

For Primary Type 2 structures, for NCL/MCE design the approach shall be:

- A "weak beam - strong column" approach, 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. The formation of a plastic hinge shall take place in the beam element at a distance not less than twice the beam depth away from the face of the joint by adequate detailing.

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- The structure 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 (non-ductile). The station or building shall be provided with at least 1 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.
- Detailing and proportioning requirements for full-ductility structures shall be satisfied. No brittle failure shall be allowed.

If energy dissipation, seismic response modification, or base isolation systems are used, then a design variance shall be submitted per the *General* chapter. In addition, the use of such systems shall be identified in the SDAP, as discussed in Section 11.3.

11.8.2.2 OPL/OBE Design Approach

For Primary Type 2 structures, for OPL/OBE design, the approach shall be:

- The station or building shall respond as essentially elastic.

11.8.3 Seismic Demands on Structural Components

11.8.3.1 Analysis Techniques – General

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

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 LDP or NSP can be satisfied, all structures shall be analyzed using NDP.

11.8.3.2 Linear Dynamic Procedure

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

When response spectrum analysis is used, modal combination shall be performed using the CQC approach, while spatial combination shall be performed using the square root of the sum of the squares (SRSS) technique.

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When the time history method is used, input ground motions shall be applied to a three-dimensional model of the structure. Where the relative orientation of the ground motions cannot be determined, the ground motion shall be applied in the direction that results in the maximum structural demands.

When the time history method is used, seven (7) sets of ground motions shall be used, and the average maximum values of each response parameter (e.g., force or strain in a member, displacement or rotation at a particular location) shall be used for design.

The ground motion sets shall meet the requirements of Section 11.5.2.

For buildings that have 1 or more of the following conditions, LDP shall not be used:

- In-Plane Discontinuity Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.1 of ASCE 41
- Out-of-Plane Discontinuity Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.2 of ASCE 41
- Weak Story Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.3 of ASCE 41
- Torsional Strength Irregularity, unless it is shown that the building remains linear elastic as per requirements of Section 2.4.1.1.4 of ASCE 41
- Building structures subject to potential foundation sliding, uplift and/or separation from supporting soil (near field soil nonlinearity)
- Building structures which include components with nonlinear behavior such as, but not limited to, buckling, expansion joint closure
- When energy dissipation, seismic response modification, or base isolation systems are used
- When the building site is in close proximity (within 12.5 miles [20 km]) to hazardous faults, or for ground motions with near-field pulse-type characteristics, a time history analysis shall be used.

11.8.3.3 Nonlinear Static Procedure

For NSP, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be developed and subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. Mathematical modeling and analysis procedures shall comply with the requirements of ASCE 41. The target displacement shall be calculated by the procedure described in ASCE 41. At least 2 types of lateral load pattern shall be considered as described in ASCE 41. The pushover analysis shall be performed in 2 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

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opposite directions. In these cases the NSP shall include pushover analysis in 2 opposite directions (for a total of 4 analyses for 2 principal directions). When the response of the structure is not primarily in 1 of the principal directions, the pushover analysis shall consider non-orthogonal directions to develop a spatial envelope of capacity.

For buildings that have 1 or more of the following conditions, NSP shall not be used:

- For buildings that the effective modal mass participation factor in any 1 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 1 of the structure's main response is torsion
- When energy dissipation, seismic response modification, or base isolation systems are used

11.8.3.4 Nonlinear Dynamic Procedure

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

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

When NDP is used, seven (7) sets of ground motions shall be used, and the average maximum values of each response parameter (e.g., force or strain in a member, displacement or rotation at a particular location) shall be used for design.

The ground motion sets shall meet the requirements of Section 11.5.2.

As a minimum, NDP 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

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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 properties, do in fact remain elastic and satisfy strength requirements.
- For the deformation-controlled action members the deformations shall be compared with the strain limits for each performance level as specified in this document.
- For force-controlled action members the force demand shall be resisted by capacities calculated as per ASCE 41, ACI, and AISC.

11.8.3.5 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.

11.8.3.6 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.

11.8.3.7 Building Separation

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

11.8.3.8 Material Properties for Demands

Material properties for demands shall be per Section 11.7.3.6

11.8.3.9 Effective Sectional Properties

Effective sectional properties shall be per Section 11.7.3.4.

11.8.3.10 Foundation Stiffness

Foundation stiffness, including the effects of soil structure interaction, shall be considered for seismic analyses per the *Geotechnical* chapter.

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

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applied to the ground nodes of the soil elements. The Geotechnical Reports required by the *Geotechnical* chapter 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.

11.8.3.11 Boundary Conditions

In cases where the building is adjacent to or 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 the adjacent structures.

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

11.8.3.12 Multidirectional Seismic Effects

The ground motions shall be applied concurrently in 2 horizontal directions and vertical direction as per ASCE 41. In the demand and capacity assessment of deformation-controlled actions, simultaneous orthogonality effects shall be considered.

11.8.3.13 Load and Load Combinations

Seismic loads and load combinations shall comply with the requirements of the *Structures* chapter. 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.

11.8.3.14 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.

11.8.3.15 P-Δ Effects

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

11.8.3.16 Overturning

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

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11.8.3.17 Seismic Capacities of Structural Components

The component capacities shall be computed based on methods given in Chapters 5 and 6 of ASCE 41 for steel and concrete structures, respectively. However, strain limits described in Sections 11.7.4.5 and 11.7.4.8 of this document shall be used.

11.8.3.18 Material Properties for Capacities

Material properties for capacities shall be per Section 11.7.4.10.

11.8.3.19 Capacity of Members with Force-Controlled Action

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

11.8.3.20 Capacity Protected Element Design

For NCL/MCE design, pre-determined structural components may undergo flexural plastic hinging, and 120% over strength factors shall be applied to capacity protected members to protect against brittle failure mechanisms. All other structural components not pre-determined for flexural plastic hinging shall be designed to remain elastic under the MCE.

For OPL/OBE design, the structure shall respond as essentially elastic.

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