The effect of snow loads on a bridge structure is influenced by the pattern of snow accumulation. Windblown snow drifts may produce unbalanced loads considerably greater than those produced from uniformly distributed loads. Drifting is influenced by the terrain, structure shape, and other features that cause changes in the general wind flow. Bridge components, such as railings, can serve to contain drifting snow and cause large accumulations to develop.

3.10 EARTHQUAKE EFFECTS: *EQ*

3.10.1 General

Bridges shall be designed to have a low probability of collapse but may suffer significant damage and disruption to service when subject to earthquake ground motions that have a 7 percent probability of exceedance in 75 years. Partial or complete replacement may be required. Higher levels of performance may be used with the authorization of the bridge owner.

Earthquake loads shall be taken to be horizontal force effects determined in accordance with the provisions of Article 4.7.4 on the basis of the elastic response coefficient, C_{sm}, specified in Article 3.10.4, and the equivalent weight of the superstructure, and adjusted by the response modification factor, R, specified in Article 3.10.7.1.

The provisions herein shall apply to bridges of conventional **construction. The** Owner shall specify and/or approve appropriate provisions **for nonconventional construction**. Unless otherwise specified by the Owner, these provisions need not be applied to completely buried structures.

Seismic effects for box culverts and buried structures need not be considered, except where they cross active faults.

The potential for soil liquefaction and slope movements shall be considered.

C3.10.1

The design earthquake motions and forces specified in these provisions are based on a low probability of their being exceeded during the normal life expectancy of a bridge. Bridges that are designed and detailed in accordance with these provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking.

The principles used for the development of these Specifications are:

- Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage;
- Realistic seismic ground motion intensities and forces should be used in the design procedures; and
- Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

Bridge Owners may choose to mandate higher levels of performance for special bridges.

Earthquake loads are given by the product of the elastic seismic response coefficient C_{sm} and the equivalent weight of the superstructure. The equivalent weight is a function of the actual weight and bridge configuration and is automatically included in both the single-mode and multimode methods of analysis specified in Article 4.7.4. Design and detailing provisions for bridges to minimize their susceptibility to damage from earthquakes are contained in Sections 3, 4, 5, 6, 7, 10, and 11. A flow chart summarizing these provisions is presented in Appendix A3.

Conventional bridges include those with slab, beam, box girder, or truss superstructures, and single- or multiple-column piers, wall-type piers, or pile-bent substructures. In addition, conventional bridges are founded on shallow or piled footings, or shafts. Substructures for conventional bridges are also listed in Table 3.10.7.1-1. Nonconventional bridges include bridges with cable-stayed/cable-suspended superstructures, bridges with truss towers or hollow piers for substructures, and arch bridges.

These Specifications are considered to be "force-based" wherein a bridge is designed to have adequate strength (capacity) to resist earthquake forces (demands). In recent years there has been a trend away from "force-based" procedures to those that are "displacement-based," wherein a bridge is designed to have adequate displacement capacity to accommodate earthquake demands. Displacement-based procedures are believed to more reliably identify the limit states that cause damage leading to collapse, and in some cases produce more efficient designs against collapse. It is recommended that the displacement capacity of bridges designed in accordance with these Specifications, be checked using a displacement-based procedure, particularly those bridges in high seismic zones. The *AASHTO Guide Specifications for LRFD Seismic Design* (***AASHTO, 2008***), are "displacement-based."

3.10.2 Seismic Hazard

The seismic hazard at a bridge site shall be characterized by the acceleration response spectrum for the site and the site factors for the relevant site class.

The acceleration spectrum shall be determined using either the General Procedure specified in Article 3.10.2.1 or the Site Specific Procedure specified in Article 3.10.2.2.

A Site-Specific Procedure shall be used if any one of the following conditions exist:

- The site is located within 6 mi. of an active fault,
- The site is classified as Site Class F (Article 3.10.3.1),
- Long-duration earthquakes are expected in the region,
- The importance of the bridge is such that a lower probability of exceedance (and therefore a longer return period) should be considered.

If time histories of ground acceleration are used to characterize the seismic hazard for the site, they shall be determined in accordance with Article 4.7.4.3.4b.

This page is intentionally left blank.

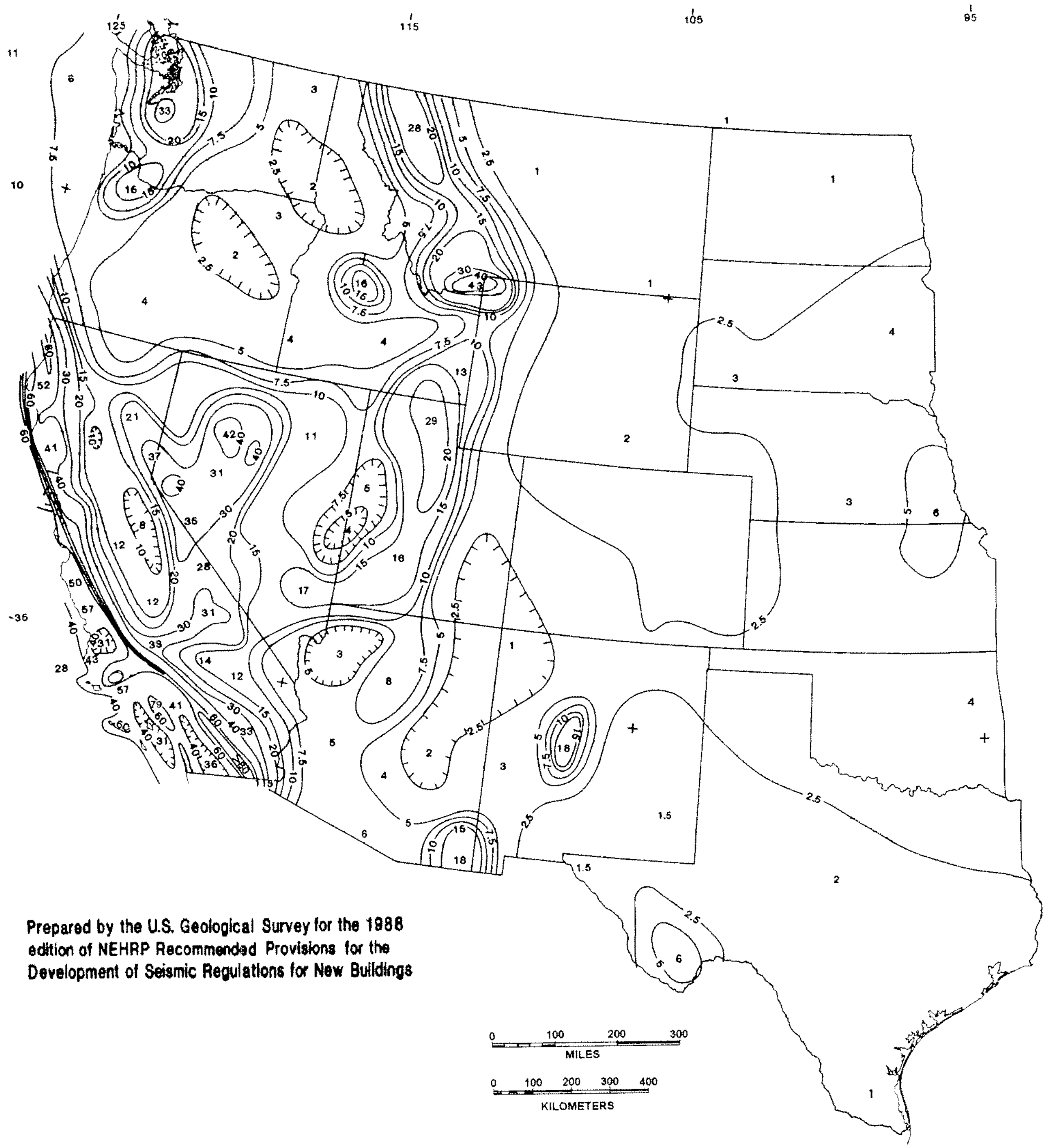

Figure 3.10.2-1 Acceleration Coefficient for Contiguous States Generally West of the 95th Longitude.

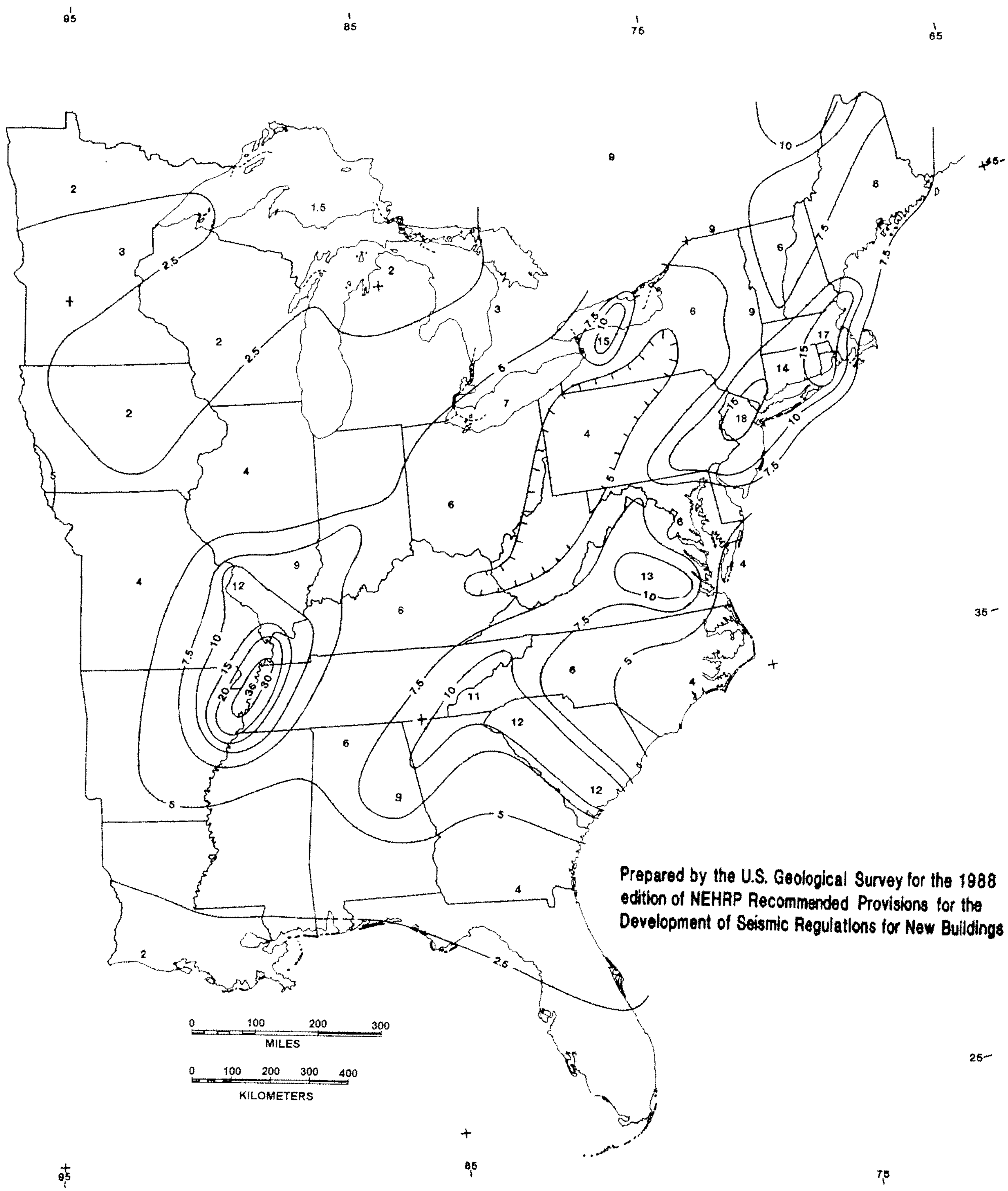

Figure 3.10.2-2 Acceleration Coefficient for Contiguous States Generally East of the 95th Longitude.

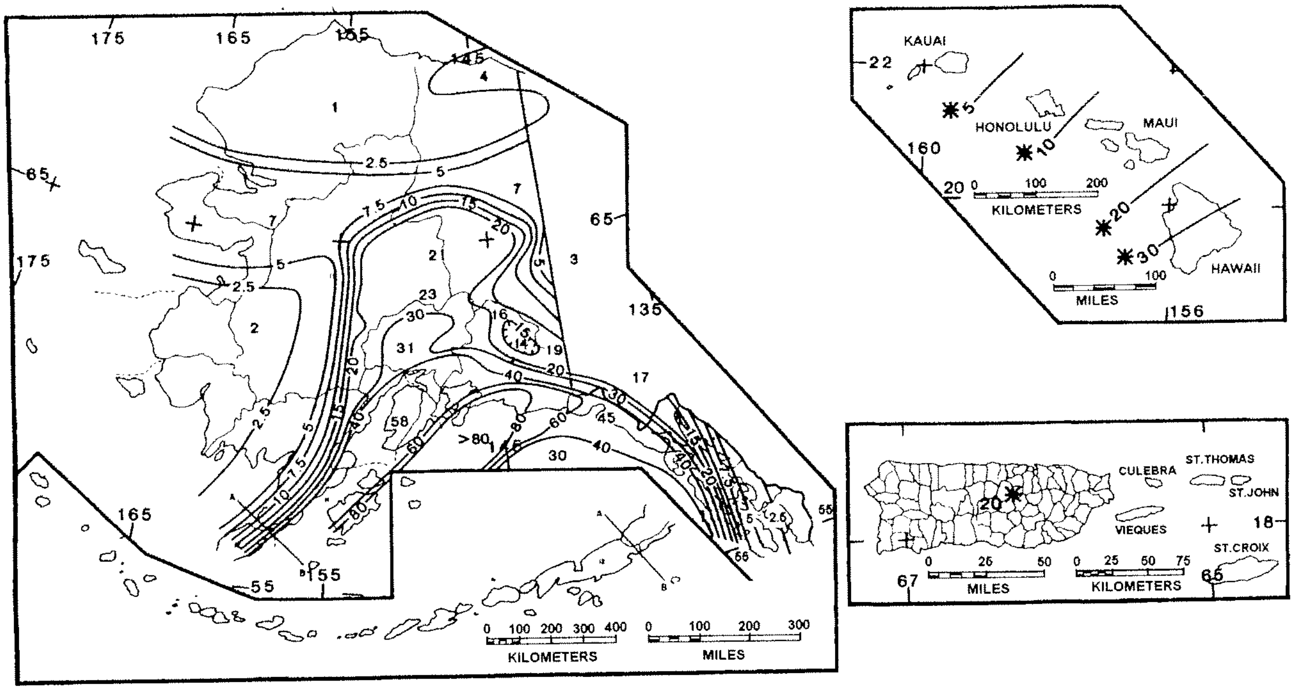

Figure 3.10.2-3 Acceleration Coefficient for Alaska, Hawaii, and Puerto Rico.

3.10.3 Importance Categories

For the purpose of Article 3.10, the Owner or those having jurisdiction shall classify the bridge into one of three importance categories as follows:

- Critical bridges,
- Essential bridges, or
- Other bridges.

The basis of classification shall include social/survival and security/defense requirements. In classifying a bridge, consideration should be given to possible future changes in conditions and requirements.

C3.10.3

Essential bridges are generally those that should, as a minimum, be open to emergency vehicles and for security/defense purposes immediately after the design earthquake, i.e., a 475-year return period event. However, some bridges must remain open to all traffic after the design earthquake and be usable by emergency vehicles and for security/defense purposes immediately after a large earthquake, e.g., a 2,500-year return period event. These bridges should be regarded as critical structures.

3.10.4 Seismic Performance Zones

Each bridge shall be assigned to one of the four seismic zones in accordance with Table 1.

Table 3.10.4-1 Seismic Zones.

Acceleration Coefficient	Seismic Zone
$A \le 0.09$	1
$0.09 < A \le 0.19$	2
$0.19 < A \le 0.29$	3
$0.29 < A$	4

C3.10.4

These seismic zones reflect the variation in seismic risk across the country and are used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures.

3.10.5 Site Effects

3.10.5.1 General

Site effects shall be included in the determination of seismic loads for bridges.

The site coefficient, *S*, specified in Table 1, shall be based upon soil profile types defined in Articles 3.10.5.2 through 3.10.5.5.

C3.10.5.1

Site effects on structural response are due to the soil conditions. Four soil profiles are used in these Specifications to define a site coefficient used to modify the acceleration coefficient. These soil profiles are representative of different subsurface conditions, which were selected on the basis of a statistical study of spectral shapes developed on such soils close to seismic source zones in past earthquakes.

The site coefficient, *S*, is used to include the effect of site conditions on the elastic seismic response coefficient as specified in Article 3.10.6.

Table 3.10.5.1-1 Site Coefficients.

Site Coefficient	Soil Profile Type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

In locations where the soil properties are not known in sufficient detail to determine the soil profile type, or where the profile does not fit any of the four types, the site coefficient for Soil Profile Type II shall be used.

The decision to specify Type II as a default site coefficient was a committee decision based on judgment during the development of the parent provisions under Project ATC-6.

3.10.5.2 Soil Profile Type I

A profile shall be taken as Type I if composed of:

- Rock of any description, either shale-like or crystalline in nature, or
- Stiff soils where the soil depth is less than 200 ft., and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays.

C3.10.5.2

These materials may be characterized by a shear wave velocity greater than 2,500 ft./sec.

3.10.5.3 Soil Profile Type II

A profile with stiff cohesive or deep cohesionless soils where the soil depth exceeds 200 ft. and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays shall be taken as Type II.

3.10.5.4 Soil Profile Type III

A profile with soft to medium-stiff clays and sands, characterized by 30.0 ft. or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils shall be taken as Type III.

3.10.5.5 Soil Profile Type IV

A profile with soft clays or silts greater than 40.0 ft. in depth shall be taken as Type IV.

C3.10.5.5

These materials may be characterized by a shear wave velocity of less than 500 ft./sec. and might include loose natural deposits or manmade, nonengineered fill.

3.10.6 Elastic Seismic Response Coefficient

3.10.6.1 General

Unless specified otherwise in Article 3.10.6.2, the elastic seismic response coefficient, C_{sm}, for the m^{th} mode of vibration shall be taken as:

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A \qquad (3.10.6.1\text{-}1)$$

where:

T_m = period of vibration of the m^{th} mode (sec.)

A = acceleration coefficient specified in Article 3.10.2

S = site coefficient specified in Article 3.10.5

The determination of the period of vibration, T_m, should be based on the nominal, unfactored mass of the component or structure.

C3.10.6.1

The elastic seismic response coefficient may be normalized using the input ground acceleration A and the result plotted against the period of vibration. Such a plot is given in Figure C1 for different soil profiles, based on 5 percent damping.

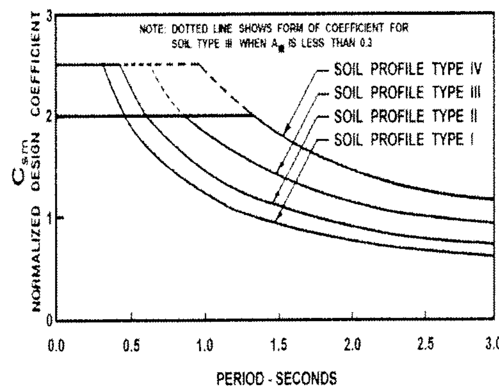

Figure C3.10.6.1-1 Seismic Response Coefficients for Various Soil Profiles, Normalized with Respect to Acceleration Coefficient *A*.

An earthquake may excite several modes of vibration in a bridge and, therefore, the elastic response coefficient should be found for each relevant mode.

The discussion of the single-mode method in the commentary to Article 4.7.4.3.2 is used to illustrate the relation between period, C_{sm}, and quasi-static seismic forces, $P_e(x)$. The structure is analyzed for these seismic forces in the single-mode method. In the multimode method, the structure is analyzed for several seismic forces, each corresponding to the period and mode shape of one of the fundamental modes of vibration, and the results are combined using acceptable methods, such as the root-mean-square method.

3.10.6.2 Exceptions

For bridges on soil profiles III or IV and in areas where the coefficient A is not less than 0.30, C_{sm} need not exceed $2.0A$.

For soil profiles III and IV, and for modes other than the fundamental mode that have periods less than 0.3 sec., C_{sm} shall be taken as:

$$C_{sm} = A(0.8 + 4.0T_m) \qquad (3.10.6.2\text{-}1)$$

If the period of vibration for any mode exceeds 4.0 sec., the value of C_{sm} for that mode shall be taken as:

$$C_{sm} = \frac{3AS}{T_m^{4/3}} \quad (3.10.6.2\text{-}2)$$

3.10.7 Response Modification Factors

3.10.7.1 General

To apply the response modification factors specified herein, the structural details shall satisfy the provisions of Articles 5.10.2.2, 5.10.11, and 5.13.4.6.

Except as noted herein, seismic design force effects for substructures and the connections between parts of structures, listed in Table 2, shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor, R, as specified in Tables 1 and 2, respectively.

As an alternative to the use of the R-factors, specified in Table 2 for connections, monolithic joints between structural members and/or structures, such as a column-to-footing connection, may be designed to transmit the maximum force effects that can be developed by the inelastic hinging of the column or multicolumn bent they connect as specified in Article 3.10.9.4.3.

If an inelastic time history method of analysis is used, the response modification factor, R, shall be taken as 1.0 for all substructure and connections.

C3.10.7.1

These Specifications recognize that it is uneconomical to design a bridge to resist large earthquakes elastically. Columns are assumed to deform inelastically where seismic forces exceed their design level, which is established by dividing the elastically computed force effects by the appropriate R-factor.

R-factors for connections are smaller than those for substructure members in order to preserve the integrity of the bridge under these extreme loads. For expansion joints within the superstructure and connections between the superstructure and abutment, the application of the R-factor results in force effect magnification. Connections that transfer forces from one part of a structure to another include, but are not limited to, fixed bearings, expansion bearings with either restrainers, STUs, or dampers, and shear keys. For one-directional bearings, these R-factors are used in the restrained direction only. In general, forces determined on the basis of plastic hinging will be less than those given by using Table 2, resulting in a more economical design.

Table 3.10.7.1-1 Response Modification Factors—Substructures.

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers—larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
• Vertical pile only	1.5	3.5	5.0
• With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

Table 3.10.7.1-2 Response Modification Factors—Connections.

Connection	All Importance Categories
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundations	1.0

3.10.7.2 Application

Seismic loads shall be assumed to act in any lateral direction.

The appropriate R-factor shall be used for both orthogonal axes of the substructure.

A wall-type concrete pier may be analyzed as a single column in the weak direction if all the provisions for columns, as specified in Section 5, are satisfied.

C3.10.7.2

Usually the orthogonal axes will be the longitudinal and transverse axes of the bridge. In the case of a curved bridge, the longitudinal axis may be the chord joining the two abutments.

Wall-type piers may be treated as wide columns in the strong direction, provided the appropriate R-factor in this direction is used.

3.10.8 Combination of Seismic Force Effects

The elastic seismic force effects on each of the principal axes of a component resulting from analyses in the two perpendicular directions shall be combined to form two load cases as follows:

- 100 percent of the absolute value of the force effects in one of the perpendicular directions combined with 30 percent of the absolute value of the force effects in the second perpendicular direction, and
- 100 percent of the absolute value of the force effects in the second perpendicular direction combined with 30 percent of the absolute value of the force effects in the first perpendicular direction.

Where foundation and/or column connection forces are determined from plastic hinging of the columns specified in Article 3.10.9.4.3, the resulting force effects may be determined without consideration of combined load cases specified herein. For the purpose of this provision, "column connection forces" shall be taken as the shear and moment, computed on the basis of plastic hinging. The axial load shall be taken as that resulting from the appropriate load combination with the axial load, if any, associated with plastic hinging taken as *EQ*. If a pier is designed as a column as specified in Article 3.10.7.2, this exception shall be taken to apply for the weak direction of the pier where force effects resulting from plastic hinging are used; the combination load cases specified must be used for the strong direction of the pier.

C3.10.8

The exception to these load combinations indicated at the end of this section should also apply to bridges in Zone 2 where foundation forces are determined from plastic hinging of the columns.

3.10.9 Calculation of Design Forces

3.10.9.1 General

For single-span bridges, regardless of seismic zone, the minimum design connection force effect in the restrained direction between the superstructure and the substructure shall not be less than the product of the site coefficient, the acceleration coefficient, and the tributary permanent load.

Seat widths at expansion bearings of multispan bridges shall either comply with Article 4.7.4.4 or STUs, and dampers shall be provided.

C3.10.9.1

This Article refers to superstructure effects carried into substructure. Abutments on multispan bridges, but not single-span bridges, and retaining walls are subject to acceleration-augmented soil pressures as specified in Articles 3.11.4 and 11.6.5. Wingwalls on single-span structures are not fully covered at this time, and the Engineer should use judgment in this area.

3.10.9.2 Seismic Zone 1

For bridges on sites in Zone 1 where the acceleration coefficient is less than 0.025 and the soil profile is either Type I or Type II, the horizontal design connection force in the restrained directions shall not be taken to be less than 0.1 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

For all other sites in Zone 1, the horizontal design connection force in the restrained directions shall not be taken to be less than 0.2 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment.

If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing.

Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in Seismic Zone 1 and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

C3.10.9.2

These provisions arise because, as specified in Article 4.7.4, seismic analysis for bridges in Zone 1 is not generally required. These default values are used as minimum design forces in lieu of rigorous analysis. The division of Zone 1 at an acceleration coefficient 0.025 for sites with favorable soil condition is an arbitrary expedience intended to provide some relief to parts of the country with very low seismicity.

If each bearing supporting a continuous segment or simply supported span is an elastomeric bearing, there are no restrained directions due to the flexibility of the bearings.

The magnitude of live load assumed to exist at the time of the earthquake should be consistent with the value of γ_{eq} used in conjunction with Table 3.4.1-1.

3.10.9.3 Seismic Zone 2

Structures in Seismic Zone 2 shall be analyzed according to the minimum requirements specified in Articles 4.7.4.1 and 4.7.4.3.

Except for foundations, seismic design forces for all components, including pile bents and retaining walls, shall be determined by dividing the elastic seismic forces, obtained from Article 3.10.8, by the appropriate response modification factor, R, specified in Table 3.10.7.1-1.

Seismic design forces for foundations, other than pile bents and retaining walls, shall be determined by dividing elastic seismic forces, obtained from Article 3.10.8, by half of the response modification factor, R, from Table 3.10.7.1-1, for the substructure component to which it is attached. The value of $R/2$ shall not be taken as less than 1.0.

Where a group load other than EXTREME EVENT I, specified in Table 3.4.1-1, governs the design of columns, the possibility that seismic forces transferred to the foundations may be larger than those calculated using the procedure specified above, due to possible overstrength of the columns, shall be considered.

C3.10.9.3

This Article specifies the design forces for foundations which include the footings, pile caps and piles. The design forces are essentially twice the seismic design forces of the columns. This will generally be conservative and was adopted to simplify the design procedure for bridges in Zone 2. However, if seismic forces do not govern the design of columns and piers there is a possibility that during an earthquake the foundations will be subjected to forces larger than the design forces. For example, this may occur due to unintended column overstrengths which may exceed the capacity of the foundations. An estimate of this effect may be found by using a resistance factor, ϕ, of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. It is also possible that even in cases when seismic loads govern the column design, the columns may have insufficient shear strength to enable a ductile flexural mechanism to develop, but instead allow a brittle shear failure to occur. Again, this situation is due to potential overstrength in the flexural capacity of columns and could possibly be prevented by arbitrarily increasing the column design shear by the overstrength factor cited above.

Conservatism in the design, and in some cases underdesign, of foundations and columns in Zone 2 based on the simplified procedure of this Article has been widely debated (*Gajer and Wagh 1994*). In light of the above discussion, it is recommended that for critical or essential bridges in Zone 2 consideration should be given to the use of the forces specified in Article 3.10.9.4.3f for foundations in Zone 3 and Zone 4. Ultimate soil and pile strengths are to be used with the specified foundation seismic design forces.

3.10.9.4 Seismic Zones 3 and 4

3.10.9.4.1 General

Structures in Seismic Zones 3 and 4 shall be analyzed according to the minimum requirements specified in Articles 4.7.4.1 and 4.7.4.3.

The design forces of each component shall be taken as the lesser of those determined using:

- the provisions of Article 3.10.9.4.2; or
- the provisions of Article 3.10.9.4.3,

for all components of a column, column bent and its foundation and connections.

C3.10.9.4.1

In general, the design forces resulting from an R-factor and inelastic hinging analysis will be less than those from an elastic analysis. However, in the case of architecturally oversized column(s), the forces from an inelastic hinging analysis may exceed the elastic forces in which case the elastic forces may be used for that column, column bent and its connections and foundations.

3.10.9.4.2 Modified Design Forces

Modified design forces shall be determined as specified in Article 3.10.9.3, except that for foundations the R-factor shall be taken as 1.0.

C3.10.9.4.2

Acceptable damage is restricted to inelastic hinges in the columns. The foundations should, therefore, remain in their elastic range. Hence the value for the R-factor is taken as 1.0.

3.10.9.4.3 Inelastic Hinging Forces

3.10.9.4.3a General

Where inelastic hinging is invoked as a basis for seismic design, the force effects resulting from plastic hinging at the top and/or bottom of the column shall be calculated after the preliminary design of the columns has been completed utilizing the modified design forces specified in Article 3.10.9.4.2 as the seismic loads. The consequential forces resulting from plastic hinging shall then be used for determining design forces for most components as identified herein. The procedures for calculating these consequential forces for single column and pier supports and bents with two or more columns shall be taken as specified in the following Articles.

Inelastic hinges shall be ascertained to form before any other failure due to overstress or instability in the structure and/or in the foundation. Inelastic hinges shall only be permitted at locations in columns where they can be readily inspected and/or repaired. Inelastic flexural resistance of substructure components shall be determined in accordance with the provisions of Sections 5 and 6.

Superstructure and substructure components and their connections to columns shall also be designed to resist a lateral shear force from the column determined from the factored inelastic flexural resistance of the column using the resistance factors specified herein.

These consequential shear forces, calculated on the basis of inelastic hinging, may be taken as the extreme seismic forces that the bridge is capable of developing.

C3.10.9.4.3a

By virtue of Article 3.10.9.4.2, alternative conservative design forces are specified if plastic hinging is not invoked as a basis for seismic design.

In most cases, the maximum force effects on the foundation will be limited by the extreme horizontal force that a column is capable of developing. In these circumstances, the use of a lower force, lower than that specified in Article 3.10.9.4.2, is justified and should result in a more economic foundation design.

See also Appendix B3.

3.10.9.4.3b Single Columns and Piers

Force effects shall be determined for the two principal axes of a column and in the weak direction of a pier or bent as follows:

- Step 1—Determine the column overstrength moment resistance. Use a resistance factor, ϕ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. For both materials, the applied axial load in the column shall be determined using Extreme Event Load Combination I, with the maximum elastic column axial load from the seismic forces determined in accordance with Article 3.10.8 taken as *EQ*.

C3.10.9.4.3b

The use of the factors 1.3 and 1.25 corresponds to the normal use of a resistance factor for reinforced concrete. In this case, it provides an increase in resistance, i.e., overstrength. Thus, the term "overstrength moment resistance" denotes a factor resistance in the parlance of these Specifications.

- Step 2—Using the column overstrength moment resistance, calculate the corresponding column shear force. For flared columns, this calculation shall be performed using the overstrength resistances at both the top and bottom of the flare in conjunction with the appropriate column height. If the foundation of a column is significantly below ground level, consideration should be given to the possibility of the plastic hinge forming above the foundation. If this can occur, the column length between plastic hinges shall be used to calculate the column shear force.

Force effects corresponding to a single column hinging shall be taken as:

- Axial Forces—Those determined using Extreme Event Load Combination I, with the unreduced maximum and minimum seismic axial load of Article 3.10.8 taken as EQ.

- Moments—Those calculated in Step 1.

- Shear Force—That calculated in Step 2.

3.10.9.4.3c Piers with Two or More Columns

Force effects for bents with two or more columns shall be determined both in the plane of the bent and perpendicular to the plane of the bent. Perpendicular to the plane of the bent, the forces shall be determined as for single columns in Article 3.10.9.4.3b. In the plane of the bent, the forces shall be calculated as follows:

- Step 1—Determine the column overstrength moment resistances. Use a resistance factor, ϕ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. For both materials the initial axial load should be determined using the Extreme Event Load Combination I with $EQ = 0$.

- Step 2—Using the column overstrength moment resistance, calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the pier. If a partial-height wall exists between the columns, the effective column height should be taken from the top of the wall. For flared columns and foundations below ground level, the provisions of Article 3.10.9.4.3b shall apply. For pile bents, the length of pile above the mud line shall be used to calculate the shear force.

C3.10.9.4.3c

See Article C3.10.9.4.3b.

- Step 3—Apply the bent shear force to the center of mass of the superstructure above the pier and determine the axial forces in the columns due to overturning when the column overstrength moment resistances are developed.

- Step 4—Using these column axial forces as *EQ* in the Extreme Event Load Combination I, determine revised column overstrength moment resistance. With the revised overstrength moment resistances, calculate the column shear forces and the maximum shear force for the bent. If the maximum shear force for the bent is not within 10 percent of the value previously determined, use this maximum bent shear force and return to Step 3.

The forces in the individual columns in the plane of a bent corresponding to column hinging shall be taken as:

- Axial Forces—The maximum and minimum axial loads determined using Extreme Event Load Combination I, with the axial load determined from the final iteration of Step 3 taken as *EQ* and treated as plus and minus.

- Moments—The column overstrength moment resistances corresponding to the maximum compressive axial load specified above.

- Shear Force—The shear force corresponding to the column overstrength moment resistances specified above, noting the provisions in Step 2 above.

3.10.9.4.3d Column and Pile Bent Design Forces

Design forces for columns and pile bents shall be taken as a consistent set of the lesser of the forces determined as specified in Article 3.10.9.4.1, applied as follows:

- Axial Forces—The maximum and minimum design forces determined using Extreme Event Load Combination I with either the elastic design values determined in Article 3.10.8 taken as *EQ*, or the values corresponding to plastic hinging of the column taken as *EQ*.

- Moments—The modified design moments determined for Extreme Event Limit State Load Combination I.

C3.10.9.4.3d

The design axial forces which control both the flexural design of the column and the shear design requirements are either the maximum or minimum of the unreduced design forces or the values corresponding to plastic hinging of the columns. In most cases, the values of axial load and shear corresponding to plastic hinging of the columns will be lower than the unreduced design forces. The design shear forces are specified so that the possibility of a shear failure in the column is minimized.

When an inelastic hinging analysis is performed, these moments and shear forces are the maximum forces that can develop and, therefore, the directional load combinations of Article 3.10.8 do not apply.

- Shear Force—The lesser of either the elastic design value determined for Extreme Event Limit State Load Combination I with the seismic loads combined as specified in Article 3.10.8 and using an R-factor of 1 for the column, or the value corresponding to plastic hinging of the column.

3.10.9.4.3e Pier Design Forces

The design forces shall be those determined for Extreme Event Limit State Load Combination I, except where the pier is designed as a column in its weak direction. If the pier is designed as a column, the design forces in the weak direction shall be as specified in Article 3.10.9.4.3d and all the design requirements for columns, as specified in Section 5, shall apply. When the forces due to plastic hinging are used in the weak direction, the combination of forces, specified in Article 3.10.8, shall be applied to determine the elastic moment which is then reduced by the appropriate R-factor.

C3.10.9.4.3e

The design forces for piers specified in Article 3.10.9.4.3e are based on the assumption that a pier has low ductility capacity and no redundancy. As a result, a low R-factor of 2 is used in determining the reduced design forces, and it is expected that only a small amount of inelastic deformation will occur in the response of a pier when subjected to the forces of the design earthquake. If a pier is designed as a column in its weak direction, then both the design forces and, more importantly, the design requirements of Articles 3.10.9.4.3d and Section 5 are applicable.

3.10.9.4.3f Foundation Design Forces

The design forces for foundations including footings, pile caps and piles may be taken as either those forces determined for the Extreme Event Load Combination I, with the seismic loads combined as specified in Article 3.10.8, or the forces at the bottom of the columns corresponding to column plastic hinging as determined in Article 3.10.8.

When the columns of a bent have a common footing, the final force distribution at the base of the columns in Step 4 of Article 3.10.9.4.3c may be used for the design of the footing in the plane of the bent. This force distribution produces lower shear forces and moments on the footing because one exterior column may be in tension and the other in compression due to the seismic overturning moment. This effectively increases the ultimate moments and shear forces on one column and reduces them on the other.

C3.10.9.4.3f

The foundation design forces specified are consistent with the design philosophy of minimizing damage that would not be readily detectable. The recommended design forces are the maximum forces that can be transmitted to the footing by plastic hinging of the column. The alternate design forces are the elastic design forces. It should be noted that these may be considerably greater than the recommended design forces, although where architectural considerations govern the design of a column, the alternate elastic design forces may be less than the forces resulting from column plastic hinging.

See also the second paragraph of C3.10.9.4.3d.

3.10.9.5 Longitudinal Restrainers

Friction shall not be considered to be an effective restrainer.

Restrainers shall be designed for a force calculated as the acceleration coefficient times the permanent load of the lighter of the two adjoining spans or parts of the structure.

If the restrainer is at a point where relative displacement of the sections of superstructure is designed to occur during seismic motions, sufficient slack shall be allowed in the restrainer so that the restrainer does not start to act until the design displacement is exceeded.

Where a restrainer is to be provided at columns or piers, the restrainer of each span may be attached to the column or pier rather than to interconnecting adjacent spans.

3.10.5 Operational Classification

For the purpose of Article 3.10, the Owner or those having jurisdiction shall classify the bridge into one of three **operational** categories as follows:

- Critical bridges,
- Essential bridges, or
- Other bridges.

The basis of classification shall include social/survival and security/defense requirements. In classifying a bridge, consideration should be given to possible future changes in conditions and requirements.

C3.10.5

Essential bridges are generally those that should, as a minimum, be open to emergency vehicles and for security/defense purposes immediately after the design earthquake, i.e., a 1,000-year return period event. However, some bridges must remain open to all traffic after the design earthquake and be usable by emergency vehicles and for security/defense purposes immediately after a large earthquake, e.g., a 2,500-year return period event. These bridges should be regarded as critical structures.

3.10.6 Seismic Performance Zones

Each bridge shall be assigned to one of the four seismic zones in accordance with Table 1 using the value of S_{D1} given by Eq. 3.10.4.2-6.

Table 3.10.6-1 Seismic Zones.

Acceleration Coefficient, S_{D1}	Seismic Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

C3.10.6

These seismic zones reflect the variation in seismic risk across the country and are used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures.

3.10.7 Response Modification Factors

3.10.7.1 General

To apply the response modification factors specified herein, the structural details shall satisfy the provisions of Articles 5.10.2.2, 5.10.11, and 5.13.4.6.

Except as noted herein, seismic design force effects for substructures and the connections between parts of structures, listed in Table 2, shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor, R, as specified in Tables 1 and 2, respectively.

As an alternative to the use of the R-factors, specified in Table 2 for connections, monolithic joints between structural members and/or structures, such as a column-to-footing connection, may be designed to transmit the maximum force effects that can be developed by the inelastic hinging of the column or multicolumn bent they connect as specified in Article 3.10.9.4.3.

If an inelastic time history method of analysis is used, the response modification factor, R, shall be taken as 1.0 for all substructure and connections.

C3.10.7.1

These Specifications recognize that it is uneconomical to design a bridge to resist large earthquakes elastically. Columns are assumed to deform inelastically where seismic forces exceed their design level, which is established by dividing the elastically computed force effects by the appropriate R-factor.

R-factors for connections are smaller than those for substructure members in order to preserve the integrity of the bridge under these extreme loads. For expansion joints within the superstructure and connections between the superstructure and abutment, the application of the R-factor results in force effect magnification. Connections that transfer forces from one part of a structure to another include, but are not limited to, fixed bearings, expansion bearings with either restrainers, STUs, or dampers, and shear keys. For one-directional bearings, these R-factors are used in the restrained direction only. In general, forces determined on the basis of plastic hinging will be less than those given by using Table 2, resulting in a more economical design.

Table 3.10.7.1-1 Response Modification Factors—Substructures.

Substructure	**Operational** Category		
	Critical	Essential	Other
Wall-type piers—larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
• Vertical pile only	1.5	3.5	5.0
• With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

Table 3.10.7.1-2 Response Modification Factors—Connections.

Connection	All **Operational** Categories
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundations	1.0

3.10.7.2 Application

Seismic loads shall be assumed to act in any lateral direction.

The appropriate R-factor shall be used for both orthogonal axes of the substructure.

A wall-type concrete pier may be analyzed as a single column in the weak direction if all the provisions for columns, as specified in Section 5, are satisfied.

C3.10.7.2

Usually the orthogonal axes will be the longitudinal and transverse axes of the bridge. In the case of a curved bridge, the longitudinal axis may be the chord joining the two abutments.

Wall-type piers may be treated as wide columns in the strong direction, provided the appropriate R-factor in this direction is used.