Section 5
ANALYSIS REQUIREMENTS

5.1 DEFINITION OF PROCEDURES

5.1.1 General

When seismic analysis is required for Seismic Design and Analysis Procedure (SDAP) C, D, and E, the bridge shall be analyzed using a mathematical model that considers the geometry, boundary conditions and material behavior of the structure. The Engineer should consider the force and deformation effects being quantified and the accuracy required when defining a mathematical model.

A representation of the foundation and soil that supports the bridge may be included in the mathematical model of the foundations depending on the type of foundation, the Seismic Design and Analysis Procedure (SDAP), and the Seismic Design Requirement (SDR). When the foundations and abutments are included in the mathematical model, the assumed properties shall be consistent with the expected deformations of the soil.

In the case of seismic design, gross soil movement and liquefaction shall also be considered in the analysis when applicable.

5.1.2 Selection of Analysis Procedure

For seismic design the choice of the mathematical model and analysis procedure shall be based on the requirements of Article 3.7.

Table 3.7-2 identifies the Seismic Design and Analysis Procedure. When required, the Seismic Design and Analysis Procedures use the following seismic demand analysis and/or seismic displacement capacity verification procedures in order of increasingly higher-level of ability to represent structural behavior. A higher level analysis may be used in place of a lower-level analysis.

- *Capacity Spectrum Analysis* - Seismic response of a very regular structure is modeled as a single degree-of-freedom system, and the demand analysis and capacity evaluation are combined in a single procedure. The capacity spectrum analysis may be used for seismically isolated bridges.

- *Elastic Response Spectrum Analysis* - Seismic demands are determined by a response spectrum analysis using the spectrum defined in Article 3.4. For bridges with a regular configuration, the uniform load method may be used, otherwise a multi-mode dynamic analysis is required.

- *Nonlinear Static Displacement Capacity Verification ("Pushover" Analysis)* - The displacement capacity of individual piers or bents is determined by a lateral load-displacement analysis accounting for the nonlinear behavior of the inelastic components.

- *Nonlinear Dynamic Analysis* – Nonlinear dynamic analysis using earthquake ground motion records to evaluate the displacement and force demands accounting for the inelastic behavior of the components.

A nonlinear dynamic analysis is required for structures with seismic isolation systems with (1) an effective vibration period greater than 3 seconds, or (2) effective damping greater than 30 percent.

The displacements from any demand analysis must satisfy the requirements in Articles 7.3 or 8.3.

5.2 SEISMIC LATERAL LOAD DISTRIBUTION

5.2.1 Applicability

These provisions shall apply to decks, girders, diaphragms (cross-frames), lateral bracing, and connections between the superstructure and the substructure, which are part of the earthquake

resisting system in structures with Seismic Design Requirements (SDR) 3 and 4. These provisions do not apply in Seismic Design Requirements 1 and 2.

5.2.2 Design Criteria

The Engineer shall demonstrate that a clear, straightforward load path from the superstructure to the substructure exists and that all components and connections are capable of resisting the imposed load and displacement effects consistent with the chosen load path.

If the overstrength forces are chosen for use in the design of the superstructure, then the elastic force distribution in the superstructure obtained from an elastic response spectrum analysis is not appropriate for use in the superstructure design. Unless a more refined analysis is made when using the overstrength forces in the superstructure design, the inertial forces expected to act on the superstructure may be assumed to vary linearly along the superstructure, and they shall produce both translational and rotational equilibrium when combined with the plastic mechanism forces from the substructure.

The flow of forces in the assumed load path must be accommodated through all affected components and details including, but not limited to, flanges and webs of main beams or girders, cross-frames, connections, slab-to-girder interfaces, and all components of the bearing assembly from top flange interface through the confinement of anchor bolts or similar devices in the substructure.

The analysis and design of end diaphragms and cross-frames shall consider horizontal supports at an appropriate number of bearings. Slenderness and connection requirements of bracing members that are part of the lateral force resisting system shall comply with applicable provisions specified for main member design.

Members of diaphragms and cross-frames identified by the Designer as part of the load path carrying seismic forces from the superstructure to the bearings shall be designed and detailed to remain elastic, based on the applicable gross area criteria, under all design earthquakes, regardless of the type of bearings used. The applicable provisions for the design of main members shall apply.

However, if elements of the earthquake resisting system are explicitly intended and designed to respond inelastically, then the previous paragraph does not apply to such elements. All other elements of the earthquake resisting system shall either be capacity-protected or designed for the elastic forces.

If elements of the earthquake resisting system are designed to fuse (i.e. breakaway) in the larger earthquake, then the redistribution of force that occurs with such alteration of the seismic load path shall be accounted for in the analysis.

All load-resisting elements shall have sufficient deformation capacity at the displacement of the center of mass of structure as determined from the seismic analysis.

5.2.3 Load Distribution

A viable load path shall be established to transmit seismic loads to the substructure based on the stiffness characteristics of the deck, girders, diaphragms – end, intermediate and pier – (often referred to as cross-frames in steel bridges), lateral bracing, and connections between the superstructure and substructure. Unless a more refined analysis is made, an approximate load path shall be assumed as noted below.

In bridges with:

- A concrete deck that can provide horizontal diaphragm action, or
- A horizontal bracing system in the plane of the deck,

the lateral loads applied to the deck shall be assumed to be transmitted directly to the bearings through end diaphragms and/or pier diaphragms. The development and analysis of the load path through the deck or through the lateral bracing, if present, shall utilize assumed structural actions analogous to those used for the analysis of wind loading.

In bridges that have:

- Decks that cannot provide horizontal diaphragm action and
- No lateral bracing in the plane of the deck,

the lateral loads applied to the deck shall be distributed through the intermediate diaphragms to the bottom lateral bracing or the bottom flange, and then to the bearings, and through the end diaphragms and pier diaphragms in proportion to their relative rigidity and the respective tributary mass of the deck.

If a lateral bracing system is not present, and the bottom flange is not adequate to carry the imposed force effects, the first procedure shall be used, and the deck shall be designed and detailed to provide the necessary horizontal diaphragm action.

5.3 MODELING REQUIREMENTS

5.3.1 General

For the dynamic analysis of structures subjected to earthquakes, the geometric configuration, strength, stiffness, mass, and energy dissipation mechanisms of the structural components and footings shall be included in the mathematical model.

Bridges with multiple frames may be analyzed using models of a partial number of frames. Each model shall represent the geometry, mass, stiffness, and boundary conditions for the frames included in the model.

The seismic analysis shall consider at least the two horizontal ground motion components.

The combination of loads from different horizontal and vertical components is given in Article 3.6.

The effect of the vertical component ground motion on bridges within 10 km of an active fault shall be included according to the requirements in Article 3.4.5.

5.3.2 Distribution of Mass

The modeling of mass shall be made with consideration of the degree of discretization in the model and the anticipated motion due to seismic excitation.

The number of degrees-of-freedom shall be selected to represent the total mass and mass distribution of the structure.

5.3.3 Stiffness and Strength

5.3.3.1 General

The mathematical model shall represent the stiffness of individual structural elements considering the materials, section dimensions, and force transfer between elements. For ductile earthquake resisting elements the stiffness shall be representative of the stiffness near the yield deformation (e.g., cracked section properties shall be used for reinforced concrete elements). For capacity protected elements, including the superstructure, the elastic stiffness shall be represented in the mathematical model.

For Displacement Capacity Verification (“pushover” or nonlinear static analysis), the mathematical model shall include the strength based on nominal material properties. For nonlinear dynamic analysis, the models shall represent the stiffness, strength, and hysteretic behavior of the inelastic seismic resisting elements under cyclic loads.

5.3.3.2 Substructure

The flexural stiffness of columns and pier walls shall consider the effect of axial load. For reinforced concrete columns and pier walls, the stiffness shall represent the effects of cracking. When required by Article 4.10.2, the secant stiffness of columns responding inelastically shall be used.

For Displacement Capacity Verification (“pushover” or non-linear static analysis), the strength of structural steel components in the model shall be based on the nominal plastic capacity. The flexural strength of reinforced and prestressed elements shall be based on nominal material properties of the steel and concrete.

The stiffness of capacity protected elements shall be based on elastic properties, including the effects of concrete cracking.

5.3.3.3 Superstructure

The stiffness of the superstructure shall be consistent with the load path identified in accordance with Article 5.2.3, including composite behavior between girders and decks and effective width of the superstructure that are monolithic with piers.

5.3.4 Foundations

Foundations may be modeled using the Foundation Modeling Method (FMM) defined in Table 5.3.4-1. Articles 8.4.2 and 8.4.3 provide the requirements for estimating foundation springs and the depth to fixity.

The required foundation modeling method depends on the Seismic Design Requirement (SDR) and the Seismic Design and Analysis Procedure (SDAP).

For SDR 3, Foundation Modeling Method I (FMM I) is required for any SDAP.

For SDR 4, Foundation Modeling Method I may be used for SDAP C. SDAP D and E require the use of Foundation Modeling Method II (FMM II).

For SDAP E, FMM II is required in the Displacement Capacity Verification ("pushover") analysis if it is used in the multi-mode dynamic analysis for displacement demand. The foundation models in the multi-mode dynamic analysis and Displacement Capacity Verification shall be consistent and representative of the footing behavior.

Table 5.3.4-1 Definition of Foundation Modeling Method

Foundation Type	FMM I	FMM II
Spread Footing	Rigid	Rigid for Soil Types A and B. For other soil types, foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Footing with Pile Cap	Rigid	Foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Bent/Drilled Shaft	Estimated depth to fixity	Estimated depth to fixity or soil-springs based on P-y curves.

For sites identified as susceptible to liquefaction or lateral spread, the model of the foundations and structures shall consider the nonliquefied and liquefied conditions using the procedures specified in Articles 7.6 and 8.6.

5.3.5 Abutments

The model of the abutment shall reflect the expected behavior of the abutment under seismic loads in each orthogonal direction. Resistance of structural components shall be represented by cracked section properties for multi-mode response spectrum analysis. The resistance from passive pressure shall be represented by a value for the secant stiffness consistent with the maximum displacement – Articles 7.5 and 8.5. For the Displacement Capacity Verification, the strength of each component in the abutment, including soil, shall be included.

5.3.6 Seismic Isolator Units

Seismic isolator units shall be modeled by an effective stiffness based on the properties of the isolator unit.

To simplify the nonlinear behavior of the isolator unit, a bi-linear simplification may be used. The analysis shall be repeated using upper-bound properties in one analysis and lower-bound properties in another if required by Article 15.4. The purpose of the upper- and lower-bound analyses is to determine the maximum forces in the substructure and maximum displacement of the isolation system.

The upper- and lower-bound analyses are not required if the displacements, using equation (5.4.1.1-1), do not vary from the design values by more than 15 percent when the maximum and minimum values of the isolator unit properties are used (Article 15.4). For these simplified calculations, damping ratios greater than 30 percent may be used to establish the 15 percent limit.

5.3.7 Bearing and Joints

Two models shall represent expansion bearings and intermediate joints. The compression model assumes the superstructure at the bearing or joint is closed and can transfer longitudinal forces. The tension model assumes the bearing or joint is open and cannot transfer longitudinal forces. The

stiffness of restraining devices, if any, shall be included in the tension model.

A compression model need not be considered for expansion bearings if it can be demonstrated by calculation that longitudinal forces cannot be transferred through the superstructures at the bearing location.

5.3.8 Damping

Energy dissipation in the structure, including, footings and abutments, may be represented by viscous damping. The selection of the viscous damping ratio depends on the type of dynamic analysis and the configuration of the bridge.

For elastic response spectrum analysis, the viscous damping ratio inherent in the specified ground spectra is 5% damping and this is specified for all structural systems except those with isolation systems. For the Capacity Spectrum method, damping is inherent in the B-Factor.

5.4 ANALYSIS PROCEDURES

The regularity requirements that permit use of the Capacity Spectrum Analysis Method are given in Article 4.4.2. The regularity requirements for using the Uniform Load Method and Multi-mode Methods of Analyses are given in Article 5.4.2.1.

5.4.1 Capacity Spectrum Analysis

The lateral strength of each pier in the longitudinal and transverse directions shall be at least C_c times the tributary weight for the pier.

The lesser of the following equations shall be used to assess C_c for the 50% PE in 75 year and 3% PE in 75-year/1.5 mean-deterministic earthquake loadings:

$$C_c\Delta = \left(\frac{F_v S_1}{2\pi B_L}\right)^2 g \qquad (5.4.1\text{-}1)$$

$$C_c = \frac{F_a S_s}{B_s} \qquad (5.4.1\text{-}2)$$

where B_s and B_L are response reduction factors for short and long period structures, respectively, and are defined in Table 5.4.1-1. The response spectrum values and soil factors, $F_V S_1$ and $F_a S_s$, are defined in Article 3.4. In Equation 5.4.1-1, Δ is the displacement of the pier.

When equation 5.4.1-1 governs for the 3% PE in 75-year/1.5 mean deterministic earthquake, the displacement of the superstructure, Δ, shall satisfy the requirements of Articles 7.3.4 or 8.3.4. When equation 5.4.1-1 governs for the 50% PE in 75-year earthquake, Δ shall be taken as 1.3 times the yield displacement of the pier.

5.4.1.1 Seismic Isolation Systems

The capacity spectrum analysis procedure may be used for structures with seismic isolation systems that meet the regularity requirements for the Uniform Load Method of Article 5.4.2.1 and the effective vibration period is 3 seconds or less, and the effective damping is less than or equal to 30 percent of critical. Article 15.4 specifies other analysis procedures for seismically isolated structures.

The displacement, Δ, (meters) of the superstructure (including the substructure and bearing unit deformation) is given by

$$\Delta = \frac{0.25 F_v S_1 T_{eff}}{B} \text{ (meters)} \qquad (5.4.1.1\text{-}1)$$

$$\Delta = \frac{10 F_v S_1 T_{eff}}{B} \text{ (inches)} \qquad (5.4.1.1\text{-}1(b))$$

where

$$T_{eff} = 2\pi\sqrt{\frac{W}{K_{eff} g}} \qquad (5.4.1.1\text{-}2)$$

The damping coefficient, B, is based on the percentage of critical damping according to Table 5.4.1.1-1. The percentage of critical damping depends on the energy dissipation by the isolation system, which shall be determined by test of the isolation systems characteristics, as specified in Article 15.10. The damping coefficient may be determined by linear interpolation of the values in Table 5.4.1.1-1.

Table 5.4.1-1 Capacity Spectrum Response Reduction Factors for Bridges with Ductile Piers

(a) 50% in 75 Year Earthquake Loading

Performance Level	B_S	B_L
Operational	1	1
Life Safety	1	1

(b) 3% in 75 Year Earthquake Loading

Performance Level	B_S	B_L
Operational	1	1
Life Safety	2.3	1.6

Table 5.4.1.1-1 Capacity Spectrum Response Reduction Factors for Bridges with Seismic Isolation Systems

	Damping (as percentage of critical)						
	≤ 2	5	10	20	30	40	50
B	0.8	1.0	1.2	1.5	1.7	1.9	2.0

5.4.2 Elastic Response Spectrum Analysis

5.4.2.1 Selection of Analysis Method

The uniform load method may be used for structures satisfying the requirements in Table 5.4.2.1-1. For structures not satisfying the regularity requirements of Table 5.4.2.1-1, the multi-mode dynamic analysis shall be used.

Table 5.4.2.1-1 Requirements for Uniform Load Method

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	20°	20°	30°	30°	30°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span to span, excluding abutments	---	4	4	3	2

5.4.2.2 Uniform Load Method

The uniform load method shall be based on the fundamental mode of vibration in the longitudinal or transverse direction. The period of this mode of vibration shall be taken as that of an equivalent single mass-spring oscillator. The stiffness of this equivalent spring shall be calculated using the maximum displacement that occurs when an arbitrary uniform lateral load is applied to the bridge. The seismic coefficient demand, C_d, specified in Article 3.4 by the response spectra at the appropriate period shall be used to calculate the equivalent uniform seismic load from which seismic force effects are found. However, for periods less than T_s, the seismic coefficient demand shall be equal to S_{DS}.

5.4.2.3 Multi-Mode Dynamic Analysis Method

The elastic multi-mode dynamic analysis method shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. As a minimum, linear dynamic analysis using a three-dimensional model shall be used to represent the structure.

The number of modes included in the analysis shall be at least three times the number of spans in the model for regular bridges, and the total modal mass shall be at least 90%.

The elastic seismic response spectrum as specified in Article 3.4 shall be used for each mode with its inherent 5% damping. The spectrum at the fundamental vibration periods shall be scaled for damping ratios other than 5 percent for an isolated structure. For structures with seismic isolation the scaling shall apply only for periods greater than $0.8T_{eff}$ where T_{eff} is defined in Article 15.4.1. The 5 percent response spectrum shall be used for other modes.

The member forces and displacements due to a single component of ground motion may be estimated by combining the respective response quantities (moment, force, displacement, or relative displacement) from the individual modes by the Complete Quadratic Combination (CQC) method. Combination of forces from orthogonal components of ground motion are specified in Article 3.6.

5.4.3 Seismic Displacement Capacity Verification

The displacement capacity verification analysis shall be applied to individual piers or bents to determine the lateral load-displacement behavior of the pier or bent. The capacity evaluation shall be performed for individual piers or bents in the longitudinal and transverse direction separately.

The capacity evaluation shall identify the component in the pier or bent that first reaches its inelastic deformation capacity as given in Articles 7.7.9, 7.8.6, 8.7.9 or 8.8.6. The displacement at which the first component reaches its maximum permitted deformation capacity defines the maximum displacement capacity, $\Delta_{capacity}$ for the pier or bent and this shall exceed the demand given in Articles 7.3.5 or 8.3.5. The model shall represent all components providing seismic load resistance.

When required by Article 5.3.4, the model for the foundation shall include soil springs or an estimated depth to fixity.

The model for the displacement capacity verification is based on nominal capacities of the inelastic components. Stiffness and strength degradation of inelastic components and effects of loads acting through the lateral displacement shall be considered.

Maximum plastic hinge rotations for structural components are specified in Articles 7.7.9, 7.8.6, 8.7.9 or 8.8.6. The maximum deformation for foundation and abutments are limited by geometric constraints on the structure and given in Article C3.2.

The model of the foundation for the displacement capacity evaluation shall be consistent with the demand analysis, Article 5.3.4.

For the purpose of this Article, the displacement is the displacement at the center of mass for superstructure supported by the pier or bent under consideration.

5.4.4 Nonlinear Dynamic Analysis Procedure

Nonlinear dynamic analysis provides displacements and member actions (forces and deformations) as a function of time for a specified earthquake ground motion. All loads in Extreme Load Case I shall be included in the analysis.

The ground motion time histories shall satisfy the requirements of Article 3.4.4.

A minimum of three ground motions, representing the design event, shall be used in the analysis. Each ground motion shall include two horizontal components and a vertical component. The maximum action for the three ground motions shall be used for design. If more than seven ground motions are used, the design action may be the mean of the actions for the individual ground motions.