Section 5: Commentary
ANALYSIS REQUIREMENTS

C5.1 DEFINITION OF PROCEDURES

C5.1.1 General

Seismic analysis encompasses a demand analysis and a displacement capacity verification. The objective of a demand analysis is to estimate the forces and displacements induced by the seismic excitation. Depending on the design procedure, a verification of displacement capacity of piers or bents may be required. The objective of a displacement capacity verification is to determine the displacement of an individual pier or at which the deformation capacity of the inelastic earthquake resisting elements is reached. The displacement capacity must be greater than the displacement demand. The accuracy of the demand and capacity analyses depend on the assumption of the model related to the geometry, boundary conditions, material properties, and energy dissipation incorporated in the model. It is the responsibility of the Engineer to assess the reasonableness of a model in representing the behavior of the structure at the level of forces and deformations expected for the seismic excitation.

Very flexible bridges, e.g., suspension and cable-stayed bridges, shall be analyzed accounting for the nonlinear geometry. The need for modeling of foundations and abutments depends on the sensitivity of the structure to foundation flexibility and associated displacements. This in turn depends on whether the foundation is a spread footing, pile footing with pile cap, a pile bent, or drilled shaft. Article 5.3.4 defines the requirements for the foundation modeling in the seismic analysis.

When gross soil movement or liquefaction is determined to be possible, the model shall represent the change in support conditions and additional loads on the substructure associated with soil movement.

For structures whose response is sensitive to the support conditions, such as in a fixed-end arch, the model of the foundation shall account for the conditions present.

C5.1.2 Selection of Analysis Procedure

Bridges are designed to remain essentially elastic when subjected to earthquakes with a high-probability of occurrence (50% PE in 75 years). For low-probability earthquakes (3% PE in 75 years/1.5 mean deterministic) and depending on the desired performance level, bridges are designed to dissipate energy through inelastic deformation in earthquake resisting elements. Depending on the type of analysis, the demand and capacity may be expressed in terms of forces (bending moments in the plastic hinge zones or shear forces in isolation bearings) and/or displacements of the structure at the centroid of the mass.

In specifying the minimum Seismic Design and Analysis Procedure (SDAP), two principles are followed. First, as the seismic hazard increases, improved modeling and analysis for seismic demands is necessary because the behavior may be sensitive to the maximum demands. Secondly, as the complexity of the bridge increases, more sophisticated models are required for seismic demand and displacement capacity evaluation. No seismic analysis is required for regular bridges in SDAP B because minimum ductile detailing and capacity design principles provide sufficient displacement capacity for the hazard levels and performance requirements in which SDAP B is permitted. For bridges with a very regular configuration, a single degree-of-freedom model is sufficiently accurate to represent the seismic response. For these types of bridges, the capacity spectrum method in SDAP C combines the demand and capacity evaluation. The capacity spectrum method is appropriate for most structures with seismic isolation systems.

For structures that do not satisfy the requirements for a capacity spectrum analysis, an elastic response spectrum analysis, SDAP D, must be used to determine the displacement demands and the forces in the plastic hinge of structural

components. Two elastic response spectrum analyses methods are permitted: the uniform load method, or the multi-mode response spectrum method depending on the configuration of the structure.

The uniform load method is suitable for structures with regular configuration. Long bridges, or those with significant skew or horizontal curvature, have dynamic characteristics that shall be represented in a multi-mode dynamic analysis.

The model for an elastic response spectrum analysis is linear, and as such it does not represent the inelastic behavior of earthquake resisting elements under strong ground motion. However, with the proper representation of the inelastic elements and interpretation of responses, an elastic analysis provides reasonable estimates of seismic demands. The model must be based on cracked section properties for concrete components and secant stiffness coefficients for the foundations, abutments, and seismic isolation components that are consistent with the expected level of deformation of the element. The only forces that are meaningful from an elastic response spectrum analysis are the forces in the earthquake resisting substructure elements, such as the bending moment at a plastic hinge in a column. The elastic forces in the earthquake resisting elements are reduced a factor that accounts for ductility of the earthquake resisting system. The displacements at the center of mass, generally the superstructure, can be used to estimate the displacement demand of the structure including the effect of inelastic behavior in the earthquake resisting elements as discussed in Article C3.3.

For SDAP E, a displacement capacity evaluation is required. The displacement capacity evaluation involves determining the displacement at which the first component reaches its inelastic deformation capacity. All non-ductile components shall be designed using capacity design principles to avoid brittle failure. For simple piers or bents, the displacement capacity can be evaluated by hand calculations using the geometry of displaced shapes and forces and deformation at the plastic capacity. For more complicated piers or bents, particularly when foundations and abutments are included in the model, a nonlinear static (“pushover”) analysis may be used to evaluate the displacement capacity. It is recommended that the nonlinear static analysis continue beyond the displacement at which the first component reaches its inelastic deformation capacity in order to understand the behavior beyond the displacement capacity.

The displacement capacity is compared against the displacement demand determined from an elastic response spectrum analysis. The displacement capacity must exceed the demand by at least 50%. There are several reasons for this requirement. While on average the displacement of the elastic model, using a design response spectrum, should be approximately equal to the inelastic displacement, a significant difference is possible because of variability of the ground motion and its effect on inelastic behavior. Secondly, the demand analysis is performed on a three-dimensional model, whereas the displacement capacity verification is performed for individual bents or piers in the longitudinal and transverse directions separately. In Article 7.3.5 or 8.3.5, the displacement demand is multiplied by 1.5 to account for ground motion variability and the differences in the demand and capacity models and analysis methods.

A nonlinear dynamic analysis is the most general analysis method because the effect of inelastic behavior is included in the demand analysis. Depending on the mathematical model, the deformation capacity of the inelastic elements may or may not be included in dynamic analysis. A nonlinear dynamic analysis requires a suite of time histories (Article 3.4.4) of earthquake ground motion that are representative of the hazard and conditions at the site. Because of the complexity involved with nonlinear dynamic analysis, it is best used in conjunction with SDAP E.

Seismically isolated structures with very long periods or large damping ratios require a nonlinear dynamic analysis because the analysis procedures using an effective stiffness and damping may not properly represent the effect of isolation units on the response of the structure. The model for nonlinear analysis shall represent the hysteretic relationships for the isolator units.

C5.2 SEISMIC LATERAL LOAD DISTRIBUTION

C5.2.2 Design Criteria

If the forces from the substructure corresponding to the overstrength condition are used to design the superstructure, it shall be recognized that the distribution of these forces may not be the same as that of the elastic demand analysis forces. The Engineer may calculate a more refined distribution of the inertial forces present when a full inelastic mechanism has developed in the ERE's. However, in lieu of such a calculation, the simpler linear distribution may be used, so long as the applied forces are in equilibrium with the plastic substructure forces. The vertical spatial relationship between location of the substructure plastic resistance and the location of the superstructure inertia force application shall also be considered in this analysis.

Diaphragms, cross-frames, lateral bracing, bearings, and substructure elements are part of a earthquake resisting system in which the lateral loads and performance of each element are affected by the strength and stiffness characteristics of the other elements. Past earthquakes have shown that when one of these elements responded in a ductile manner or allowed some movement, damage was limited. In the strategy taken herein, it is assumed that ductile plastic hinging in substructure or seismic isolator units are the primary source of energy dissipation.

Even if a component does not participate in the load path for seismic forces it must deform under the seismic loads. Such components must be checked that they have deformation capacity sufficient to maintain their load resistance under seismic induced deformations.

C5.2.3 Load Distribution

A continuous path is necessary for the transmission of the superstructure inertia forces to the substructure. Concrete decks have significant rigidity in their horizontal plane, and in short to medium slab-on-girder spans, their response approaches rigid body motion. Therefore, the lateral loading of the intermediate diaphragms is minimal, consisting primarily of local tributary inertia forces from the girders, themselves.

Bearings do not usually resist load simultaneously, and damage to only some of the bearings at one end of a span is not uncommon. When this occurs, high load concentrations can result at the location of the other bearings, and this effect shall be taken into account in the design of the end and pier diaphragms. Also, a significant change in the load distribution among end and pier diaphragm members may occur.

C5.3 MODELING REQUIREMENTS

C5.3.1 General

Depending on the seismic analysis method different types of approximations may be used for modeling the strength, stiffness, and energy dissipation mechanisms. One-dimensional beam-column elements are sufficient for dynamic analysis of structures due to earthquake ground motion (referred to as "spine" models or "stick" models). For seismic analysis, grid or finite element analysis are generally not necessary. They greatly increase the size of the model and complicate the understanding of the force and deformation distribution through the substructure because of the large number of vibration modes.

The geometry of skew, horizontal curvature, and joint size shall be included in the model. However, two-dimensional models are adequate for bridges with skew less than 30 degrees and a subtended angle of horizontal curvature less than 20 degrees. When skew is included in a three-dimensional model, the geometry and boundary conditions at the abutments and bearing shall be represented in order to determine the forces and displacements at these locations. Short columns or piers may be modeled with a single element, but tall columns may have two or more elements, particularly if they have significant mass, in the case of concrete, or are modeled as framed substructures.

For bridges with multiple frames, separated by expansion bearings or hinges, it is unnecessary to model and analyze the entire bridge for seismic loads. Each frame shall have sufficient strength to resist inertia loads from the mass of the frame. However, when adjacent frames have large

differences in vibration period, the frame with the longer period may increase the seismic load on the frame with the shorter period by impact across the bearing or hinge or by transverse forces through shear keys. To account for these effects, the number of frames included in a model depends on the ratio of vibration period of the frames. For bridges in which the period ratio of adjacent frames is less than 0.70 (shortest period frame divided by longest period frame), it is recommended to limit a model to five frames. The first and fifth frames in the model are considered to be boundary frames, representing the interaction with the remainder of the structure. The response of the three interior frames can be used for design of those frames. For a bridge with more than five frames, several different models are then used in the design.

For bridges with period ratios of frames between 0.70 and 1.0, fewer than five frames may be used in a model.

A common practice is to define the longitudinal direction as the chord connecting the ends of the bridge, and the transverse direction orthogonal to the longitudinal direction.

Bridges within 10 km of active fault require a site specific study and inclusion of vertical ground motion in the seismic analysis. For bridges located more than 10 km from an active fault the procedures in Article 3.4.5 are used to account for the response to vertical ground motion in lieu of including the vertical component in the seismic analysis. If the vertical ground motion component is not included in the dynamic analysis, the forces from the analysis must be modified to account for the effect. For bridges with long, flexible spans, C-bents, or other large eccentricity in the load path for vertical loads, it is recommended to include vertical ground motion in the dynamic analysis.

C5.3.2 Distribution of Mass

The distributions of stiffness and mass are included in the model for dynamic analysis. The discretization of the model shall account for geometric and material variation in stiffness and mass. Most of the mass of a bridge is in the superstructure. Four to five elements per span are generally sufficient to represent the mass and stiffness distribution of the superstructure. For spine models of the superstructure, the line of elements shall be located at the mass centroid. Rigid links can be used to represent the geometric location of mass relative to the spine elements in the model.

For single column piers, C-bents, or other unusual configurations, the rotational mass moment of inertia of the superstructure about the longitudinal axis shall be included.

The inertia of live loads need not be included in the seismic analysis. However, the probability of a large live load being on the bridge during an earthquake shall be considered when designing bridges with high live-to-dead load ratios that are located in metropolitan areas where traffic congestion is likely to occur.

C5.3.3 Stiffness and Strength

C5.3.3.1 General

For elastic analysis methods, there is a significant approximation in representing the force-deformation relationship of inelastic structural elements by a single linearized stiffness. For inelastic columns or other inelastic earthquake resisting elements, the common practice is to use an elastic stiffness for steel elements and cracked stiffness for reinforced concrete elements. However, the stiffness of seismic isolator units, abutments, and soil in foundations are represented by a secant stiffness consistent with the maximum deformation. The Engineer shall consider the distribution of displacements from an elastic analysis to verify that they are consistent with the inelastic behavior of the earthquake resisting elements.

C5.3.3.2 Substructure

Seismic design procedures have been calibrated using stiffness that is representative of deformations close to the yield deformations. At these levels of deformation reinforced concrete elements will have cracked. The effects of cracking on the stiffness depend on the cross-section, longitudinal reinforcement ratio, axial load, and amount of bond slip. The cracked flexural stiffness of a reinforced concrete member can be obtained by moment-curvature analysis of the cross section, with modifications for bond-slip.

In lieu of a moment-curvature analysis, the cracked section stiffness may be estimated by:

$$EI_{eff} = \frac{M_n}{2\varepsilon_y / D'}$$

where M_n is the nominal flexural strength of the section considering axial load, ε_y is the yield strain of the reinforcement, and D' is the effective depth of the column. If the flexural strength has not been selected, the effective stiffness may be approximated by $EI_{eff} = 0.50EI_g$ for columns and pier walls (in the weak direction), where EI_g is the cross-sectional stiffness based on gross geometry and nominal material properties.

Where the load path depends on torsion of a reinforced concrete column or substructure element, the cracked torsional stiffness may be taken as one-fifth of the uncracked torsional stiffness.

The objective of the nonlinear displacement capacity verification is to determine the displacement at which the inelastic components reach their deformation capacity. The deformation capacity is the sum of elastic and plastic deformations. The plastic deformation is expressed in terms of the rotation of the plastic hinges. A nonlinear analysis using nominal strengths of the components gives larger plastic deformations than an analysis including overstrength. Hence, it is appropriate to use the nominal strength of the components when estimating the displacement capacity.

The stiffness of pier caps shall be included in the model. Pile caps and joints in reinforced concrete substructures may be assumed to be rigid. The strength of capacity protected elements need not be included in the model.

5.3.3.3 Superstructure

For a spine or stick model of the superstructure, the stiffness is represented by equivalent section properties for axial deformation, flexure about two-axes, torsion, and possibly shear deformation in two directions. The calculation of the section stiffness shall represent reasonable assumptions about the three-dimensional flow of forces in the superstructure, including composite behavior.

The effects of skew can be neglected in the model of the superstructure. However, for large skew angles, the geometry of the piers with respect to the superstructure, and connections between the two, must be included in the model.

For reinforced box girders the effective stiffness may be based on three-quarters of the gross stiffness to account for cracking. For prestressed box girders, the full gross stiffness shall be used. The torsional stiffness may be based on a rationale shear flow without reduction due to cracking.

The flexural stiffness of the superstructure about a transverse axis is reduced near piers when there is a moment transfer between the superstructure and pier because of shear lag effects. The reduced stiffness shall be represented in the model of the superstructure.

C5.3.4 Foundations

A wide range of methods for modeling foundations for seismic analysis are possible. Generally a refined model is unnecessary for seismic analysis. For many cases the assumption of a rigid foundation is adequate. Flexibility of a pile bent or shaft can be estimated using an assumed point of flexibility associated with the stiffness estimate of the pile or shaft and the soil. Spread footings and piles can be modeled with rotational and translational springs.

The requirement for including soil springs for Foundation Modeling Method II depends on the contribution of the foundation to the elastic displacement of the pier. Foundation springs for a pier are required when the foundation increases the elastic displacement of the pier by more than 20%. This comparison may be made on individual piers using estimates of the pier stiffness with hand calculations. If the contributions exceeds 15% for a majority of piers in a bridge, then it is recommended that foundation springs be included in all piers for the seismic analysis.

This approach is based on judgement that the forces and displacements from a seismic analysis with and without foundation springs that contribute less than 20% of the displacement of a pier will be comparable for design. More flexible spread and pile footings should be modeled and included in the seismic analysis.

If foundation springs are included in the multi-mode dynamic analysis, they must be included in the pushover analysis so the two models are consistent for the displacement comparison.

For most spread footings and piles with pile cap a secant stiffness for the soil springs is adequate. If the design limits for spread or pile footings are exceeded, according to the requirements in Articles 7.4 or 8.4, bi-linear soil springs are required for the pushover analysis.

For pile bents and drilled shafts, an estimated depth to fixitity is generally adequate for representing the relative flexibility of the soil and pile or shaft. Soil springs with secant stiffness may be used to provide a better representation based a P-y curves for the footing and soil. Bi-linear springs may be used in the pushover analysis if there is particular concern with depth of the plastic hinge and effective depth of fixity.

If bi-linear springs are used in a pushover analysis, a secant stiffness typical of the expected level of soil deformation is used in the multi-mode dynamic analysis for valid comparison of displacement demand and capacity.

C5.3.5 Abutments

Articles 7.5.2.2, 7.5.3.2, 8.5.2.2 and 8.5.3.2 provide requirements for the modeling of abutments in the longitudinal and transverse directions, respectively. The iterative procedure with secant stiffness coefficients defined in those articles are included in the mathematical of the bridge to represent the resistance of the abutments in an elastic analysis. The load-displacement behavior of the abutment may be used in a static nonlinear analysis when the resistance of the abutment is included in the design of the bridge.

C5.3.6 Seismic Isolator Units

The requirements for analysis of bridges with seismic isolation systems are specified in Article 15.4 and are based on the 1999 AASHTO *Guide Specifications for Seismic Isolation Design*, which provide requirements for modeling seismic isolator units, including the use of property modification factors as given in Article 15.5.

The force-deformation characteristics can be idealized as a bilinear relationship with two key variables: second slope stiffness and characteristic strength. The area under the bilinear curve is energy dissipated by hysteretic work during cyclic loading. For design, the force-deformation relationship can be represented by an effective stiffness based on the secant stiffness and a damping coefficient.

The requirements for determining the upper-bound and lower-bound properties is provided in Article 15.4.

C5.3.7 Hinges

The use of compression and tension models is expected to provide a reasonable bound on forces (compression model) and displacements (tension model).

C5.3.8 Damping

Damping may be neglected in the calculation of natural frequencies and associated nodal displacements. The effects of damping shall be considered when the dynamic response for seismic loads is considered. The specified ground motion spectra are for 5% viscous damping and this is a reasonably conservative value.

Suitable damping values may be obtained from field measurement of induced free vibration or by forced vibration tests. In lieu of measurements, the following values may be used for the equivalent viscous damping ratio of time history analysis:

- Concrete construction: 5 percent
- Welded and bolted steel construction: 2 percent
- Timber: 5 percent

For one or two-span continuous bridges with abutments designed to activate significant passive pressure in the longitudinal direction, a damping ratio of up to 10 percent may be used for longitudinal vibration modes.

Equivalent viscous damping may be considered to represent the energy dissipation due to cyclic loading of yielding members. Equivalent damping shall only be used with a secant stiffness estimate for the entire structure. For single degree-of-freedom models the equivalence can be established within a satisfactory degree of accuracy. For bridges with seismic isolation or other seismic protection components, the equivalence is established in an approximate

manner. Equivalent viscous damping shall not be used to represent inelastic energy dissipation for any other model or method of dynamic analysis.

C5.4 ANALYSIS PROCEDURES

C5.4.1 Capacity Spectrum Analysis

The capacity spectrum analysis may be used for bridges that are designed to respond to earthquake ground motion as a single degree-of-freedom system in the longitudinal and transverse direction with the columns being the primary energy dissipation mechanism. Very regular bridges that satisfy the special requirements are expected to respond as a single degree-of-freedom system and the capacity spectrum approach may be used for such cases.

Bridges that have elastomeric or sliding bearings at each pier shall be designed as an isolated structure using the provisions of Article 5.4.1.2 and Article 15. The B_S and B_L factors of Table 5.4.1-1 are for configurations where the columns are the primary dissipation elements.

The capacity spectrum analysis uses the elastic response spectrum defined in Article 3.4.1. The elastic spectrum is reduced to account for dissipation of energy in the inelastic earthquake resisting elements. The reduced elastic spectrum is evaluated at the effective vibration period, which is based on an effective stiffness equal to the design strength divided by the maximum displacement. An advantage of the capacity spectrum method is that the vibration period does not need to be calculated because it is implicit in equations 5.4.1-1 and 5.4.1-2. Equation 5.4.1-1 will govern for most bridges, and as a result the Designer has several choices in selecting the lateral strength and maximum displacement as described in Article C4.4.

For stiff bridges, the maximum displacement may give a seismic capacity coefficient C_c greater than required by Equation 5.4.1-1. In such cases the strength need not be greater than the value defined by Equation 5.4.1-2.

The basis of the capacity spectrum method is to <u>linearize</u> nonlinear structural behavior by determining a "secant" period and effective damping factor based on hysteretic response. This approach was originally proposed by Gulkan and Sozen (1974) and called the "Substitute Structure Method".

Assuming the peak response of the nonlinear structure is equal to the displacement of an equivalent (substitute) SDOF system, the effective period is given by

$$T_{eff} = 2\pi\sqrt{\frac{m}{K_{eff}}} = 2\pi\sqrt{\frac{W/g}{F_y/\Delta_{max}}} = 2\pi\sqrt{\frac{\Delta_{max}}{C_c g}} \qquad \text{(C5.4.1-1)}$$

in which m = structure mass; W = seismic structure weight; F_y and D_{max} are the idealized response force and maximum displacement shown in Figure C3.3-2 as M_y and Δ_{max};

C_c = normalized base shear capacity given by $C_c = F_y/W$;
g = gravitational acceleration.

The seismic demand coefficient ($C_d = F_{elastic}/W$ where $F_{elastic}$ = elastic design force) can be expressed in terms of the design spectrum with the appropriate damping as used for seismic isolation such that the <u>lesser</u> of the following governs

$$C_d = \frac{F_a S_s}{B_s} \qquad \text{(C5.4.1-2)}$$

$$C_d = \frac{F_v S_1}{T_{eff} B_L} \qquad \text{(C5.4.1-3)}$$

in which $F_a S_s$ and $F_v S_1$ are obtained from Article 3.4, and B_s and B_L are modification factors for the short and long period portions of the design spectra that account for hysteretic damping effects, given by

$$B_s = \left(\frac{\xi_{eff}}{0.05}\right)^{0.5} \quad \text{and} \quad B_L = \left(\frac{\xi_{eff}}{0.05}\right)^{0.3} \qquad \text{(C5.4.1-4)}$$

where for an equivalent elasto-plastic system

$$\xi_{eff} = 0.05 + \frac{2}{\pi}\eta\left(1 - \frac{1}{\mu}\right) \qquad \text{(C5.4.1-5)}$$

in which μ = displacement ductility factor; η = energy absorption efficiency factor.

Based on extensive experimental calibration, η may be taken as follows:

- seismically detailed reinforced concrete elements

$$\eta = 0.35 - 0.4$$

- poorly detailed (non-ductile) reinforced concrete

$$\eta = 0.25$$

- For timber structures

$$\eta = 0.1 - 0.15$$

- For steel structures

$$\eta = 0.70$$

Assuming the capacity is equal to the reduced demand and taking equation (C5.4.1-1) and substituting it into (C5.4.1-3) and rearranging, gives for long period structures:

$$F_v S_1 = 2\pi B_L \sqrt{\frac{C_c^* \Delta^*}{g}} \qquad \text{(C5.4.1-6)}$$

for short period structures

$$F_a S_s = C_c B_s \qquad \text{(C5.4.11-7)}$$

Note the *greater* of the above two equations governs.

In the above, $C_c^* = C_c / \alpha_2$ and $\Delta^* = \Delta / \alpha_1$ where α_1 and α_2 are transformation factors that account for converting a MDOF system into a substitute SDOF structure. These are defined as

$$\alpha_1 = \phi_{mn} \frac{\sum_{i=1}^{N} w_i \phi_{im}}{\sum_{i=1}^{N} w_i \phi_{im}^2} \qquad \text{(C5.4.1-8)}$$

$$\alpha_2 = \frac{\left[\sum_{i=1}^{N} w_i \phi_{im}\right]^2}{\sum_{i=1}^{N} w_i \sum_{i=1}^{N} w_i \phi_{im}^2} \qquad \text{(C5.4.1-9)}$$

where $\sum_{i=1}^{N} w_i = W$ = total seismic weight; w_i = tributary weight at location i; and ϕ_{mn} = mth mode shape at the nth location.

It should be noted that if the bridge structure has a simple configuration such that the deck is subjected to pure translation (that is there is no substantial deck bending due to favorable support conditions), then the structure will behave in a single-degree-of-freedom fashion, thus α_1 and α_2 are set to unity. Such a condition can be orchestrated by design, particularly when all the piers have a similar stiffness and the deck is uncoupled from the abutments through the use of low stiffness bearing supports as required for the application of this analysis method.

The maximum displacement of the superstructure for the 3% PE in 75-year/1.5 mean deterministic earthquake is limited by the plastic deformation capacity of the substructure, taken as $\Delta = \theta_p H$ with $\theta_p = 0.035$ for reinforced concrete and the P-Δ limitation in Article 7.3.4 or 8.3.4. The maximum displacement of the superstructure for the 50% PE in 75-year earthquake is limited to 1.3 times the elastic displacement of the substructure.

C5.4.1.2 Seismic Isolation Systems

The requirements of Article 15.4 is the capacity spectrum method. Using the capacity spectrum equation in the velocity-controlled region of the spectrum (5.4.1-1), the maximum displacement is

$$\Delta = \left(\frac{F_v S_1}{2\pi B}\right)^2 g \frac{1}{C_c} \qquad \text{(C5.4.1.2-1)}$$

In the capacity spectrum method, the effective period is defined by the maximum displacement and seismic coefficient:

$$T_{eff} = 2\pi \sqrt{\frac{\Delta}{C_c g}} \qquad \text{(C5.4.1.2-2)}$$

With the effective stiffness expressed as $K_{eff} = C_c W / \Delta$, the effective period is

$$T_{eff} = 2\pi \sqrt{\frac{W}{K_{eff} g}} \qquad \text{(C5.4.1.2-3)}$$

Solving (C5.4.1.2-2) for the seismic capacity coefficient and substituting into (C5.4.1.2-1) and simplifying gives

$$\Delta = \frac{g}{(2\pi)^2} \frac{F_v S_1 T_{eff}}{B} \qquad \text{(C5.4.1.2-4)}$$

In meter units the coefficient for the expression is 0.25, and in inches units the coefficient is 10. This is the same as (3a) and (3b) in the 1999 *Guide Specifications* with AS_i replaced by $F_v S_1$ for the 3% PE in 75 year earthquake loading. In

the *Guide Specifications*, the reduction factor *B* is defined for the long-period range as is *B* in this article.

Alternatively, the seismic capacity coefficient evaluated at the effective period and reduced for the effects of energy dissipation is:

$$C_c = \frac{F_v S_1}{T_{eff} B} \qquad \text{(C5.4.1.2-5)}$$

This is the same as equation (2a) in the 1999 Guide Specifications with AS_i replaced by $F_V S_1$ for the 3% in 75 year earthquake loading and the B values from the 1999 Guide Specifications are given in Table 5.4.2-1.

C5.4.2 Elastic Response Spectrum Analysis

C5.4.2.2 Uniform Load Method

The uniform load method, described in the following steps, may be used for both transverse and longitudinal earthquake motions. It is essentially an equivalent static method of analysis that uses a uniform lateral load to approximate the effect of seismic loads. The method is suitable for regular bridges that respond principally in their fundamental mode of vibration. The capacity spectrum analysis is similar to the uniform load method, in that they are both appropriate for bridges whose dynamic response can be represented by an equivalent single degree-of-freedom system. Capacity spectrum analysis may only be used for bridges in which abutments do not resist significant longitudinal or transverse seismic forces. For such bridges, the vibration mode shape is essentially a rigid body displacement of the superstructure, providing a uniform lateral load.

Whereas displacements are calculated with reasonable accuracy, the method can overestimate the transverse shears at the abutments by up to 100 percent. Consequently, the columns may have inadequate lateral strength because of the overestimate of abutment forces. A multi-mode dynamic analysis is recommended to avoid unrealistic distributions of seismic forces.

The steps in the uniform load method are:

1. Calculate the static displacements $v_s(x)$ due to an assumed uniform load p_o, as shown in Figure C5.4.2.2-1. The uniform loading p_o is applied over the length of the bridge; it has dimension of force/unit length and may be arbitrarily set equal to 1.0. The static displacement $v_s(x)$ has the dimension of length.
2. Calculate the bridge lateral stiffness, K, and total weight, W, from the following expressions:

$$K = \frac{p_0 L}{V_{s,MAX}} \qquad \text{(C5.4.2.2-1)}$$

$$W = \int_0^L w(x)dx \qquad \text{(C5.4.2.2-2)}$$

where:
L = total length of the bridge
$v_{s,MAX}$ = maximum value of $v_s(x)$
$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure.

The weight shall take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns, and footings. Other loads, such as live loads, may be included.

3. Calculate the period of the bridge, T_m, using the expression:

$$T_m = 2\pi\sqrt{\frac{W}{Kg}} \qquad \text{(C5.4.2.2-3)}$$

where:
g = acceleration of gravity

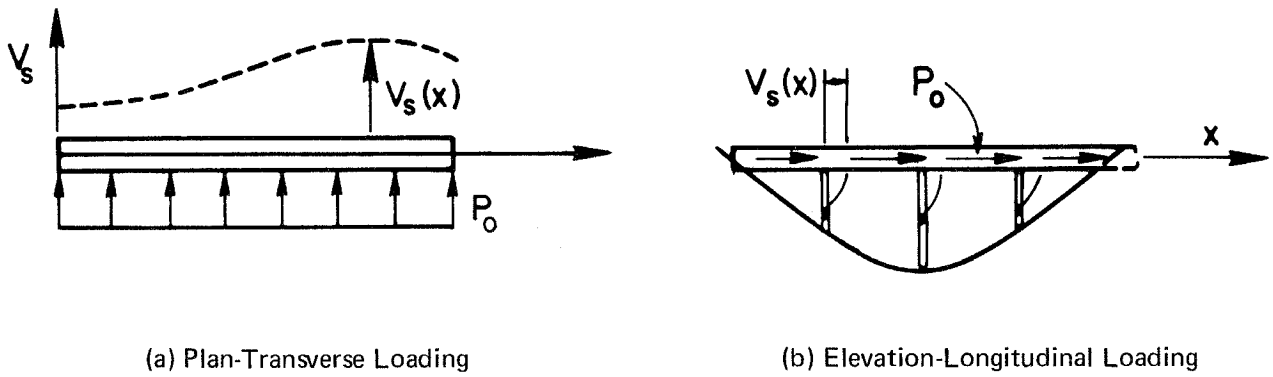

Figure C5.4.2.2-1 Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading

4. Calculate the equivalent static earthquake loading p_e from the expression:

$$p_e = \frac{C_d W}{L} \qquad \text{(C5.4.2.2-4)}$$

where:

C_d = the dimensionless elastic seismic response demand coefficient obtained from Article C3.4.1 with the coefficient taken as S_{DS} for short periods.

p_e = equivalent uniform static seismic loading per unit length of bridge applied to represent the primary mode of vibration.

5. Calculate the displacements and member forces for use in design either by applying p_e to the structure and performing a second static analysis or by scaling the results of the first step above by the ratio p_e/p_o.

C5.4.2.3 Multi-Mode Dynamic Analysis Method

Vibration modes are convenient representation of dynamic response for response spectrum analysis. Enough modes shall be included to provide sufficient participation for bending moments in columns, or other components with inelastic deformation. Dynamic analysis programs, however, usually only compute participation factors for base shear, often expressed as a percentage of total mass. For regular bridges the guideline of including 90% of the modal mass for horizontal components generally provides sufficient number of modes for accurate estimate of forces in lateral load resisting components. For irregular bridges, or large models of multiple-frame bridges, the participating mass may not indicate the accuracy for forces in specific components. It is for this reason that the models of long bridges are limited to five frames.

The response spectrum in Article 3.4.1 is based on 5 percent damping. For bridges with seismic isolation the additional damping from the seismic isolator units applies only to the isolated vibration modes. Other vibration modes have 5% damping.

A suitable modification of the 5 percent response spectrum is to divide the spectrum by:

$$\left(\frac{\beta}{5}\right)^{0.3}$$

for vibration periods greater than T_S and divide by

$$\left(\frac{\beta}{5}\right)^{0.5}$$

for vibration periods less than or equal to T_s, where β is the damping ratio in percent up to 30 percent.

Member forces and displacements obtained using the CQC combination method are generally adequate for most bridge systems (Wilson et al. 1981).

If the CQC method is not readily available, alternative methods include the square root of the sum of the squares method (SRSS), but this method is best suited for combining responses from modes with well-separated frequencies. For closely spaced modes, the absolute sum of the modal responses shall be used.

C5.4.3 Seismic Displacement Capacity Verification

The objective of the displacement capacity verification analysis is to determine the displacement at which the earthquake resisting elements achieve their inelastic deformation capacity. Damage states are defined by local deformation limits, such as plastic hinge rotation, footing settlement or uplift, or abutment displacement. Displacement may be limited by loss of capacity such as degradation of strength under large inelastic deformations or P-Δ effects.

For simple piers or bents, the maximum displacement capacity can be evaluated by hand calculations using the defined mechanism and the maximum allowable deformations of the plastic hinges. If axial force-moment interaction is significant, iteration is necessary to determine the mechanism.

For more complicated piers or foundations, displacement capacity can be evaluated using a nonlinear static analysis procedure, commonly known as a pushover analysis.

Displacement capacity verification is required for individual piers or bents. Although it is recognized that force redistribution may occur as the displacement increases, particularly for frames with piers of different stiffness and strength, the objective of the capacity verification is to determine the maximum displacement capacity of each pier. The displacement capacity is to be compared with an elastic demand analysis, which considers the effects of different stiffness and is specified in Article 7.3.5 or 8.3.5.

Nominal inelastic capacities are used for the displacement capacity verification. Although the displacement capacity verification considers a monotonically increasing displacement, the effects of cyclic loading must be considered when selecting an appropriate model and establishing a maximum inelastic deformation. This includes strength and stiffness degradation and low-cycle fatigue.

Generally, the center of mass is at the elevation of the mass centroid of the superstructure.

C5.4.4 Nonlinear Dynamic Analysis Procedure

The nonlinear dynamic analysis procedure is normally only used for the 3% in 75 year earthquake. The structure is expected to remain essentially elastic for the 50% in 75 year earthquake, hence a multi-mode response spectrum analysis is adequate.

The results of a nonlinear dynamic analysis should be compared with the a multi-mode response spectrum analysis as a check for reasonableness of the nonlinear model.