4.7.3 Inelastic Dynamic Responses

4.7.3.1 General

During a major earthquake or ship collision, energy may be dissipated by one or more of the following mechanisms:

- Elastic and inelastic deformation of the object that may collide with the structure,
- Inelastic deformation of the structure and its attachments,
- Permanent displacement of the masses of the structure and its attachments, and
- Inelastic deformation of special-purpose mechanical energy dissipators.

4.7.3.2 Plastic Hinges and Yield Lines

For the purpose of analysis, energy absorbed by inelastic deformation in a structural component may be assumed to be concentrated in plastic hinges and yield lines. The location of these sections may be established by successive approximation to obtain a lower bound solution for the energy absorbed. For these sections, moment-rotation hysteresis curves may be determined by using verified analytic material models.

4.7.4 Analysis for Earthquake Loads

4.7.4.1 General

Minimum analysis requirements for seismic effects shall be as specified in Table 4.7.4.3.1-1.

For the modal methods of analysis, specified in Articles 4.7.4.3.2 and 4.7.4.3.3, the design response spectrum specified in Figure 3.10.4.1-1 and Eqs. 3.10.4.2-1, 3.10.4.2-3, and 3.10.4.2.4 shall be used.

Bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their **operational classification** and geometry. However, the minimum requirements, as specified in Articles 4.7.4.4 and 3.10.9, shall apply.

4.7.4.2 Single-Span Bridges

Seismic analysis is not required for single-span bridges, regardless of seismic zone.

Connections between the bridge superstructure and the abutments shall be designed for the minimum force requirements as specified in Article 3.10.9.

Minimum support length requirements shall be satisfied at each abutment as specified in Article 4.7.4.4.

C4.7.4.2

A single-span bridge is comprised of a superstructure unit supported by two abutments with no intermediate piers.

4.7.4.3 Multispan Bridges

4.7.4.3.1 Selection of Method

For multispan structures, the minimum analysis requirements shall be as specified in Table 1 in which:

* = no seismic analysis required

UL = uniform load elastic method

SM = single-mode elastic method

MM = multimode elastic method

TH = time history method

C4.7.4.3.1

The selection of the method of analysis depends on seismic zone, regularity, and **operational classification** of the bridge.

Regularity is a function of the number of spans and the distribution of weight and stiffness. Regular bridges have less than seven spans; no abrupt or unusual changes in weight, stiffness, or geometry; and no large changes in these parameters from span to span or support-to-support, abutments excluded. A more rigorous analysis procedure may be used in lieu of the recommended minimum.

Table 4.7.4.3.1-1 Minimum Analysis Requirements for Seismic Effects.

Seismic Zone	Single-Span Bridges	Multispan Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		regular	irregular	regular	irregular	regular	irregular
1	No seismic analysis required	*	*	*	*	*	*
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	TH
4		SM/UL	MM	MM	MM	TH	TH

Except as specified below, bridges satisfying the requirements of Table 2 may be taken as "regular" bridges. Bridges not satisfying the requirements of Table 2 shall be taken as "irregular" bridges.

Table 4.7.4.3.1-2 Regular Bridge Requirements.

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	90°	90°	90°	90°	90°
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

Curved bridges comprised of multiple simple-spans shall be considered to be "irregular" if the subtended angle in plan is greater than 20°. Such bridges shall be analyzed by either the multimode elastic method or the time-history method.

A curved continuous-girder bridge may be analyzed as if it were straight, provided all of the following requirements are satisfied:

- The bridge is "regular" as defined in Table 2, except that for a two-span bridge the maximum span length ratio from span to span must not exceed 2;

- The subtended angle in plan is not greater than 90°; and
- The span lengths of the equivalent straight bridge are equal to the arc lengths of the curved bridge.

If these requirements are not satisfied, then curved continuous-girder bridges must be analyzed using the actual curved geometry.

4.7.4.3.2 Single-Mode Methods of Analysis

4.7.4.3.2a General

Either of the two single-mode methods of analysis specified herein may be used where appropriate.

4.7.4.3.2b Single-Mode Spectral Method

The single-mode method of spectral analysis shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction. For regular bridges, the fundamental modes of vibration in the horizontal plane coincide with the longitudinal and transverse axes of the bridge structure. This mode shape may be found by applying a uniform horizontal load to the structure and calculating the corresponding deformed shape. The natural period may be calculated by equating the maximum potential and kinetic energies associated with the fundamental mode shape. The amplitude of the displaced shape may be found from the elastic seismic response coefficient, C_{sm}, specified in Article 3.10.4.2, and the corresponding spectral displacement. This amplitude shall be used to determine force effects.

C4.7.4.3.2b

The single-mode spectral analysis method described in the following steps may be used for both transverse and longitudinal earthquake motions. Examples illustrating its application are given in AASHTO (*1983*) and ATC (*1981*).

- Calculate the static displacements $v_s(x)$ due to an assumed uniform loading p_o as shown in Figure C1:

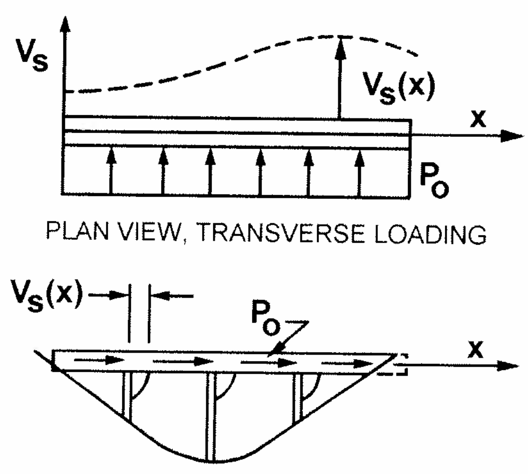

Figure C4.7.4.3.2b-1 Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading.

- Calculate factors α, β, and γ as:

$$\alpha = \int v_s(x)\,dx \qquad \text{(C4.7.4.3.2b-1)}$$

$$\beta = \int w(x)\,v_s(x)\,dx \qquad \text{(C4.7.4.3.2b-2)}$$

$$\gamma = \int w(x) v_s^2(x) dx \qquad \text{(C4.7.4.3.2b-3)}$$

where:

p_o = a uniform load arbitrarily set equal to 1.0 (kip/ft.)

$v_s(x)$ = deformation corresponding to p_o (ft.)

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (kip/ft.)

The computed factors, α, β, and γ have units of (ft.^{2}), (kip-ft.), and (kip-ft.^{2}), respectively.

- Calculate the period of the bridge as:

$$T_m = 2\pi \sqrt{\frac{\gamma}{p_o g \alpha}} \qquad \text{(C4.7.4.3.2b-4)}$$

where:

g = acceleration of gravity (ft./sec.^{2})

- Using T_m and Eq. 3.10.6.1-1, calculate C_{sm}
- Calculate the equivalent static earthquake loading $p_e(x)$ as:

$$p_e(x) = \frac{\beta C_{sm}}{\gamma} w(x) v_s(x) \qquad \text{(C4.7.4.3.2b-5)}$$

where:

C_{sm} = the dimensionless elastic seismic response coefficient given by Eq. 3.10.6.1-1

$p_e(x)$ = the intensity of the equivalent static seismic loading applied to represent the primary mode of vibration (kip/ft.)

- Apply loading $p_e(x)$ to the structure, and determine the resulting member force effects.

4.7.4.3.2c Uniform Load Method

The uniform load method shall be based on the fundamental mode of vibration in either 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 elastic seismic response coefficient, C_{sm}, specified in Article 3.10.6 shall be used to calculate the equivalent uniform seismic load from which seismic force effects are found.

C4.7.4.3.2c

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. Whereas all displacements and most member forces are calculated with good accuracy, the method is known to overestimate the transverse shears at the abutments by up to 100 percent. If such conservatism is undesirable, then the single-mode spectral analysis method specified in Article 4.7.4.3.2b is recommended.

- Calculate the static displacements $v_s(x)$ due to an assumed uniform load p_o, as shown in Figure C4.7.4.3.2b-1. The uniform loading p_o is applied over the length of the bridge; it has units of force per unit length and may be arbitrarily set equal to 1.0. The static displacement $v_s(x)$ has units of length.

- Calculate the bridge lateral stiffness, K, and total weight, W, from the following expressions:

$$K = \frac{p_o L}{v_{s,MAX}} \qquad \text{(C4.7.4.3.2c-1)}$$

$$W = \int w(x)dx \qquad \text{(C4.7.4.3.2c-2)}$$

where:

L = total length of the bridge (ft.)

$v_{s,MAX}$ = maximum value of $v_s(x)$ (ft.)

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (kip/ft.)

The weight should 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. Generally, the inertia effects of live loads are not included in the analysis; however, the probability of a large live load being on the bridge during an earthquake should 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.

- Calculate the period of the bridge, T_m, using the expression:

$$T_m = 2\pi\sqrt{\frac{W}{gK}} \qquad \text{(C4.7.4.3.2c-3)}$$

where:

g = acceleration of gravity (ft./sec.^{2})

- Calculate the equivalent static earthquake loading p_e from the expression:

$$p_e = \frac{C_{sm} W}{L} \qquad \text{(C4.7.4.3.2c-4)}$$

where:

C_{sm} = the dimensionless elastic seismic response coefficient given by Eq. 3.10.6.1-1

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

- 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.

4.7.4.3.3 Multimode Spectral Method

The multimode spectral 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 should be at least three times the number of spans in the model. The elastic seismic response spectrum as specified in Article 3.10.6 shall be used for each mode.

The member forces and displacements 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.

C4.7.4.3.3

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 well-separated modes. For closely spaced modes, the absolute sum of the modal responses should be used.

4.7.4.3.4 Time-History Method

Any step-by-step time-history method of analysis used for either elastic or inelastic analysis shall satisfy the requirements of Article 4.7.

The sensitivity of the numerical solution to the size of the time step used for the analysis shall be determined. A sensitivity study shall also be carried out to investigate the effects of variations in assumed material hysteretic properties.

The time histories of input acceleration used to describe the earthquake loads shall be selected in consultation with the Owner. Unless otherwise directed, five spectrum-compatible time histories shall be used when site-specific time histories are not available. The spectrum used to generate these five time histories shall be the same as that used for the modal methods, as specified in Article 3.10.6, modified for the appropriate soil profile.

C4.7.4.3.4

Rigorous methods of analysis are required for critical structures, which are defined in Article 3.10.3, and/or those that are geometrically complex or close to active earthquake faults. Time history methods of analysis are recommended for this purpose, provided care is taken with both the modeling of the structure and the selection of the input time histories of ground acceleration.

Site-specific spectrum is preferred, if available.

4.7.4.4 Minimum Displacement Requirements

Bridge seat widths at expansion bearings without restrainers, STUs, or dampers shall either accommodate the greater of the maximum displacement calculated in accordance with the provisions of Article 4.7.4.3, except for bridges in Zone 1, or a percentage of the empirical seat width, *N*, specified by Eq. 1. Otherwise, longitudinal restrainers complying with Article 3.10.9.5 shall be provided. Bearings restrained for longitudinal movement shall be designed in compliance with Article 3.10.9. The percentages of *N*, applicable to each seismic zone, shall be as specified in Table 1.

The empirical seat width shall be taken as:

$$N = (8 + 0.02L + 0.08H)(1 + 0.000125S^2) \qquad (4.7.4.4\text{-}1)$$

where:

N = minimum support length measured normal to the centerline of bearing (in.)

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within a span, L shall be the sum of the distances to either side of the hinge; for single-span bridges, L equals the length of the bridge deck (ft.)

H = for abutments, average height of columns supporting the bridge deck to the next expansion joint (ft.)

for columns and/or piers, column, or pier height (ft.)

for hinges within a span, average height of the adjacent two columns or piers (ft.)

0.0 for single-span bridges (ft.)

S = skew of support measured from line normal to span (°)

Table 4.7.4.4-1 Percentage *N* by Zone and Acceleration Coefficient.

Zone	Acceleration Coefficient	Soil Type	Percent *N*
1	<0.025	I or II	≥50
1	<0.025	III or IV	100
1	>0.025	All	100
2	All Applicable	All	100
3	All Applicable	All	150
4	All Applicable	All	150

for columns and/or piers, column, or pier height (ft.)

for hinges within a span, average height of the adjacent two columns or piers (ft.)

0.0 for single-span bridges (ft.)

S = skew of support measured from line normal to span (°)

Table 4.7.4.4-1 Percentage N by Zone and Acceleration Coefficient A_S, Specified in Eq. 3.10.4.2-2.

Zone	Acceleration Coefficient, A_S	Percent, N
1	≤ 0.05	≥ 75
1	**≥ 0.05**	100
2	All Applicable	150
3	All Applicable	150
4	All Applicable	150

4.7.4.5 P-Δ Requirements

The displacement of any column or pier in the longitudinal or transverse direction shall satisfy:

$$\Delta P_u < 0.25\phi M_n \tag{4.7.4.5-1}$$

in which:

$$\Delta = R_d \Delta_e \tag{4.7.4.5-2}$$

- If $T < 1.25T_s$, then:

$$R_d = \left(1 - \frac{1}{R}\right)\frac{1.25T_s}{T} + \frac{1}{R} \tag{4.7.4.5-3}$$

- If $T \geq 1.25T_s$, then:

$$R_d = 1 \tag{4.7.4.5-4}$$

where:

Δ = displacement of the point of contraflexure in the column or pier relative to the point of fixity for the foundation (in.)

Δ_e = displacement calculated from elastic seismic analysis (in.)

T = period of fundamental mode of vibration (sec.)

T_S = reference period specified in Article 3.10.4.2 (sec.)

C4.7.4.5

Bridges subject to earthquake ground motion may be susceptible to instability due to P-Δ effects. Inadequate strength can result in ratcheting of structural displacements to larger and larger values causing excessive ductility demand on plastic hinges in the columns, large residual deformations, and possibly collapse. The maximum value for Δ given in this Article is intended to limit the displacements such that P-Δ effects will not significantly affect the response of the bridge during an earthquake.

P-Δ effects lead to a loss in strength once yielding occurs in the columns of a bridge. In severe cases, this can result in the force-displacement relationship having a negative slope once yield is fully developed. The value for Δ given by Eq. 1 is such that this reduction in strength is limited to 25 percent of the yield strength of the pier or bent.

An explicit P-Δ check was not required in the previous edition of these Specifications but has been introduced herein because two conservative provisions have been relaxed in this revised edition. These are:

- The shape of the response spectrum (Figure 3.10.4.1-1) has been changed from being proportional to $1/T^{2/3}$ to $1/T$. The reason for the $1/T^{2/3}$ provision in the previous edition was to give conservative estimates of force and displacement in bridges with longer periods (>1.0 secs) which, in an indirect way, provided for such effects as P-Δ. With the change of the spectrum to being proportional to $1/T$, an explicit check for P-Δ is now required.

R = R-factor specified in Article 3.10.7

P_u = axial load on column or pier (kip)

ϕ = flexural resistance factor for column specified in Article 5.10.11.4.1b

M_n = nominal flexural strength of column or pier calculated at the axial load on the column or pier(kip-ft.)

- The flexural resistance factor, ϕ, for seismic design of columns with high axial loads has been increased from a minimum value of 0.5 to 0.9 (Article 5.10.11.4.1b). Use of a low resistance factor led to additional strength being provided in heavily loaded columns that could be used to offset reductions due to P-Δ, in the previous edition. The increased value for ϕ now permitted in Section 5 is a second reason for requiring an explicit check for P-Δ.

4.7.5 Analysis for Collision Loads

Where permitted by the provisions of Section 3, dynamic analysis for ship collision may be replaced by an equivalent static elastic analysis. Where an inelastic analysis is specified, the effect of other loads that may also be present shall be considered.

4.7.6 Analysis of Blast Effects

As a minimum, bridge components analyzed for blast forces should be designed for the dynamic effects resulting from the blast pressure on the structure. The results of an equivalent static analysis shall not be used for this purpose.

C4.7.6

Localized spall and breach damage should be accounted for when designing bridge components for blast forces. Data available at the time these provisions were developed are not sufficient to develop expressions for estimating the extent of spall/breach in concrete columns; however, spall and breach damage can be estimated for other types of components using guidelines found in Department of the Army (1986).

Due to the uncertainties that exist when considering likely attack scenarios and associated blast loads, an appropriate equivalent static load can not be used for design. Moreover, the highly impulsive nature of blast loads warrants the consideration of inertial effects during the analysis of a structural component. Therefore, an equivalent static analysis is not acceptable for the design of any structural member. Information on designing structures to resist blast loads may be found in ASCE (*1997*), Department of the Army (*1990*), E. J. Conrath, et al. (*1999*), J. M. Biggs (*1964*), and W. Bounds (*1998*).

4.8 ANALYSIS BY PHYSICAL MODELS

4.8.1 Scale Model Testing

To establish and/or to verify structural behavior, the Owner may require the testing of scale models of structures and/or parts thereof. The dimensional and material properties of the structure, as well as its boundary conditions and loads, shall be modeled as accurately as possible. For dynamic analysis, inertial scaling, load/excitation, and damping functions shall be applied as appropriate. For strength limit state tests, factored dead load shall be simulated. The instrumentation shall not significantly influence the response of the model.