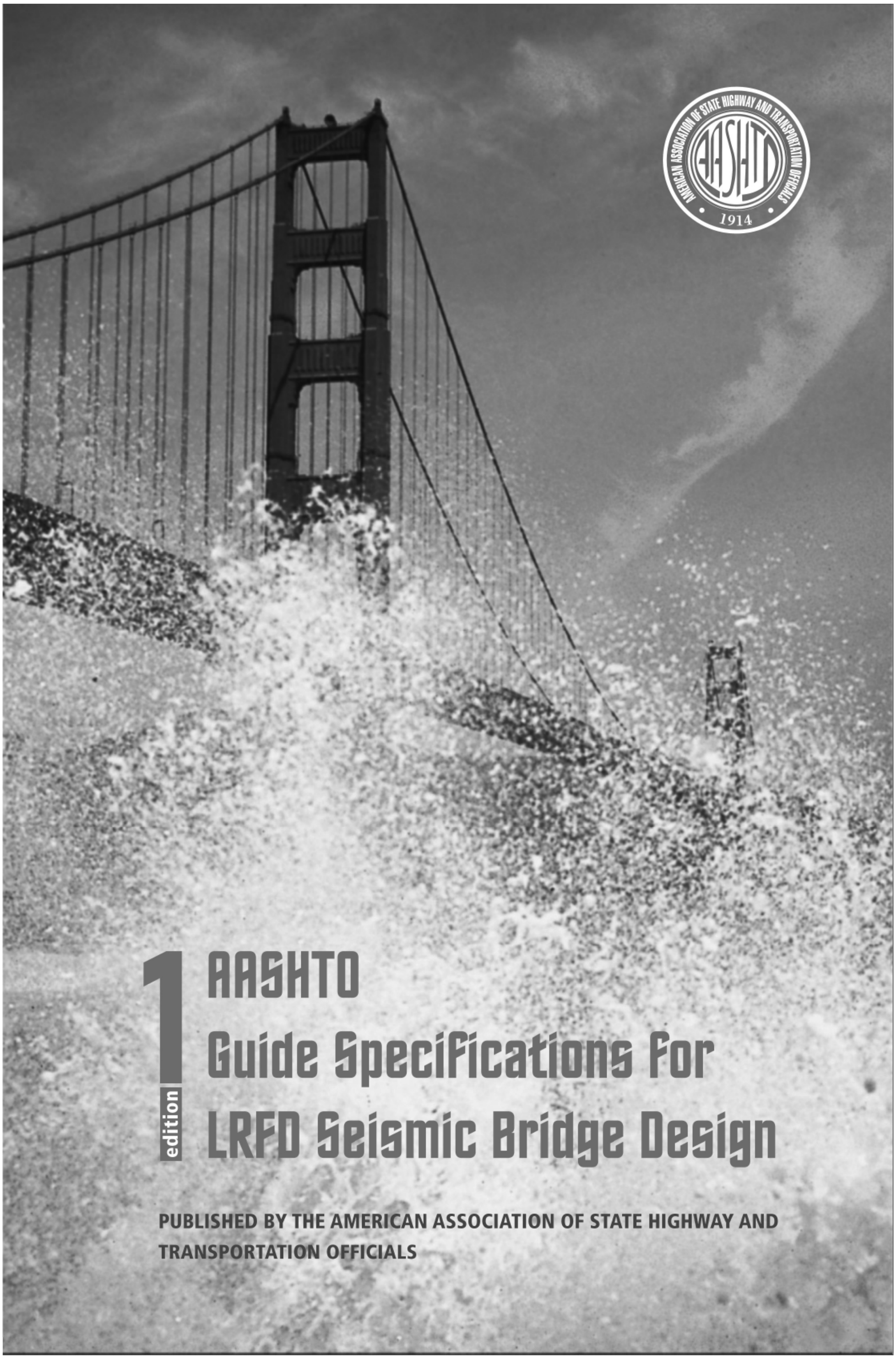

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AASHTO Guide Specifications for LRFD Seismic Bridge Design

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FOREWORD

Following the 1971 San Fernando earthquake, significant effort was expended to develop comprehensive design guidelines for the seismic design of bridges. That effort led to updates of both the AASHTO and Caltrans design provisions and ultimately resulted in the development of ATC-6, *Seismic Design Guidelines for Highway Bridges*, which was published in 1981. That document was subsequently adopted by AASHTO as a Guide Specification in 1983; the guidelines were formally adopted into the *Standard Specifications for Highway Bridges* in 1991, then revised and reformatted as Division I-A. Later, Division I-A became the basis for the seismic provisions included in the *AASHTO LRFD Bridge Design Specifications*.

After damaging earthquakes in 1980s and 1990s, and as more recent research efforts were completed, it became clear that improvements to the seismic design practice for bridges should be undertaken. Several efforts culminated in the publication of ATC-32, *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations* in 1996; the development of Caltrans' *Seismic Design Criteria*; publication of MCEER/ATC-49 (NCHRP 12-49), *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges* in 2003; and the development of the South Carolina *Seismic Design* Specifications in 2001. Thus in 2005, with the T-3 Seismic Design Technical Committee's support, work began to identify and consolidate the best practices from these four documents into a new seismic design specification for AASHTO. The resulting document was founded on displacement-based design principles, recommended a 1000-yr return period earthquake ground motion, and comprised a new set of guidelines for seismic design of bridges. During 2007, a technical review team refined the document into the Guide Specifications that were adopted at the 2007 annual AASHTO Highways Subcommittee on Bridges and Structures meeting. The following year, further refinement was completed by the team and was adopted. The 2007 document, combined with the modifications approved in 2008, form the basis of these Guide Specifications.

The scope of these Guide Specifications covers seismic design for typical bridge types and applies to noncritical and non-essential bridges. The title of the document reflects the fact that the Guide Specifications are approved as an alternate to the seismic provisions in the *AASHTO LRFD Bridge Design Specifications.* These Guide Specifications differ from the current procedures in the LRFD Specifications in the use of displacement-based design procedures, instead of the traditional, force-based "R-Factor" method. This new approach is split into a simplified implicit displacement check procedure and a more rigorous pushover assessment of displacement capacity. The selection of which procedure to use is based on seismic design categories, similar to the seismic zone approach used in the *AASHTO LRFD Bridge Design Specifications*. Also included is detailed guidance and commentary on earthquake-resisting elements and systems, global design strategies, demand modeling, capacity calculation, and liquefaction effects. Similar to the LRFD force-based method, capacity design procedures underpin the Guide Specifications' methodology, and these procedures include prescriptive detailing for plastic hinging regions and design requirements for capacity protection of those elements that should not experience damage.

These Guide Specifications incorporate recent experience, best practices, and research results and represent a significant improvement over the traditional force-based approach. It is expected that these Guide Specifications will be revised as refinements or improvements become available.

AASHTO Highways Subcommittee on Bridges and Structures

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This work was sponsored by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program (NCHRP), which is administered by the Transportation Research Board of the National Research Council. The first edition of any technical publication is especially labor intensive. AASHTO's Highways Subcommittee on Bridges and Structures gratefully acknowledges the contributions of the following people:

AASHTO Technical Committee for Seismic Design

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The technical review team:

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- Tony Allen, WSDOT
- Don Anderson, CH2M Hill

1000-yr Maps and Ground Motion CD Tool—Ed V. Leyendecker, USGS

PREFACE

This first edition of the *Guide Specifications for LRFD Seismic Bridge Design* includes technical content approved by the Highways Subcommittee on Bridges and Structures in 2007 and 2008.

An abbreviated table of contents follows this preface. Detailed tables of contents precede each Section and Appendix A.

The *AASHTO Guide Specifications for LRFD Seismic Bridge Design* includes a CD-ROM with many helpful search features that will be familiar to users of the *AASHTO LRFD Bridge Design Specifications* CD-ROM. Examples include:

- Bookmarks to all articles;
- Links within the text to cited articles, figures, tables, and equations;
- Links for current titles in reference lists to AASHTO's Bookstore; and
- The Acrobat search function.

AASHTO Publications Staff

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SECTION 1: INTRODUCTION

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SECTION 1:

INTRODUCTION

1.1—BACKGROUND

The state of practice of the seismic design of bridges is continually evolving, and the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* was developed to incorporate improvements in the practice that have emerged since publication of ATC 6, *Seismic Design Guidelines for Highway Bridges,* the basis of the current AASHTO seismic design provisions. While small improvements have been incorporated into the AASHTO seismic design procedures in the intervening years since ATC 6 was published in 1981, these Guide Specifications and related changes to the current *AASHTO LRFD Bridge Design Specifications* represent the first major overhaul of the AASHTO procedures. The development of these Guide Specifications was performed in accordance with the recommendations of the NCHRP 20-07/Task 193 Task 6 Report. The Task 6 effort combined and supplemented existing completed efforts (i.e., AASHTO Standard Specifications Division I-A, NCHRP 12-49 guidelines, SCDOT specifications, Caltrans *Seismic Design Criteria,* NYCDOT *Seismic Intensity Maps* (1998), and ATC-32) into a single document that could be used at a national level to design bridges for seismic effects. Based on the Task 6 effort and that of a number of reviewers, including representatives from State Departments of Transportation, the Federal Highway Administration, consulting engineers, and academic researchers, these Guide Specifications were developed.

Key features of these Guide Specifications follow.

- Adopt the seven percent in 75 yr design event for development of a design spectrum.
- Adopt the NEHRP Site Classification system and include site factors in determining response spectrum ordinates.
- Ensure sufficient conservatism (1.5 safety factor) for minimum support length requirement. This conservatism is needed to accommodate the full capacity of the plastic hinging mechanism of the bridge system.

C1.1

This commentary is included to provide additional information to clarify and explain the technical basis for the specifications provided in the *Guide Specifications for LRFD Seismic Bridge Design*. These specifications are for the design of new bridges.

The term "shall" denotes a requirement for compliance with these Specifications.

The term "should" indicates a strong preference for a given criterion.

The term "may" indicates a criterion that is usable, but other local and suitably documented, verified, and approved criterion may also be used in a manner consistent with the LRFD approach to bridge design.

The term "recommended" is used to give guidance based on past experiences. Seismic design is a developing field of engineering that has not been uniformly applied to all bridge types; thus, the experiences gained to date on only a particular type are included as recommendations.

- Establish four Seismic Design Categories (SDCs) with the following requirements:

  SDC A

  - No displacement capacity check needed
  - No capacity design required
  - SDC A minimum requirements
  - No liquefaction assessment required

  SDC B

  - Implicit displacement capacity check required (i.e., use a closed form solution formula)
  - Capacity checks suggested
  - SDC B level of detailing
  - Liquefaction assessment recommended for certain conditions

  SDC C

  - Implicit displacement capacity check required
  - Capacity design required
  - SDC C level of detailing
  - Liquefaction assessment required

  SDC D

  - Pushover analysis required
  - Capacity design required
  - SDC D level of detailing
  - Liquefaction assessment required

- Allow for three types of a bridge structural system:
  - *Type 1*—Design a ductile substructure with an essentially elastic superstructure.
  - *Type 2*—Design an essentially elastic substructure with a ductile superstructure.
  - *Type 3*—Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.

1.2—TECHNICAL ASSISTANCE AGREEMENT BETWEEN AASHTO AND USGS

Under the agreement, the U.S. Geological Survey (USGS) prepared two types of products for use by the American Association of State Highway and Transportation Officials (AASHTO). The first product was a set of paper maps of selected seismic design parameters for a seven percent probability of exceedance in 75 yr. The second product was a ground motion software tool to simplify determination of the seismic design parameters.

These guidelines use spectral response acceleration with a seven percent probability of exceedance in 75 yr as the basis of the seismic design requirements. As part of the National Earthquake Hazards Reduction Program, the USGS's National Seismic Hazards Mapping Project prepares seismic hazard maps of different ground motion parameters with different probabilities of exceedance. The maps used in these Guide Specifications were prepared by the USGS under a separate Technical Assistance Agreement with AASHTO, for use by AASHTO and, in particular, the Highways Subcommittee on Bridges and Structures.

1.2.1—Maps

The set of paper maps covered the 50 states of the United States and Puerto Rico. Some regional maps were also included to improve resolution of contours. Maps of the conterminous 48 states were based on USGS data used to prepare maps for a 2002 update. Alaska was based on USGS data used to prepare a map for a 2006 update. Hawaii was based on USGS data used to prepare 1998 maps. Puerto Rico was based on USGS data used to prepare 2003 maps.

The maps included in the package were prepared in consultation with the Subcommittee on Bridges and Structures. The package included a series of maps that provide:

- The peak horizontal ground acceleration coefficient, PGA,
- A short-period (0.2-sec) value of spectral acceleration coefficient, S_s, and
- A longer-period (1.0-sec) value of spectral acceleration coefficient, S_1.

The maps are for spectral accelerations for a reference Site Class B.

1.2.2—Ground Motion Tool

The ground motion software tool was packaged on a CD-ROM for installation on a PC using a Windows-based operating system. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude–longitude or zip code from the USGS data.
- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.

In addition to calculation of the basic parameters, the CD allows the user to obtain the following additional information for a specified site:

- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data, the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also includes the seven percent in 75-y maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.

1.3—FLOWCHARTS

It is envisioned that the flowcharts herein will provide the engineer with a simple reference to direct the design process needed for each of the four SDCs.

Flowcharts outlining the steps in the seismic design procedures implicit in these Guide Specifications are given in Figures 1a to 6.

The Guide Specifications were developed to allow three global seismic design strategies based on the characteristics of the bridge system, which include:

- *Type 1*—Design a ductile substructure with an essentially elastic superstructure.
- *Type 2*—Design an essentially elastic substructure with a ductile superstructure.
- *Type 3*—Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.

The flowchart in Figure 1a guides the designer on the applicability of the Guide Specifications and the breadth of the design procedure dealing with a single-span bridge versus a multispan bridge and a bridge in SDC A versus a bridge in SDC B, C, or D.

Figure 1b shows the core flowchart of procedures outlined for bridges in SDCs B, C, and D. Figure 2 outlines the demand analysis. Figure 3 directs the designer to determine displacement capacity. Figure 4 shows the modeling procedure. Figures 5a and 5b establish member detailing requirements based on the type of the structure chosen for seismic resistance. Figure 6 shows the foundation design.

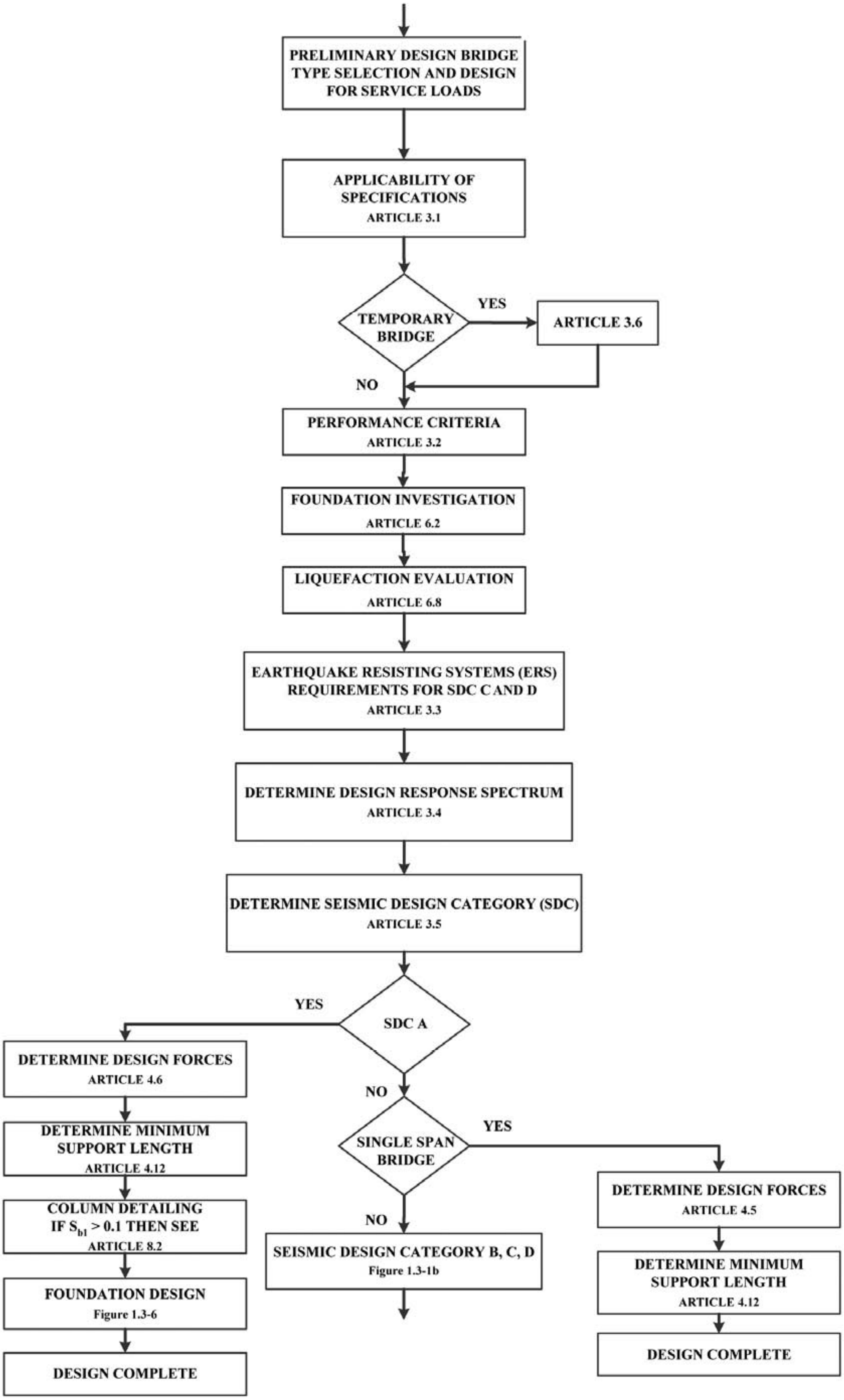

Figure 1.3-1a—Seismic Design Procedure Flowchart

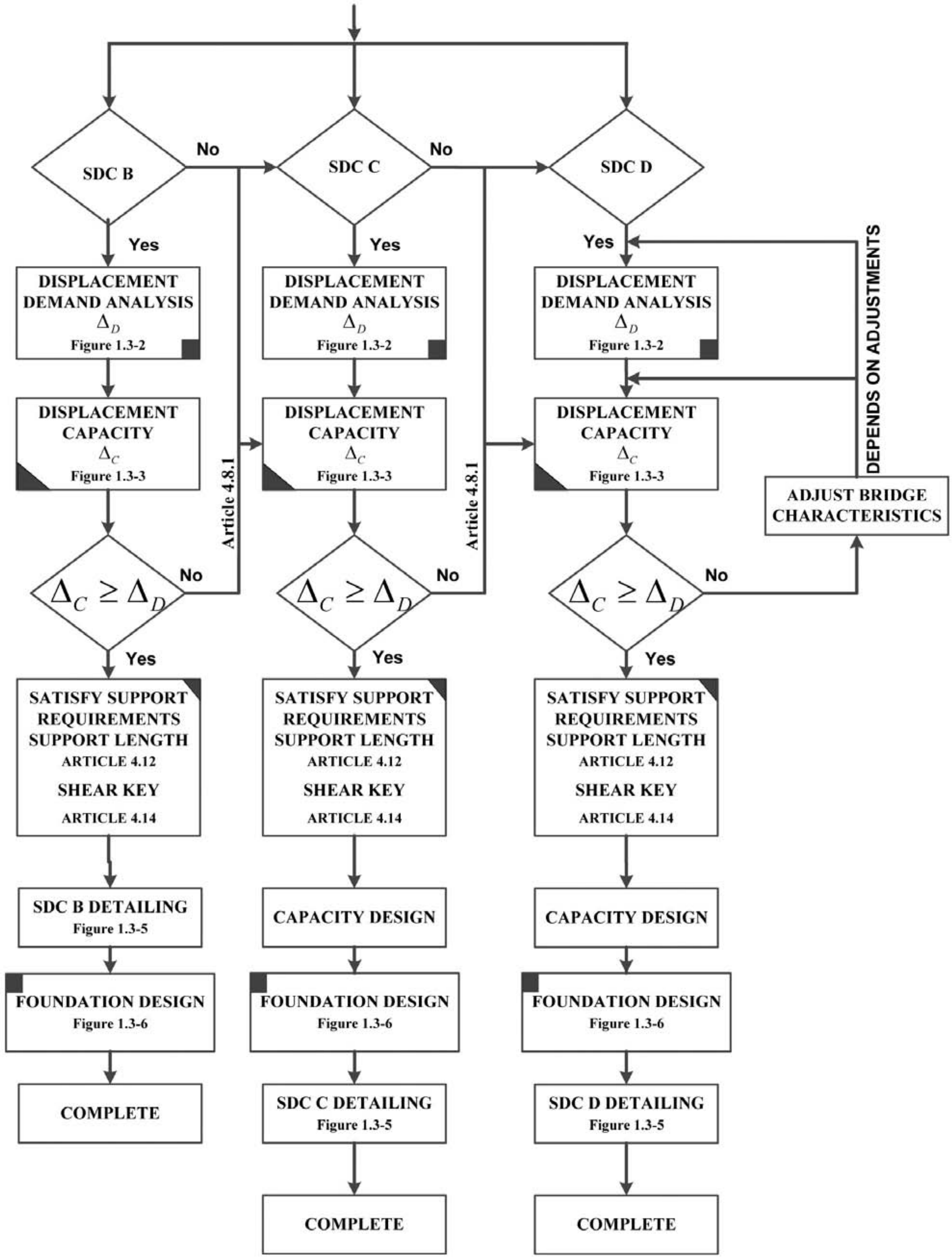

Figure 1.3-1b—Seismic Design Procedure Flowchart

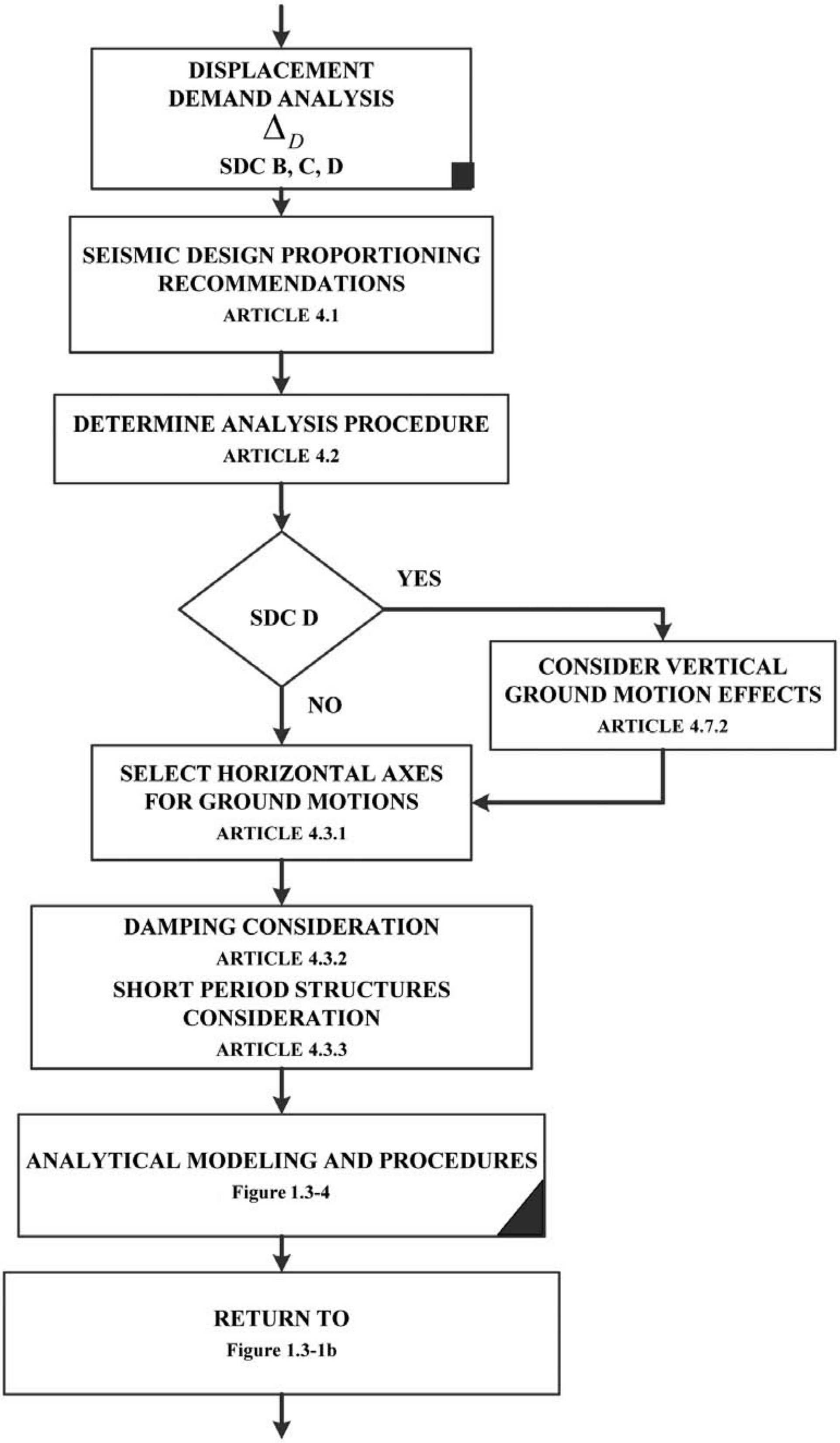

Figure 1.3-2—Demand Analysis Flowchart

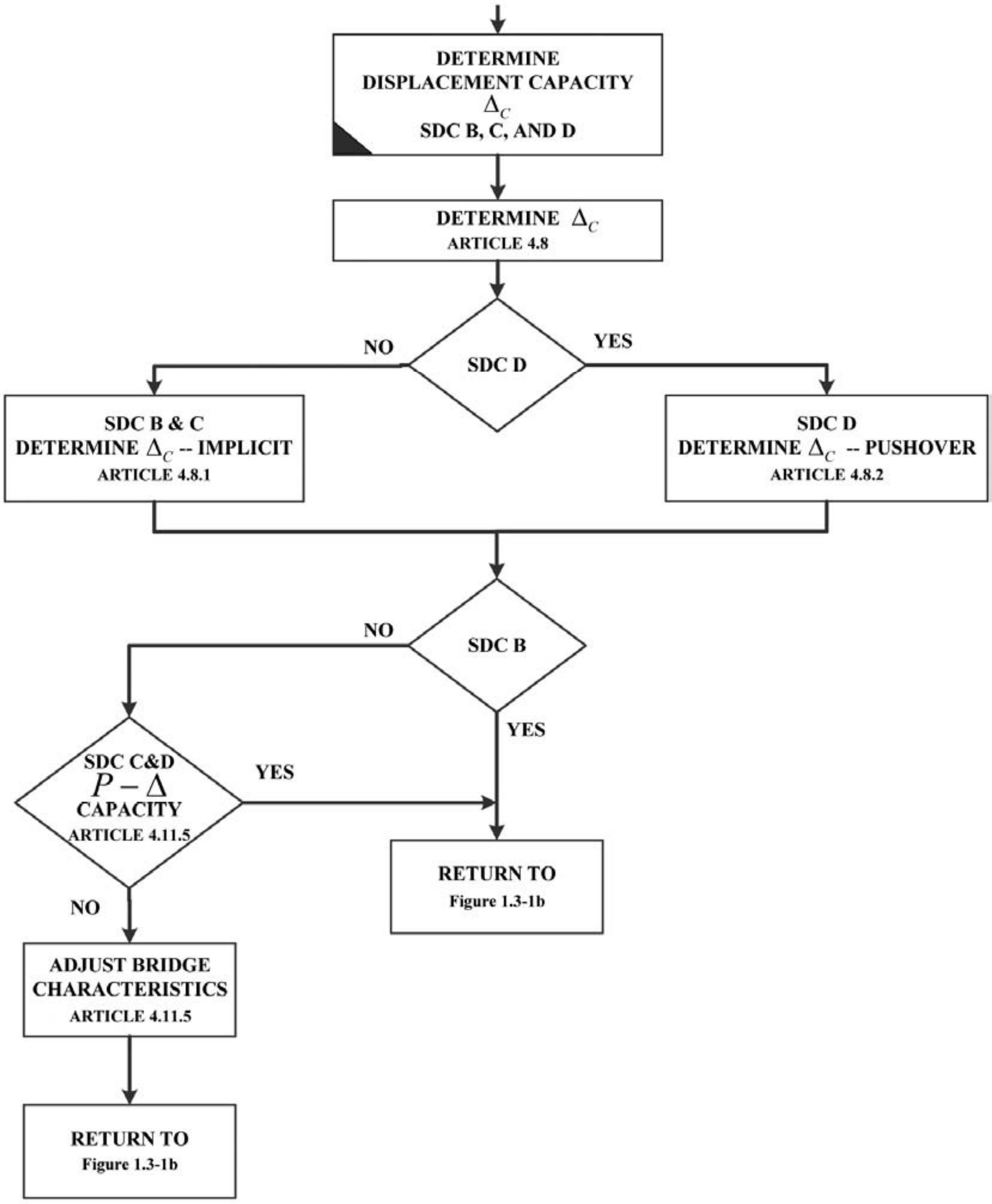

Figure 1.3-3—Displacement Capacity Flowchart

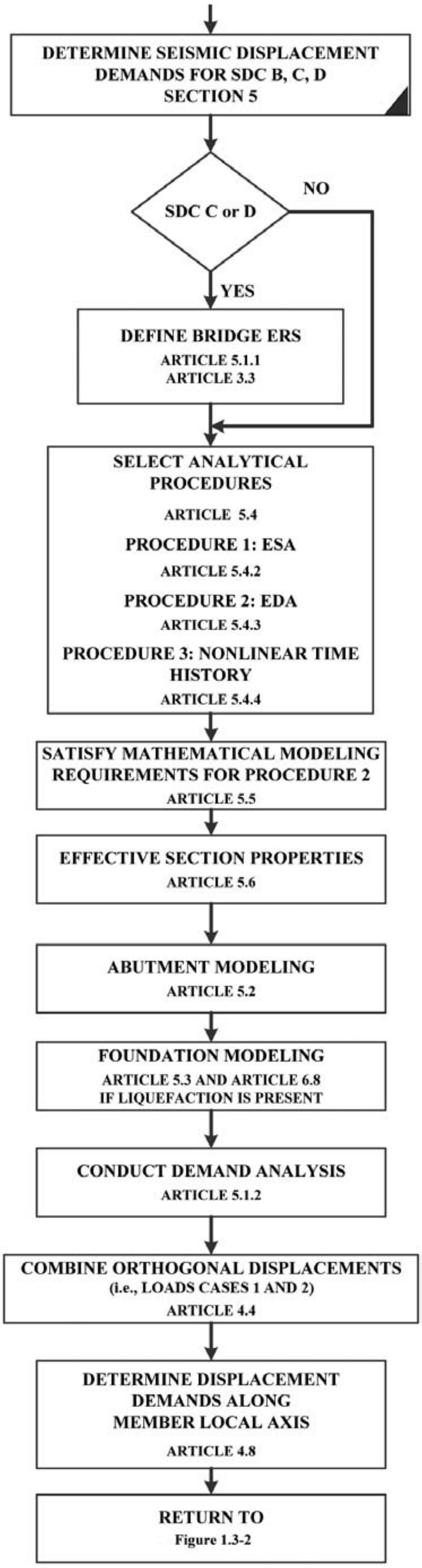

Figure 1.3-4—Modeling Procedure Flowchart

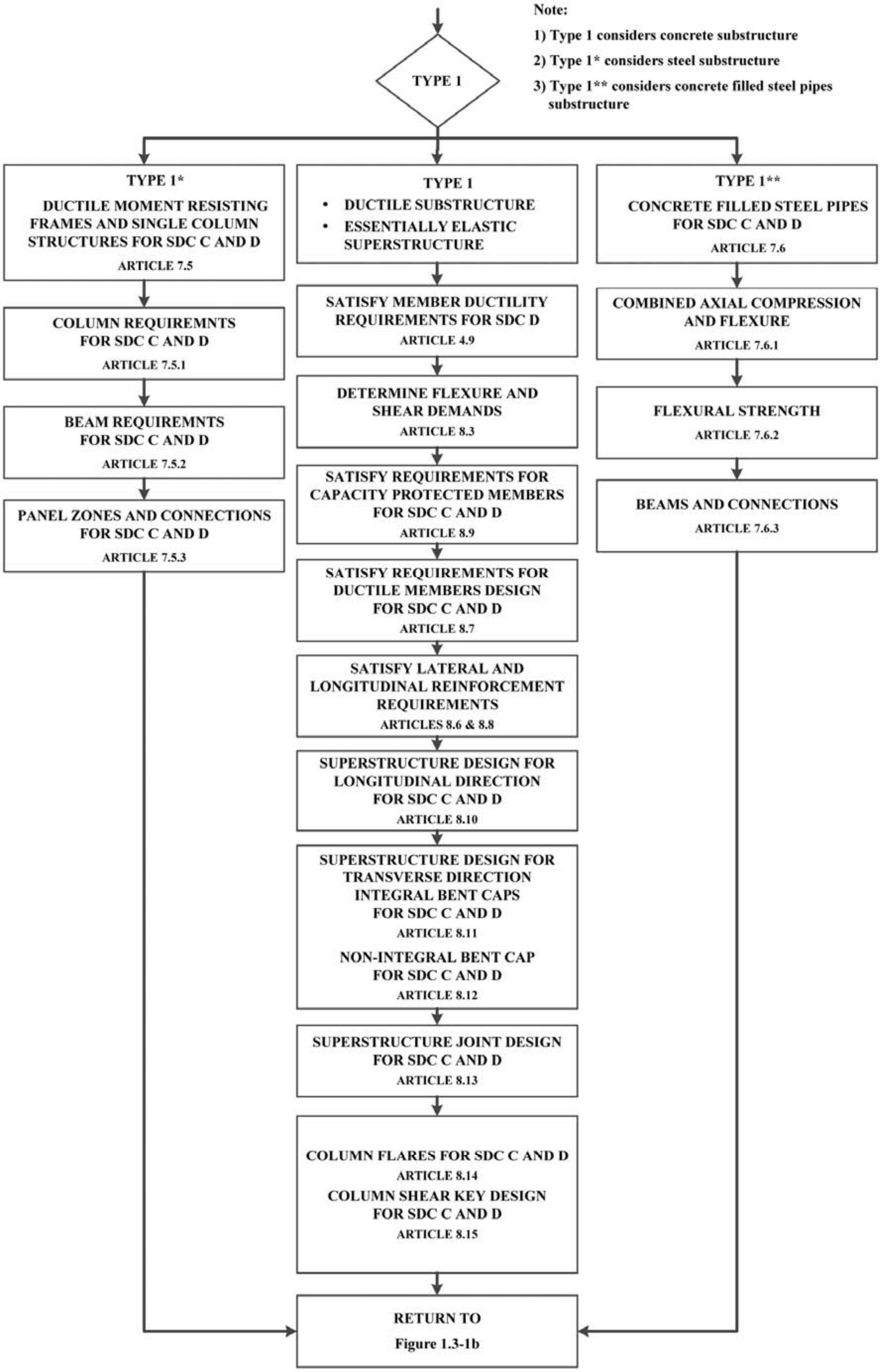

Figure 1.3-5a—Detailing Procedure Flowchart

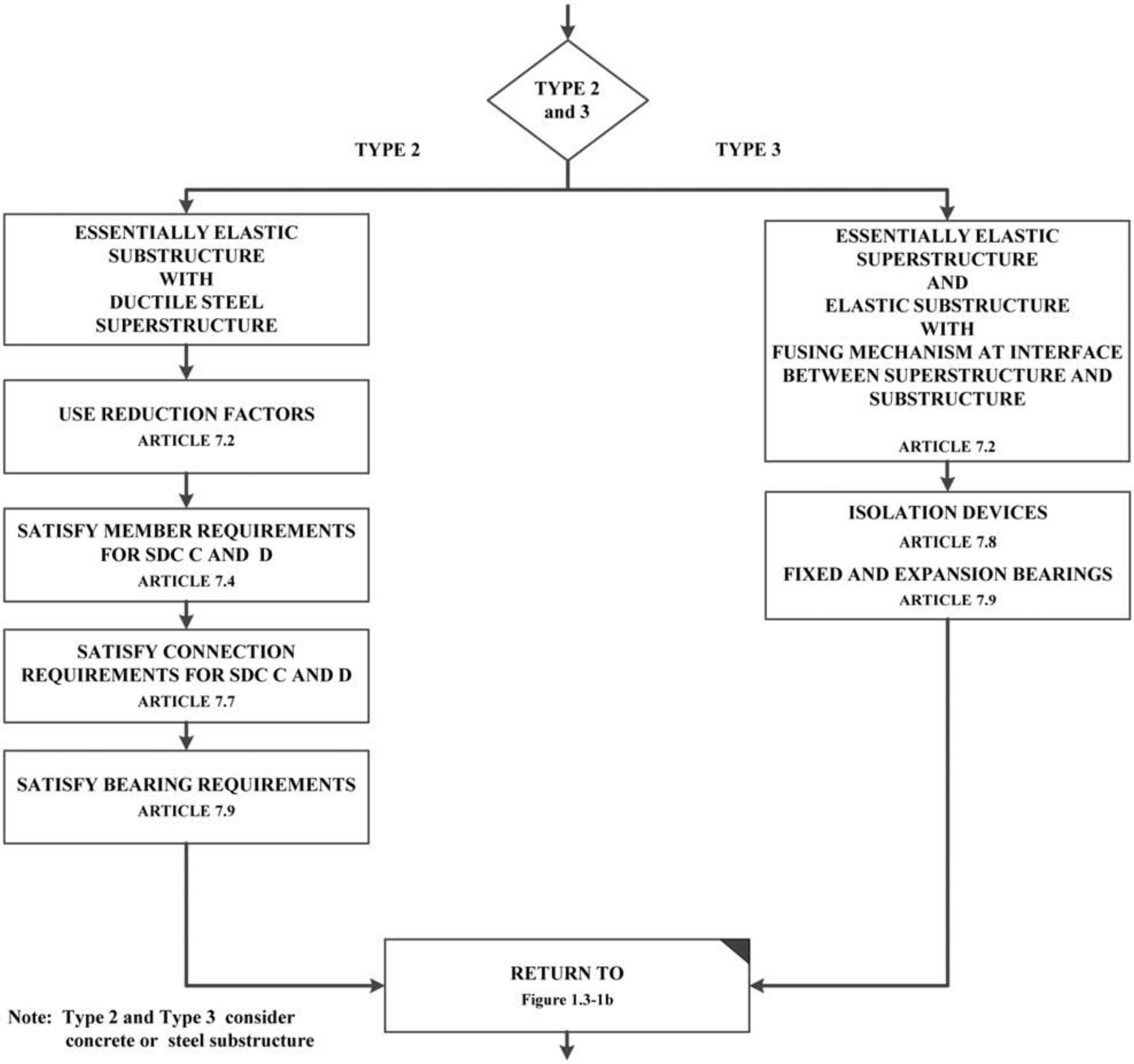

Figure 1.3-5b—Detailing Procedure Flowchart

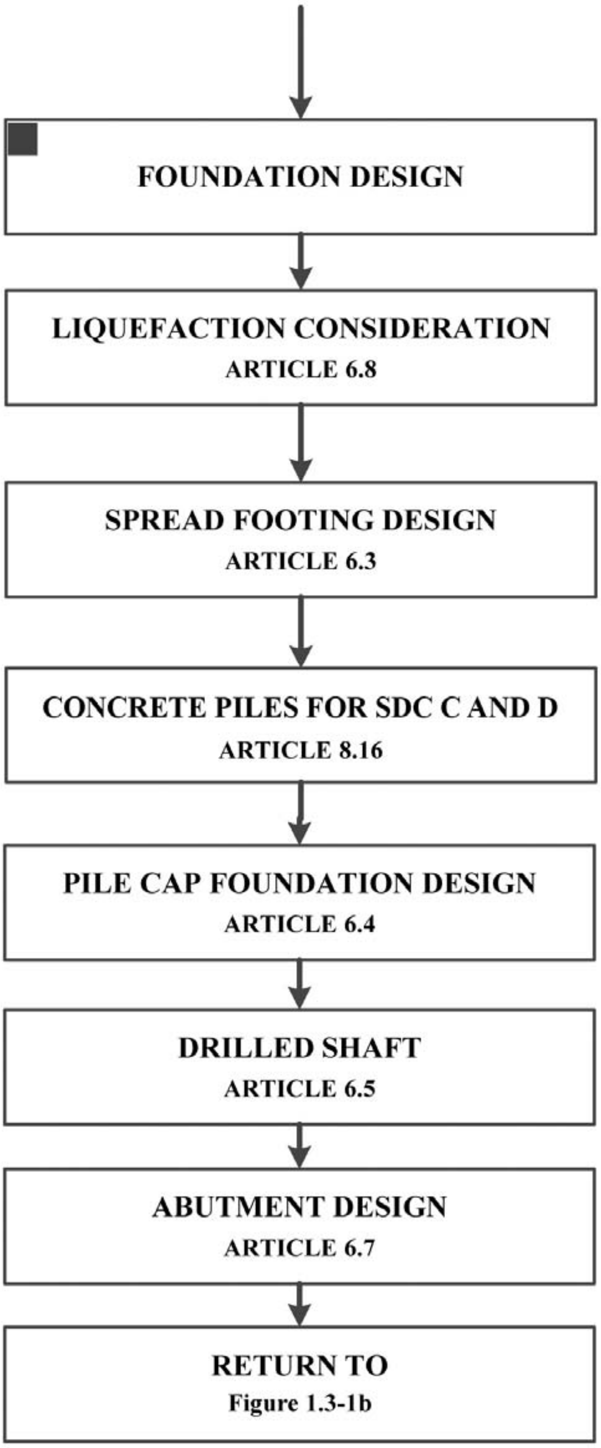

Figure 1.3-6—Foundation Design Flowchart

SECTION 2: DEFINITIONS AND NOTATION

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SECTION 2:

DEFINITIONS AND NOTATION

2.1—DEFINITIONS

Capacity Checks—Capacity design checks made with the overstrength magnifiers set to 1.0. The expected strengths of materials are included. Capacity checks are permitted in lieu of full capacity design for SDC B.

Capacity Design—A method of component design that allows the designer to prevent damage in certain components by making them strong enough to resist loads that are generated when adjacent components reach their overstrength capacity.

Capacity-Protected Element—Part of the structure that is either connected to a critical element or within its load path and that is prevented from yielding by virtue of having the critical member limit the maximum force that can be transmitted to the capacity-protected element.

Collateral Seismic Hazard—Seismic hazards other than direct ground shaking, such as liquefaction, fault rupture, etc.

Complete Quadratic Combination (CQC)—A statistical rule for combining modal responses from an earthquake load applied in a single direction to obtain the maximum response due to this earthquake load.

Critical or Ductile Elements—Parts of the structure that are expected to absorb energy and undergo significant inelastic deformations while maintaining their strength and stability.

Damage Level—A measure of seismic performance based on the amount of damage expected after one of the design earthquakes.

Displacement Capacity Verification—A design and analysis procedure that requires the designer to verify that his or her structure has sufficient displacement capacity. It generally involves the Nonlinear Static Procedure (NSP), also commonly referred to as “pushover” analysis.

Ductile Substructure Elements—See Critical or Ductile Elements.

Earthquake-Resisting Element (ERE)—The individual components, such as columns, connections, bearings, joints, foundation, and abutments, that together constitute the earthquake-resisting system (ERS).

Earthquake-Resisting System (ERS)—A system that provides a reliable and uninterrupted load path for transmitting seismically induced forces into the ground and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements.

Life Safety Performance Level—The minimum acceptable level of seismic performance allowed by this Guide Specification; intended to protect human life during and following a rare earthquake.

Liquefaction—Seismically induced loss of shear strength in loose, cohesionless soil that results from a buildup of pore water pressure as the soil tries to consolidate when exposed to seismic vibrations.

Liquefaction-Induced Lateral Flow—Lateral displacement of relatively flat slopes that occurs under the combination of gravity load and excess pore water pressure (without inertial loading from earthquake); often occurs after the cessation of earthquake loading.

Liquefaction-Induced Lateral Spreading—Incremental displacement of a slope that occurs from the combined effects of pore water pressure buildup, inertial loads from the earthquake, and gravity loads.

Local—Descriptor used to denote direction, displacement, and other response quantities for individual substructure locations. Seismic analysis is performed “globally” on the entire structure, while evaluations are typically performed at the local level.

Minimum Support Width—The minimum prescribed length of a bearing seat that is required to be provided in a new bridge designed according to this Guide Specification.

Nominal Resistance—Resistance of a member, connection, or structure based on the expected yield strength (F_{ye}) or other specified material properties, and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0.

Operational Performance Level—A higher level of seismic performance that may be selected by a bridge owner who wishes to have immediate service and minimal damage following a rare earthquake.

Overstrength Capacity—The maximum expected force or moment that can be developed in a yielding structural element assuming overstrength material properties and large strains and associated stresses.

Performance Criteria—The levels of performance in terms of post-earthquake service and damage that are expected to result from specified earthquake loadings if bridges are designed according to this specification.

Plastic Hinge—The region of a structural component, usually a column or a pier in bridge structures, that undergoes flexural yielding and plastic rotation while still retaining sufficient flexural strength.

Plastic Hinge Zone—Those regions of structural components that are subject to potential plastification and thus shall be detailed accordingly.

Pushover Analysis—See Displacement Capacity Verification.

Response Modification Factor (R Factor)—Factors used to modify the element demands from an elastic analysis to account for ductile behavior and obtain design demands.

Seismic Design Category (SDC)—One of four seismic design categories (SDCs), A through D, based on the 1-sec period design spectral acceleration for the life safety design earthquake.

Service Level—A measure of seismic performance based on the expected level of service that the bridge is capable of providing after one of the design earthquakes.

Site Class—One of six classifications used to characterize the effect of the soil conditions at a site on ground motion.

Tributary Weight—The portion of the weight of the superstructure that would act on a pier participating in the ERS if the superstructure between participating piers consisted of simply supported spans. A portion of the weight of the pier itself may also be included in the tributary weight.

2.2—NOTATION

The following symbols and definitions apply to these Guide Specifications:

A = cross-sectional area of member (in.^{2}) (7.5.2)

A_c = area of the concrete core (in.^{2}) (C7.6) (7.6.1) (7.6.2)

A_{cap}^{bot} = area of bent cap bottom flexural steel (in.^{2}) (8.13.4.2.3)

A_{cap}^{top} = area of bent cap top flexural steel (in.^{2}) (8.13.4.2.3)

A_e = effective area of the cross-section for shear resistance (in.^{2}) (8.6.2) (8.6.4) (8.6.9)

A_{ew} = cross-sectional area of pier wall (in.^{2}) (5.6.2)

A_g = gross area of section along the plane resisting tension (in.^{2}); gross area of member cross-section (in.^{2}) (7.7.6) (8.6.2) (8.7.2) (8.8.1) (8.8.2)

A_{gg} = gross area of gusset plate (in.^{2}) (7.7.9)

A_{jh} = effective horizontal joint area (in.) (8.13.2)

A_{jh}^{ftg} = effective horizontal area at mid-depth of the footing assuming a 0.78-rad spread away from the boundary of the column in all directions as shown in Figure 6.4.5-1 (in.^{2}) (6.4.5)

A_{jv} = effective vertical joint area (in.) (8.13.2)

A_ℓ = area of longitudinal reinforcement in member (in.^{2}) (8.8.1) (8.8.2)

A_n = net area of section along the plane resisting tension (in.^{2}) (7.7.6)

A_s = area of the steel pipe (in.^{2}); effective peak ground acceleration coefficient (3.4.1) (4.5) (4.12.1) (5.2.4.1) (6.7.1) (C7.6)

A_s^{j-bar} = area of vertical j-dowels hooked around the longitudinal top deck steel required at moment resisting joints for integral cap of bent with a skew angle >0.34 rad (in.^{2}) (8.13.4.2.4)

A_s^{jh} = cross-sectional area of horizontal stirrups required at moment resisting joints (in.^{2}) (8.13.4.2.2)
A_s^{jl} = cross-sectional area of required additional longitudinal cap beam reinforcement (in.^{2}) (8.13.5.1.3)
A_s^{jv} = cross-sectional area of vertical stirrups required at moment resisting joints (in.^{2}) (8.13.4.2.1)
A_s^{jvi} = cross-sectional area of vertical stirrup required inside the joint region (in.^{2}) (8.13.5.1.2)
A_s^{jvo} = cross-sectional area of vertical stirrup required outside the joint region (in.^{2}) (8.13.5.1.1)
A_s^{sf} = total longitudinal (horizontal) side face reinforcement in the bent cap required at moment resisting joints (in.^{2}) (8.13.4.2.3)
A_{sp} = area of spiral or hoop reinforcement (in.^{2}) (8.6.2) (8.6.3)
A_{st} = total area of column reinforcement anchored in the joint (in.^{2}) (8.13.3) (8.13.4.2.1) (8.13.4.2.2) (8.13.4.2.4) (8.13.5.1.1) (8.13.5.1.2) (8.13.5.1.3)
A_{sur} = surface area of the side of a pile cap on which frictional force acts (kips) (C6.4.3)
A_{tg} = gross area of section along the plane resisting tension in block shear failure mode (in.^{2}) (7.7.6)
A_{tn} = net area of section along the plane resisting tension in block shear failure mode (in.^{2}) (7.7.6)
A_v = cross-sectional area of shear reinforcement in the direction of loading (in.^{2}) (8.6.2) (8.6.3) (8.6.9)
A_{vg} = gross area of section along the plane resisting shear in block shear failure mode (in.^{2}) (7.7.6)
A_{vn} = net area of section along the plane resisting shear in block shear failure mode (in.^{2}) (7.7.6)
a = depth of soil stress block beneath footing at maximum rocking (ft) (A.1)
B = width of footing measured normal to the direction of loading (ft) (6.3.4) (6.3.6)
B_c = diameter or width of column or wall measured normal to the direction of loading (in.) (6.3.6) (6.4.5)
B_{cap} = thickness of the bent cap (in.) (8.11) (8.13.2)
B_{eff} = effective width of the superstructure or bent cap for resisting longitudinal seismic moment (in.) (8.10) (8.11)
B_{eff}^{ftg} = effective width of footing (in.) (6.4.5)
B_o = column diameter or width measured parallel to the direction of displacement under consideration (ft) (4.8.1)
B_r = footing width (ft) (A.1)
b = width of unstiffened or stiffened element (in.); width of column or wall in direction of bending (in.) (7.4.2) (8.6.2) (8.6.9)
b_{eff} = effective width of the footing used to calculate the nominal moment capacity of the footing (ft) (6.3.6)
b/t = width–thickness ratio of unstiffened or stiffened element (7.4.2)
$C_{(i)}^{pile}$ = compression force in "ith" pile (kip) (6.4.2)
c = soil cohesion (psf of ksf) (6.2.2)
$c_{x(i)}$ = distance from neutral axis of pile group to "ith" row of piles measured parallel to the y axis (ft) (6.4.2)
$c_{y(i)}$ = distance from neutral axis of pile group to "ith" row of piles measured parallel to the x axis (ft) (6.4.2)
D = distance from active fault (mi); diameter of concrete filled pipe (in.); diameter of HSS tube (in.); outside diameter of steel pipe (in.); diameter of column or pile (in.) (3.4.3.1) (3.4.4) (4.11.6) (7.4.2) (7.6.2) (8.16.1)
D' = diameter of spiral or hoop (in.) (8.6.2) (8.6.3)
D/t = diameter-to-thickness ratio of a steel pipe (7.4.2) (C7.6.1)
D^* = diameter of circular shafts or cross-section dimension in direction under consideration for oblong shafts (in.) (4.11.6)
D_c = diameter or depth of column in direction of loading (ft or in.) (6.3.2) (C6.3.6) (8.8.6) (8.10) (8.13.2) (8.13.4.2.4) (8.13.5)
D_{cj} = column width or diameter parallel to the direction of bending (in.) (6.4.5)
$D_{c,max}$ = larger cross-section dimension of the column (in.) (8.8.10) (8.8.13)
D_{ftg} = depth of the pile cap or footing (ft or in.) (6.4.2) (6.4.5)
D_g = width of gap between backwall and superstructure (ft) (5.2.3.3)
D_s = depth of superstructure at the bent cap (in.) (8.7.1) (8.10) (8.13.2)

d = depth of superstructure or cap beam (in.); overall depth of section (in.); depth of section in direction of loading (in.) (4.11.2) (8.13.5) (7.4.2) (8.6.3) (8.6.9)

$d_{b\ell}$ = nominal diameter of longitudinal column reinforcing steel bars (in.) (4.11.6) (8.8.4) (8.8.6)

d_i = thickness of "ith" soil layer (ft) (3.4.2.2)

E = modulus of elasticity of steel (ksi) (7.4.2) (7.7.5)

E_c = modulus of elasticity of concrete (ksi) (5.6.2) (C7.6)

E_cI_{eff} = effective flexural stiffness (kip-in.^{2}) (5.6.1) (5.6.2)

E_s = modulus of elasticity of steel (ksi) (C7.6) (8.4.2)

F_a = site coefficient for 0.2-sec period spectral acceleration (3.4.1) (3.4.2.3)

F_{pga} = site coefficient for the peak ground acceleration coefficient (3.4.1) (3.4.2.3)

F_s = shear force along pile cap (kip) (C6.4.3)

F_u = minimum tensile strength of steel (ksi) (7.7.6)

F_v = site coefficient for 1.0-sec period spectral acceleration (3.4.1) (3.4.2.3)

F_w = factor taken as between 0.01 and 0.05 for soils ranging from dense sand to compacted clays (5.2.3.3)

F_y = specified minimum yield strength of steel (ksi); nominal yield stress of steel pipe or steel gusset plate (ksi) (7.3) (7.4.1) (7.4.2) (7.7.6) (7.6.2) (7.7.5) (7.7.8) (7.7.9)

F_{ye} = expected yield stress of structural steel member (ksi) (7.3) (7.5.2)

f'_c = nominal uniaxial compressive concrete strength (ksi) (6.4.5) (7.6.1) (7.6.2) (8.4.4) (8.6.2) (8.6.4) (8.6.9) (8.7.2) (8.8.4) (8.8.6) (C8.13.2) (8.13.3)

f'_{cc} = confined compressive strength of concrete (ksi) (8.4.4)

f'_{ce} = expected concrete compressive strength (ksi) (8.4.4) (C8.13.2)

f_h = average normal stress in the horizontal direction within a moment resisting joint (ksi) (8.13.2)

f_{ps} = stress in prestressing steel corresponding to strain ε_{ps} (ksi) (8.4.3)

f_{ue} = expected tensile strength (ksi) (8.4.2)

f_v = average normal stress in the vertical direction within a moment resisting joint (ksi) (6.4.5) (8.13.2)

f_y = specified minimum yield stress (ksi) (8.4.2)

f_{ye} = expected yield strength (ksi) (4.11.6) (8.4.2) (8.8.4) (8.8.6) (8.11)

f_{yh} = yield stress of spiral, hoop, or tie reinforcement (ksi) (8.6.2) (8.6.3) (8.6.9) (8.8.8) (8.13.3)

G = soil dynamic (secant) shear modulus (ksi) (C5.3.2)

$(GA)_{eff}$ = effective shear stiffness parameter of the pier wall (kip) (5.6.1) (5.6.2)

G_c = shear modulus of concrete (ksi) (5.6.2)

G_cJ_{eff} = torsional stiffness (5.6.1)

G_f = gap between the isolated flare and the soffit of the bent cap (in.) (4.11.6)

G_{max} = soil low-strain (initial) shear modulus (C5.3.2)

g = acceleration due to gravity (ft/sec^2 or in./sec^2) (C5.4.2)

H = thickness of soil layer (ft); column height used to calculate minimum support length (in.) (3.4.2.1) (4.12.1)

H_f = depth of footing (ft) (6.3.2) (6.3.4) (6.3.6)

H_h = the height from the top of the footing to the top of the column or the equivalent column height for a pile extension column (ft) (8.7.1)

H_r = height of rocking column (ft) (A.1)

H_o = clear height of column (ft) (4.8.1)

H_w = height of backwall or diaphragm (ft) (5.2.3.3)

H' = length of shaft from the ground surface to point of contraflexure above ground (in.); length of pile from point of the ground surface to point of contraflexure above ground (in.) (4.11.6)

h = web depth (in.); distance from c.g. of tensile force to c.g. of compressive force on the section (in.) (7.4.2) (8.13.2)

h/t_w = web depth–thickness ratio (7.4.2)

I_c = moment of inertia of the concrete core (in.^{4}) (C7.6)

I_{eff} = effective moment of inertia of the section based on cracked concrete and first yield of the reinforcing steel (in.^{4}); effective moment of inertia of the section based on cracked concrete and first yield of the reinforcing steel or effective moment of inertia taken about the weak axis of the reinforced concrete cross-section (in.^{4}) (5.6.1) (5.6.2) (5.6.3) (5.6.4)

I_g = gross moment of inertia taken about the weak axis of the reinforced concrete cross-section (in.^{4}) (5.6.2) (5.6.3) (5.6.4)

$I_{pg(x)}$ = effective moment of inertia of pile group about the x axis (pile-ft^2) (6.4.2)

$I_{pg(y)}$ = effective moment of inertia of pile group about the y axis (pile-ft^2) (6.4.2)

I_s = moment of inertia of a single longitudinal stiffener about an axis parallel to the flange and taken at the base of the stiffener (in.^{4}); moment of inertia of the steel pipe (in.^{4}) (7.4.2) (C7.6)

J_{eff} = effective torsional (polar) moment of inertia of reinforced concrete section (in.^{4}) (5.6.1) (5.6.5)

J_g = gross torsional (polar) moment of inertia of reinforced concrete section (in.^{4}) (5.6.5)

K = effective lateral bridge stiffness (kip/ft or kip/in.); effective length factor of a member (C5.4.2) (7.4.1)

K_{DED} = stiffness of the ductile end diaphragm (kip/in.) (7.4.6)

K_{eff} = abutment equivalent linear secant stiffness (kip/ft) (5.2.3.3)

K_{eff1} = abutment initial effective stiffness (kip/ft) (5.2.3.3)

K_{eff2} = abutment softened effective stiffness (kip/ft) (5.2.3.3)

K_i = initial abutment backwall stiffness (kip/ft) (5.2.3.3)

KL/r = slenderness ratio (7.4.1)

K_{SUB} = stiffness of the substructure (kip/in.) (7.4.6)

K_0 = soil lateral stress factor (C6.4.3)

k = total number of cohesive soil layers in the upper 100 ft of the site profile below the bridge foundation; plate buckling coefficient for uniform normal stress (3.4.2.2) (7.4.2)

k_i^e = smaller effective bent or column stiffness (kip/in.) (4.1.1)

k_j^e = larger effective bent or column stiffness (kip/in.) (4.1.1)

L = length of column from point of maximum moment to the point of moment contraflexure (in.); length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck (ft); for hinges within a span, L shall be the sum of the distances to either side of the hinge (ft); for single-span bridges, L equals the length of the bridge deck (ft); total length of bridge (ft or in.); length of footing measured in the direction of loading (ft); unsupported length of a member (in.) (4.11.6) (8.8.6) (4.12.1) (C5.4.2) (6.3.2) (6.3.4) (C6.3.6) (7.4.1)

L_c = column clear height used to determine shear demand (in.) (4.11.2)

L_f = footing length (ft) (A.1)

L_{ftg} = cantilever overhang length measured from the face of wall or column to the outside edge of the pile cap or footing (ft) (6.4.2)

L_g = unsupported edge length of the gusset plate (in.) (7.7.5)

L_p = equivalent analytical plastic hinge length (in.) (4.11.6) (4.11.7)

L_{pr} = plastic hinge region that defines the portion of the column, pier, or shaft that requires enhanced lateral confinement (in.) (4.11.7)

L_u = unsupported length (in.) (C7.4.1)

ℓ_{ac} = length of column reinforcement embedded into the bent cap or footing (in.) (8.8.4) (8.13.2) (8.13.3)

M_r = restoring moment for rocking system (kip-ft) (A.1)

M_g = moment acting on the gusset plate (kip-in.) (7.7.10)

M_n = nominal moment capacity (kip-in or kip-ft) (4.11.2) (4.11.5) (6.3.6)

M_{ne}	=	nominal moment capacity of a reinforced concrete member based on expected materials properties and a concrete strain $\varepsilon_c = 0.003$ (kip-ft) (8.5) (8.7.1) (8.9)
M_{ng}	=	nominal moment strength of a gusset plate (kip-in.) (7.7.8)
M_{ns}	=	nominal flexural moment strength of a member (kip-in.) (7.4.1)
M_{nx}	=	probable flexural resistance of column (kip-ft) (7.5.2)
M_p	=	idealized plastic moment capacity of reinforced concrete member based on expected material properties (kip-in. or kip-ft) (4.11.2) (4.11.5) (8.5)
$M_{p(x)}^{col}$	=	the component of the column plastic hinging moment capacity about the *x* axis (kip-ft) (6.4.2)
$M_{p(y)}^{col}$	=	the component of the column plastic hinging moment capacity about the *y* axis (kip-ft) (6.4.2)
M_{po}	=	overstrength plastic moment capacity of the column (kip-in. or kip-ft) (4.11.2) (6.3.4) (8.5) (8.9) (8.10) (8.13.1) (8.13.2) (8.15)
M_{pg}	=	nominal plastic moment strength of a gusset plate (kip-in.) (7.7.8)
M_{px}	=	plastic moment capacity of the member based on expected material properties (kip-ft) (7.5.2)
M_{rc}	=	factored nominal moment capacity of member (kip-ft) (7.6.1)
$\mathrm{M_{rg}}$	=	factored nominal yield moment capacity of the gusset plate (kip-in.) (7.7.10)
M_{rpg}	=	factored nominal plastic moment capacity of the gusset plate (kip-in.) (7.7.10)
M_u	=	factored ultimate moment demand (kip-ft or kip-in.); factored moment demand acting on the member including the elastic seismic demand divided by the appropriate force-reduction factor, *R* (kip-ft) (6.3.6) (7.4.1) (7.6.1)
M_y	=	moment capacity of section at first yield of the reinforcing steel (kip-in.) (5.6.2)
m	=	total number of cohesionless soil layers in the upper 100 ft of the site profile below the bridge foundation (3.4.2.2)
m_i	=	tributary mass of column or bent *i* (kip) (4.1.1)
m_j	=	tributary mass of column or bent *j* (kip) (4.1.1)
N	=	minimum support length measured normal to the centerline of bearing (in.) (4.12) (4.12.1) (4.12.2)
$\overline{N}$	=	average standard penetration resistance for the top 100 ft (blows/ft) (3.4.2)
$\overline{N}_{ch}$	=	average standard penetration resistance of cohesionless soil layers for the top 100 ft (blows/ft) (3.4.2)
N_i	=	standard penetration resistance as measured directly in the field, uncorrected blow count, of "*i*th" soil layer not to exceed 100 ft (blows/ft) (3.4.2.2)
N_p	=	total number of piles in the pile group (pile) (6.4.2)
$(N_1)_{60}$	=	corrected standard penetration test (SPT) blow count (blows per foot) (6.8)
n	=	total number of distinctive soil layers in the upper 100 ft of the site profile below the bridge foundation; number of equally spaced longitudinal compression flange stiffeners; modular ratio; number of individual interlocking spiral or hoop core sections (3.4.2.2) (7.4.2) (C7.6) (8.6.3)
n_x	=	number of piles in a single row parallel to the *y* axis (pile) (6.4.2)
n_y	=	number of piles in a single row parallel to the *x* axis (pile) (6.4.2)
P_b	=	beam axial force at the center of the joint including prestressing (kip) (8.13.2)
P_{bs}	=	tensile strength of a gusset plate based on block shear (kip) (7.7.6)
P_c	=	column axial force including the effects of overturning (kip) (8.13.2)
P_{col}	=	column axial force including the effects of overturning (kip) (6.4.5)
P_{dl}	=	unfactored dead load acting on column (kip) (4.11.5)
P_g	=	axial force acting on the gusset plate (kip) (7.7.10)
PGA	=	peak horizontal ground acceleration coefficient on Class B rock (3.4.1) (4.5) (4.12.1) (5.2.4.1) (6.7.1)
PI	=	plasticity index of soil (3.4.2.1)
P_n	=	nominal axial strength of a member (kip) (7.4.1)
P_{ng}	=	nominal compression strength of the gusset plates (kip) (7.7.7)
P_p	=	abutment passive lateral earth capacity (kip) (5.2.3.3)
P_r	=	factored nominal axial capacity of member (kip) (7.6.1)

P_{rg}	=	factored nominal yield axial capacity of the gusset plate (kip) (7.7.10)
P_{ro}	=	factored nominal axial capacity of member (kip) (7.6.1)
P_{trib}	=	greater of the dead load per column or force associated with the tributary seismic mass collected at the bent (kip) (8.7.1)
P_u	=	axial force in column including the axial force associated with overstrength plastic hinging (kip); factored axial compressive load acting on the member (kip); factored axial load acting on the member (kip); ultimate compressive force acting on the section (kip); ultimate compressive force acting on the section including seismically induced vertical demands (kip) (6.3.4) (C6.3.6) (7.4.1) (7.4.2) (7.5.2) (8.6.2) (8.7.2)
P_y	=	nominal axial yield strength of a member (kip) (7.4.2)
p_c	=	principal compressive stress (ksi) (6.4.5) (8.13.2)
p_e	=	equivalent uniform static lateral seismic load per unit length of bridge applied to represent the primary mode of vibration (kip/ft or kip/in.) (C5.4.2)
p_p	=	passive lateral earth pressure behind backwall (ksf) (5.2.3.3)
p_o	=	uniform lateral load applied over the length of the structure (kip/ft or kip/in.) (C5.4.2)
p_t	=	principal tensile stress (ksi) (6.4.5) (8.13.2)
q_{ciN}	=	corrected cone penetration test (CPT) tip resistance (6.8)
q_n	=	nominal bearing capacity of supporting soil or rock (ksf) (6.3.4) (A.1)
R	=	maximum expected displacement ductility of the structure; response modification factor (4.3.3) (7.2) (7.2.2) (7.4.6)
R_D	=	damping reduction factor to account for increased damping (4.3.2)
R_d	=	magnification factor to account for short-period structure (4.3.3)
R_n	=	nominal resistance against sliding failure (6.3.5)
r	=	radius of gyration (in.) (7.4.1)
r_y	=	radius of gyration about minor axis (in.) (7.4.1)
S	=	angle of skew of support measured from a line normal to span (°) (4.12.1) (4.12.2)
S_a	=	design response spectral acceleration coefficient (3.4.1) (C5.4.2) (A.1)
S_1	=	1.0-sec period spectral acceleration coefficient on Class B rock (3.4.1)
S_{D1}	=	design earthquake response spectral acceleration coefficient at 1.0-sec period (3.4.1) (3.5)
S_{DS}	=	design earthquake response spectral acceleration coefficient at 0.2-sec period (3.4.1)
S_s	=	0.2-sec period spectral acceleration coefficient on Class B rock (3.4.1)
S_g	=	elastic section modulus of gusset plate about the strong axis (in.^{3}) (7.7.8)
s	=	spacing of spiral, hoop, or tie reinforcement (in.) (8.6.2) (8.6.3) (8.6.9)
$\overline{s}_u$	=	average undrained shear strength in the top 100 ft (psf) (3.4.2)
s_{ui}	=	undrained shear strength of "ith" soil layer not to exceed 5 (ksf) (3.4.2.2)
T	=	period of vibration (sec); fundamental period of the structure (sec) (3.4.1) (4.3.3)
T_c	=	column tensile force associated with the column overstrength plastic hinging moment, M_{po} (kip) (6.4.5) (8.13.2)
T_F	=	bridge fundamental period (sec) (3.4.3)
T_i	=	natural period of the less flexible frame (sec) (4.1.2)
$T_{(i)}^{pile}$	=	tension force in "ith" pile (kip) (6.4.2)
T_j	=	natural period of the more flexible frame (sec) (4.1.2)
T_{jv}	=	net tension force in moment resisting footing joints (kip) (6.4.5)
T_m	=	period of the mth mode of vibration (sec) (C5.4.2)
T_o	=	period at beginning of constant design spectral acceleration plateau (sec) (3.4.1)
T_s	=	period at the end of constant design spectral acceleration plateau (sec) (3.4.1) (4.3.3)
T^*	=	characteristic ground motion period (sec) (4.3.3)

t = thickness of unstiffened or stiffened element (in.); pipe wall thickness (in.); thickness of gusset plate (in.); thickness of the top or bottom slab (in.) (7.4.2) (7.6.2) (7.7.5) (8.11)

t_w = thickness of web plate (in.) (7.4.2)

V_c = nominal shear resistance of the concrete (kip) (8.6.1) (8.6.2)

V_g = shear force acting on the gusset plate (kip) (7.7.10)

V_n = nominal interface shear capacity of shear key as defined in Article 5.8.4 of the *AASHTO LRFD Bridge Design Specifications* using the expected material properties and interface surface conditions (kip); nominal shear capacity (kip) (4.14) (6.3.7) (8.6.1) (8.6.9)

V_{ng} = nominal shear strength of a gusset plate (kip) (7.7.9)

V_{ok} = overstrength capacity of shear key (4.14) (8.12)

V_{po} = overstrength shear associated with the overstrength moment M_{po} (kip) (4.11.2) (6.3.4) (6.3.5) (8.6.1)

V_{rg} = factored nominal yield shear capacity of the gusset plate (kip) (7.7.10)

V_s = nominal shear resistance provided by the transverse steel (kip) (8.6.1) (8.6.3) (8.6.4)

V_{s1} = normalized shear wave velocity (6.8)

$V_s(x)$ = static displacement calculated from the uniform load method (ft or in.) (C5.4.2)

V_u = factored ultimate shear demand in footing at the face of the column or wall (kip); shear demand of a column or wall (kip) (6.3.7) (8.6.1) (8.6.9)

v_c = concrete shear stress capacity (ksi) (8.6.2)

v_{jv} = nominal vertical shear stress in a moment resisting joint (ksi) (6.4.5) (8.13.2)

$\bar{v}_s$ = average shear wave velocity in the top 100 ft (ft/sec) (3.4.2)

v_{si} = shear wave velocity of "*i*th" soil layer (ft/sec) (3.4.2.2)

$v_{s,max}$ = maximum lateral displacement due to uniform loading p_o (ft or in.) (C5.4.2)

W = total weight of bridge (kip) (C5.4.2)

W_{cover} = weight of soil covering the footing in a rocking bent (kip) (A.1)

$W_{footing}$ = weight of footing in a rocking bent (kip) (A.1)

W_s = weight of superstructure tributary to rocking bent (kip) (A.1)

W_T = total weight at base of footing for rocking bent (kip) (A.1)

W_w = width of backwall (ft) (5.2.3.3)

w = moisture content (%) (3.4.2.1)

$w(x)$ = nominal unfactored dead load of the bridge superstructure and tributary substructure (kip-in. or kip-ft) (C5.4.2)

Z = plastic section modulus of steel pipe (in.^{3}) (7.6.2)

Z_g = plastic section modulus of gusset plate about the strong axis (in.^{3}) (7.7.8)

β = central angle formed between neutral axis chord line and the center point of the pipe found by the recursive equation (rad) (7.6.2)

γ_{EQ} = load factor for live load (C4.6)

Δ_b = displacement demand due to flexibility of essentially elastic components such as bent caps (in.) (4.3) (4.8)

Δ_{col} = displacement contributed by deformation of the columns (in.) (4.8)

Δ^y_{col} = yield displacement of the column (in.)(4.8)

Δ^L_C = displacement capacity taken along the local principal axis corresponding to Δ^L_D of the ductile member as determined in accordance with Article 4.8.1 for SDCs B and C and in accordance with Article 4.8.2 for SDC D (in.) (C3.3) (4.8) (4.8.1)

Δ_D = global seismic displacement demand (in.) (4.3.1) (4.11.5)

Δ^L_D = displacement demand taken along the local principal axis of the ductile member as determined in accordance with Article 4.4 (in.) (C3.3) (4.8)

Δ_{eq} = seismic displacement demand of the long-period frame on one side of the expansion joint (in.) (4.12.2)

Δ_F = pile cap displacement (in.) (4.11.5)

Δ_f = displacement demand attributed to foundation flexibility; pile cap displacements (in.) (4.3) (4.8)

Δ_{fo} = displacement contributed by flexural effects in column (ft or in.) (A.1)

Δ_{pd} = displacement demand attributed to inelastic response of ductile members; plastic displacement demand (in.) (4.3) (4.9)

Δ_p = displacement contributed by inelastic response (in.) (4.8)

Δ_r = relative lateral offset between the point of contraflexure and the farthest end of the plastic hinge (in.) (4.11.5)

Δ_{ro} = displacement contributed by rocking (ft or in.) (A.1)

Δ_S = pile shaft displacement at the point of maximum moment developed in ground (in.) (4.11.5)

Δ_{Y1} = displacement at which the first element yields (in.) (4.8)

Δ_{Y2} = displacement at which the second element yields (in.) (4.8)

Δ_{Y3} = displacement at which the third element yields (in.) (4.8)

Δ_{Y4} = displacement at which the fourth element yields (in.) (4.8)

Δ_y = idealized yield displacement; displacement demand attributed to elastic response of ductile members (in.) (C3.3) (4.3) (4.8)

Δ_{yi} = idealized yield displacement (in.) (C3.3) (4.9)

σ_v' = effective soil pressure (psf or ksf) (C6.4.3)

ε_{cc} = compressive strain at maximum compressive stress of confined concrete (8.4.4)

ε_{co} = unconfined concrete compressive strain at the maximum compressive stress (8.4.4)

ε_{cu} = ultimate compressive strain for confined concrete (8.4.4)

ε_{sp} = ultimate unconfined compression (spalling) strain (8.4.4)

ε_{ps} = strain in prestressing steel (in./in.) (8.4.3)

$\varepsilon_{ps,EE}$ = essentially elastic prestress steel strain (8.4.3)

$\varepsilon_{ps,u}$ = ultimate prestress steel strain (8.4.3)

$\varepsilon_{ps,u}^{R}$ = reduced ultimate prestress steel strain (8.4.3)

ε_{sh} = tensile strain at the onset of strain hardening (8.4.2)

ε_{su} = ultimate tensile strain (8.4.2)

ε_{su}^{R} = reduced ultimate tensile strain (8.4.2)

ε_{ye} = expected yield strain (8.4.2)

μ = displacement ductility capacity of the end diaphragm (7.4.6)

μ_C = ductility capacity (4.7.1)

μ_D = maximum local member displacement ductility demand (4.3.3) (4.7.1) (4.9) (8.6.2)

λ_b = slenderness parameter for flexural moment dominant members (7.4.1)

λ_{bp} = limiting slenderness parameter for flexural moment dominant members (7.4.1)

λ_c = slenderness parameter for axial compressive load dominant members (7.4.1)

λ_{cp} = limiting slenderness parameter for axial compressive load dominant members (7.4.1)

λ_{mo} = overstrength factor (4.11.2) (7.3) (8.5)

λ_p = limiting width–thickness ratio for ductile components (7.4.2)

λ_r = limiting width–thickness ratio for essentially elastic components (7.4.2)

ρ_h = horizontal reinforcement ratio in pier wall (8.6.9) (8.6.10)

ρ_s = volumetric ratio of spiral reinforcement for a circular column (8.6.2) (8.6.5) (8.8.7) (8.13.3)

ρ_v = vertical reinforcement ratio in pier wall (8.6.10)

ρ_w = reinforcement ratio in the direction of bending (8.6.2) (8.6.5) (8.8.7)

ϕ = resistance factor; soil friction angle (deg); curvature (1/ft or 1/in.) (3.7) (C6.2.2) (6.3.4) (6.3.5) (6.3.6) (7.3) (8.5)

ϕ_b = 0.9 resistance factor for flexure (7.4.2)

ϕ_{bs} = 0.80 resistance factor for block shear failure mechanisms (7.7.6)

ϕ_c = 0.75 resistance factor for concrete in compression (7.6.1)

ϕ_f = 1.0 resistance factor for structural steel in flexure (7.6.2)

ϕ_s = 0.90 resistance factor for shear in reinforce concrete (6.3.7) (8.6.1) (8.6.9)

ϕ_u = 0.80 resistance factor for fracture on net section; ultimate curvature capacity (7.7.6) (8.5)

ϕ_y = curvature of section at first yield of the reinforcing steel including the effects of the unfactored axial dead load (1/in.); 0.95 resistance factor for yield on gross section (5.6.2) (8.5); (7.7.6)

ϕ_{yi} = idealized yield curvature (8.5)

Λ = factor for column end restraint condition (4.8.1) (8.7.1)

ξ = damping ratio (maximum of 0.1) (4.3.2)

$\sum_{i=1}^{n} d_i$ = thickness of upper soil layers = 100 ft (3.4.2.2)

$\sum T_{(i)}^{pile}$ = summation of the hold-down force in the tension piles (kip) (6.4.5)

$\sum P$ = total unfactored axial load due to dead load, earthquake load, footing weight, soil overburden, and all other vertical demands acting on the pile group (kip) (6.4.2)

SECTION 3: GENERAL REQUIREMENTS

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SECTION 3:

GENERAL REQUIREMENTS

3.1—APPLICABILITY OF GUIDE SPECIFICATIONS

These Guide Specifications shall be taken to apply to the design and construction of conventional bridges to resist the effects of earthquake motions. For non-conventional bridges, the Owner shall specify appropriate provisions, approve them, or both.

Critical/essential bridges are not specifically addressed in these Guide Specifications. A bridge should be classified as critical/essential as follows:

- Bridges that are required to be open to all traffic once inspected after the design earthquake and usable by emergency vehicles and for security, defense, economic, or secondary life safety purposes immediately after the design earthquake.
- Bridges that should, as a minimum, be open to emergency vehicles and for security, defense, or economic purposes after the design earthquake and open to all traffic within days after that event.
- Bridges that are formally designated as critical for a defined local emergency plan.

For other types of construction (e.g., suspension bridges, cable-stayed bridges, truss bridges, arch type bridges, and movable bridges), the Owner shall specify or approve appropriate provisions.

Seismic effects for box culverts and buried structures need not be considered except where failure of the box culvert or buried structures will affect the function of the bridge. The potential effects of unstable ground conditions (e.g., liquefaction, landslides, and fault displacements) on the function of the bridge should be considered.

The provisions in these Guide Specifications should be taken as the minimum requirements. Additional provisions may be specified by the Owner to achieve higher performance criteria for repairable or minimum damage attributed to essential or critical bridges. Where such additional requirements are specified, they shall be site or project specific and are tailored to a particular structure type.

No detailed seismic structural analysis should be required for a single-span bridge or for any bridge in SDC A. Specific detailing requirements are applied for SDC A. For single-span bridges, minimum support length requirement shall apply according to Article 4.12.

C3.1

For the purpose of these provisions, conventional bridges have slab, beam, box girder, and truss superstructures; have pier-type or pile-bent substructures; and are founded on shallow- or piled-footings or shafts. Non-conventional bridges include bridges with cable-stayed or cable-suspended superstructures, bridges with truss towers or hollow piers for substructures, and arch bridges.

3.2—PERFORMANCE CRITERIA

Bridges shall be designed for the life safety performance objective considering a seismic hazard corresponding to a seven percent probability of exceedance in 75 yr. Higher levels of performance, such as the operational objective, may be established and authorized by the Bridge Owner. Development of design earthquake ground motions for the seven percent probability of exceedance in 75 yr shall be as specified in Article 3.4.

Life safety for the design event shall be taken to imply that the bridge has a low probability of collapse but may suffer significant damage and that significant disruption to service is possible. Partial or complete replacement may be required.

Significant damage shall be taken to include permanent offsets and damage consisting of:

- Cracking,
- Reinforcement yielding,
- Major spalling of concrete,
- Extensive yielding and local buckling of steel columns,
- Global and local buckling of steel braces, and
- Cracking in the bridge deck slab at shear studs.

These conditions may require closure to repair the damage. Partial or complete replacement of columns may be required in some cases.

For sites with lateral flow due to liquefaction, inelastic deformation may be permitted in the piles. Partial or complete replacement of the columns and piles may be necessary if significant lateral flow occurs.

If replacement of columns or other components is to be avoided, the design strategy producing minimal or moderate damage such as seismic isolation or the control and reparability design concept should be assessed. For locations where lateral flow is expected, the design strategy should consider the use of ground improvement methods that limit the amount of lateral ground movement.

Significant disruption to service shall be taken to include limited access (reduced lanes, light emergency traffic) on the bridge. Shoring may be required.

C3.2

These Guide Specifications are intended to achieve minimal damage to bridges during moderate earthquake ground motions and to prevent collapse during rare earthquakes that result in high levels of ground shaking at the bridge site. Bridge Owners may choose to mandate higher levels of bridge performance for special bridges.

The seismic hazard used in these Guide Specifications corresponds to a seven percent probability of exceedence (PE) in 75 yr. The precise definition used in the development of the ground shaking hazards maps and the ground motion design tool is five percent in 50 yr. Thus, the return period used in development of the hazard maps and in the design tool is actually 975 yr compared to that of seven percent PE in 75 yr of 1,033 yr. While this distinction has little significance in an engineering sense, it is a consideration when conducting site-specific hazard analyses.

Allowable displacements are constrained by geometric, structural, and geotechnical considerations. The most restrictive of these constraints will govern displacement capacity. These displacement constraints may apply to either transient displacements as would occur during ground shaking, permanent displacements as may occur due to seismically induced ground failure or permanent structural deformations or dislocations, or a combination. The extent of allowable displacements depends on the desired performance level of the bridge design.

Geometric constraints generally relate to the usability of the bridge by traffic passing on or under it. Therefore, this constraint will usually apply to permanent displacements that occur as a result of the earthquake. The ability to repair such displacements or the desire not to be required to repair them should be considered when establishing displacement capacities. When uninterrupted or immediate service is desired, the permanent displacements should be small or nonexistent and should be at levels that are within an accepted tolerance for normally operational highways of the type being considered.

A bridge designed to a performance level of no collapse could be expected to be unusable after liquefaction, for example, and geometric constraints would have no influence. However, because life safety is at the heart of the no collapse requirement, jurisdictions may consider establishing some geometric displacement limits for this performance level for important bridges or those with high average daily traffic (*ADT*). This can be done by considering the risk to highway users in the moments during or immediately following an earthquake. For example, an abrupt vertical dislocation of the highway of sufficient height could present an insurmountable barrier and thus result in a collision that could kill or injure. Usually these types of geometric displacement constraints will be less restrictive than those resulting from structural considerations; for bridges on liquefiable sites, it may not be economical to prevent significant displacements from occurring.

3.3—EARTHQUAKE-RESISTING SYSTEMS (ERS) REQUIREMENTS FOR SDCS C AND D

For SDC C or D (see Article 3.5), all bridges and their foundations shall have a clearly identifiable earthquake-resisting system (ERS) selected to achieve the life safety criteria defined in Article 3.2. For SDC B, identification of an ERS should be considered.

The ERS shall provide a reliable and uninterrupted load path for transmitting seismically induced forces into the surrounding soil and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen design strategy of seismic resistance and other structural requirements.

Design should be based on the following three Global Seismic Design Strategies used in these Guide Specifications based on the expected behavior characteristics of the bridge system:

- **Type 1**—Ductile Substructure with Essentially Elastic Superstructure: This category includes conventional plastic hinging in columns and walls and abutments that limits inertial forces by full mobilization of passive soil resistance. Also included are foundations that may limit inertial forces by in-ground hinging, such as pile bents and integral abutments on piles.
- **Type 2**—Essentially Elastic Substructure with a Ductile Superstructure: This category applies only to steel superstructures, and ductility is achieved by ductile elements in the pier cross-frames.
- **Type 3**—Elastic Superstructure and Substructure with a Fusing Mechanism between the Two: This category includes seismically isolated structures and structures in which supplemental energy-dissipation devices, such as dampers, are used to control inertial forces transferred between the superstructure and substructure.

See also Article 7.2 for further discussion of performance criteria for steel structures.

For the purposes of encouraging the use of appropriate systems and of ensuring due consideration of performance for the Owner, the ERS and earthquake-resisting elements (EREs) shall be categorized as follows:

- Permissible,
- Permissible with Owner's approval, and
- Not recommended for new bridges.

These terms shall be taken to apply to both systems and elements. For a system to be in the permissible category, its primary EREs shall be in the permissible category. If any ERE is not permissible, then the entire system shall be considered not permissible.

C3.3

Common examples from each of the three ERS and ERE categories are shown in Figures 1a and 1b, respectively. Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of the lateral-force-resisting concept and the selection of the necessary elements to fulfill the concept should be accomplished in the conceptual design phase, or the type, size, and location phase, or the design alternative phase of a project.

For SDC B, it is suggested that the ERS be identified. The displacement checks for SDC B are predicated on the existence of a complete lateral load resisting system; thus, the Designer should ensure that an ERS is present and that no unintentional weak links exist. Additionally, identifying the ERS helps the Designer ensure that the model used to determine displacement demands is compatible with the drift limit calculation. For example, pile-bent connections that transmit moments significantly less than the piles can develop should not be considered as fixed connections.

Seismic performance is typically better in systems with regular configurations and evenly distributed stiffness and strength. Typical geometric configuration constraints, such as skew, unequal pier heights, and sharp curves, may conflict with seismic design goals. For this reason, it is advisable to resolve potential conflicts between configuration and seismic performance early in the design effort. For example, resolution may lead to decreased skew angles at the expense of longer end spans. The resulting trade-off between performance and cost should be evaluated in the type, size, and location phase, or design alternative phase, of a project, when design alternatives are viable from a practical viewpoint.

The classification of ERS and EREs into permissible and not recommended categories is meant to trigger consideration of seismic performance that leads to the most desirable outcome, that is, seismic performance that ensures, wherever possible, post-earthquake serviceability. To achieve such an objective, special care in detailing the primary energy-dissipating elements is necessary. Conventional reinforced concrete construction with ductile plastic hinge zones can continue to be used, but designers should be aware that such detailing, although providing desirable seismic performance, will leave the structure in a damaged state following a large earthquake. It may be difficult or impractical to repair such damage.

Under certain conditions, the use of EREs that require the Owner's approval will be necessary. In previous AASHTO seismic specifications, some of the EREs in the Owner's approval category were simply not permitted for use (e.g., in-ground hinging of piles and shafts and foundation rocking). These elements are now permitted, provided their deformation performance is assessed.

This approach of allowing their use with additional analytical effort was believed to be preferable to an outright ban on their use. Thus, it is not the objective of these Guide Specifications to discourage the use of systems that require Owner approval. Instead, such systems may be used, but additional design effort and consensus between the designer and Owner are required to implement such systems. Additionally, these Guide Specifications do not provide detailed guidance for designing all such systems, for example Case 2 in Figure 2. If such systems are used, then case-specific criteria and design methodologies will need to be developed and agreed upon by the Designer and the Owner.

Bridges are seismically designed so that inelastic deformation (damage) intentionally occurs in columns so that the damage can be readily inspected and repaired after an earthquake. Capacity design procedures are used to prevent damage from occurring in foundations and beams of bents and in the connections of columns to foundations and columns to the superstructure. There are two exceptions to this design philosophy. For pile bents and drilled shafts, some limited inelastic deformation is permitted below the ground level. The amount of permissible deformation is restricted to ensure that no long-term serviceability problems occur from the amount of cracking that is permitted in the concrete pile or shaft. The second exception is with lateral spreading associated with liquefaction. For the life safety performance level, significant inelastic deformation is permitted in the piles. It is a costly and difficult problem to achieve a higher performance level from piles.

There are a number of design approaches that can be used to achieve the performance objectives. These are discussed briefly below.

Type 1—Ductile Substructure with Essentially Elastic Superstructure

Caltrans first introduced this design approach in 1973 following the 1971 San Fernando earthquake. It was further refined and applied nationally in the 1983 *AASHTO Guide Specification for Seismic Design of Highway Bridges*, which was adopted directly from the *ATC-6 Report, Seismic Design Guidelines for Highway Bridges* (ATC, 1981). These provisions were adopted by AASHTO in 1991 as their standard seismic provisions.

Permissible systems and elements depicted in Figures 1a and 1b shall have the following characteristics:

- All significant inelastic action shall be ductile and occur in locations with adequate access for inspection and repair. Piles subjected to lateral movement from lateral flow resulting from liquefaction are permitted to hinge below the ground line provided the Owner is informed and does not require any higher performance criteria for a specific objective. If all structural elements of a bridge are designed elastically, then no inelastic deformation is anticipated and elastic elements are permissible, but minimum detailing is required according to the bridge seismic design category.
- Inelastic action of a structural member does not jeopardize the gravity load support capability of the structure (e.g., cap beam and superstructure hinging).

Permissible elements depicted in Figure 2 that do not meet either criterion above may be used only with approval by the Owner.

Examples of elements that do not fall in either of the two permissible categories depicted in Figure 3 shall be considered not recommended. However, if adequate consideration is given to all potential modes of behavior and potential undesirable failure mechanisms are suppressed, then such systems may be used with the Owner's approval.

This approach is based on the expectation of significant inelastic deformation (damage) associated with ductility ≥ 4.

The other key premise of the provisions is that displacements resulting from the inelastic response of a bridge are approximately equal to the displacements obtained from an analysis using the linear elastic response spectrum. As diagrammatically shown in Figure C1, this assumes that Δ_C^L is approximately equal to Δ_D^L. Work by Miranda and Bertero (1994) and by Chang and Mander (1994a and 1994b) indicates that this is a reasonable assumption, except for short-period structures for which it is nonconservative. A correction factor to be applied to elastic displacements to address this issue is given in Article 4.3.3.

Type 2—Essentially Elastic Substructure with a Ductile Superstructure

This category applies only to steel superstructures. The ductility is achieved by constructing ductile elements as part of the cross-frames of a steel slab-on-girder bridge superstructure. The deformation capacity of the cross-frames located at each pier permits lateral displacement of the deck relative to the substructure below. This is an emerging technology and has not been widely used as a design strategy for new construction.

Type 3—Elastic Superstructure and Substructure with a Fusing Mechanism between the Two

This category comprises seismically isolated structures and structures in which energy-dissipation devices are used across articulation joints to provide a mechanism to limit energy buildup and associated displacements during a large earthquake. The two subcategories are discussed further below.

Seismic Isolation. This design approach reduces the seismic forces a bridge needs to resist by introducing an isolation bearing with an energy-dissipation element at the bearing location. The isolation bearing intentionally lengthens the period of a relatively stiff bridge, and this results in lower design forces, provided the design is in the decreasing portion of the acceleration response spectrum. This design alternative was first applied in the United States in 1984 and has been extensively reported on at technical conferences and seminars and in the technical literature. AASHTO adopted *Guide Specifications for Seismic Isolation Design of Highway Bridges* in 1991, and these have subsequently been revised. The 1999 revisions are now referred to in Section 7 of these Guide Specifications. Elastic response of the substructure elements is possible with seismic isolation because the elastic forces resulting from seismic isolation are generally less than the reduced design forces required by conventional ductile design.

Energy Dissipation. This design approach adds energy-dissipation elements between the superstructure and the substructure and between the superstructure and abutment, with the intent of dissipating energy in these elements. This eliminates the need for energy dissipation in the plastic hinge zones of columns. This design approach differs from seismic isolation in that additional flexibility is generally not part of the system and thus the fundamental period of vibration is not changed. If the equivalent viscous damping of the bridge is increased above five percent, then the displacement of the superstructure will be reduced. In general, the energy-dissipation design concept does not result in reduced design forces, but it will reduce the ductility demand on columns due to the reduction in superstructure displacement (ATC, 1993). This is an emerging technology and has not been widely used as a design strategy for new construction.

Abutments as an Additional Energy-Dissipation Mechanism

In the early phases of the development of these Guide Specifications, there was serious debate as to whether or not the abutments would be included and relied on in the ERS. Some states may require the design of a bridge in which the substructures are capable of resisting the entire lateral load without any contribution from the abutments. In this design approach, the abutments are included in a mechanism to provide an unquantifiable higher level of safety. Rather than mandate this design philosophy here, it was decided to permit two design alternatives. The first is where the ERS does not include the abutments and the substructures are capable of resisting all the lateral loads. In the second alternative, the abutments are an important part of the ERS and, in this case, a higher level of analysis is required.

If the abutment is included as part of the ERS, this design option requires a continuous superstructure to deliver longitudinal forces to the abutment. If these conditions are satisfied, the abutments can be designed as part of the ERS and become an additional source for dissipating the bridge's earthquake energy. In the longitudinal direction, the abutment may be designed to resist the forces elastically using the passive pressure of the backfill. In some cases, the longitudinal displacement of the deck will cause larger soil movements in the abutment backfill, exceeding the passive pressures there. This requires a more refined analysis to determine the amount of expected movement. In the transverse direction, the abutment is generally designed to resist the loads elastically. The design objective when abutments are relied on to resist either longitudinal or transverse loads is either to minimize column sizes or reduce the ductility demand on the columns, accepting that damage may occur in the abutment.

When the abutment is part of the ERS, the performance expectation is that inelastic deformation will occur in the columns as well as the abutments. If large ductility demands occur in the columns, then the columns may need to be replaced. If large movements of the superstructure occur, the abutment backwall may be damaged and there may be some settlement of the abutment backfill. Large movements of the superstructure can be reduced with use of energy dissipators and isolation bearings at the abutments and at the tops of the columns.

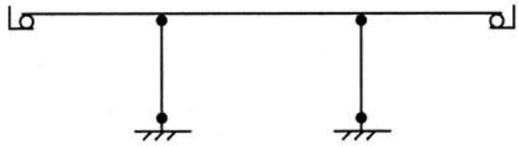
In general, the soil behind an abutment is capable of resisting substantial seismic forces that may be delivered through a continuous superstructure to the abutment. Furthermore, such soil may also substantially limit the overall movements that a bridge may experience. This is particularly so in the longitudinal direction of a straight bridge with little or no skew and with a continuous deck. The controversy with this design concept is the scenario of what may happen if there is significant abutment damage early in the earthquake ground motion duration and if the columns rely on the abutment to resist some of the load. This would be a problem in a long-duration, high-magnitude (greater than magnitude 7) earthquake. Another consideration is if a gap develops between the abutment and the soil after the first cycle of loading, due to the inelastic behavior of the soil when passive pressures are developed.

Unless shock transmission units (STUs) are used, a bridge composed of multiple simply supported spans cannot effectively mobilize the abutments for resistance to longitudinal force. It is recommended that simply supported spans not rely on abutments for any seismic resistance.

Because structural redundancy is desirable (Buckle et al., 1987), good design practice dictates the use of the design alternative in which the intermediate substructures, between the abutments, are designed to resist all seismic loads, if possible. This ensures that in the event abutment resistance becomes ineffective, the bridge will still be able to resist the earthquake forces and displacements. In such a situation, the abutments provide an increased margin against collapse.

Longitudinal Response

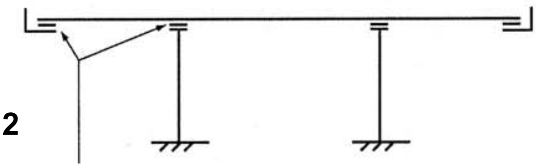
1



- Plastic hinges in inspectable locations or elastic design of columns
- Abutment resistance not required as part of ERS
- Knock-off backwalls permissible

Longitudinal Response

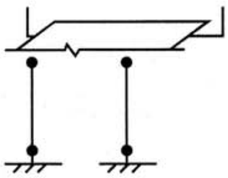
2



- Isolation bearings accommodate full displacement
- Abutment not required as part of ERS

Transverse Response

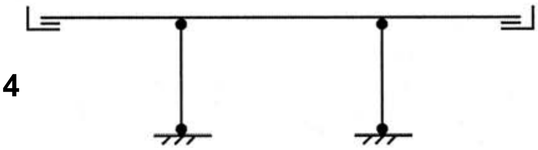
3



- Plastic hinges in inspectable locations
- Abutment not required in ERS, breakaway shear keys permissible

Transverse or Longitudinal Response

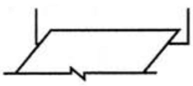
4



- Plastic hinges in inspectable locations
- Isolation bearings with or without energy dissipaters to limit overall displacements

Transverse or Longitudinal Response

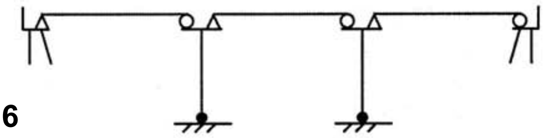
5



- Abutment required to resist the design earthquake elastically
- Longitudinal passive soil pressure shall be less than 0.70 of the value obtained using the procedure given in Article 5.2.3

Longitudinal Response

6



- Multiple simply-supported spans with adequate support lengths
- Plastic hinges in inspectable locations

Figure 3.3-1a—Permissible Earthquake-Resisting Systems (ERSs)

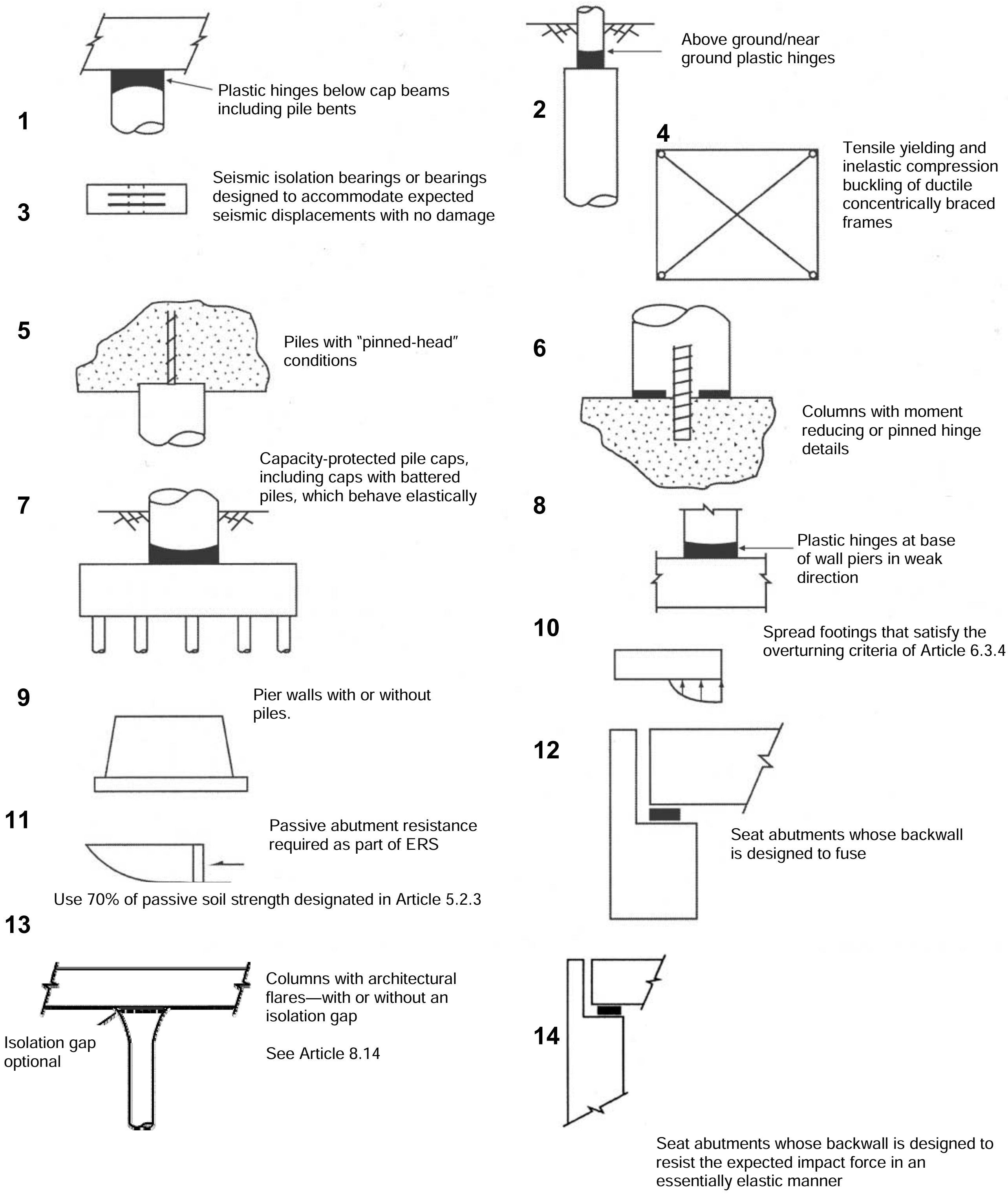


Figure 3.3-1b—Permissible Earthquake-Resisting Elements (EREs)

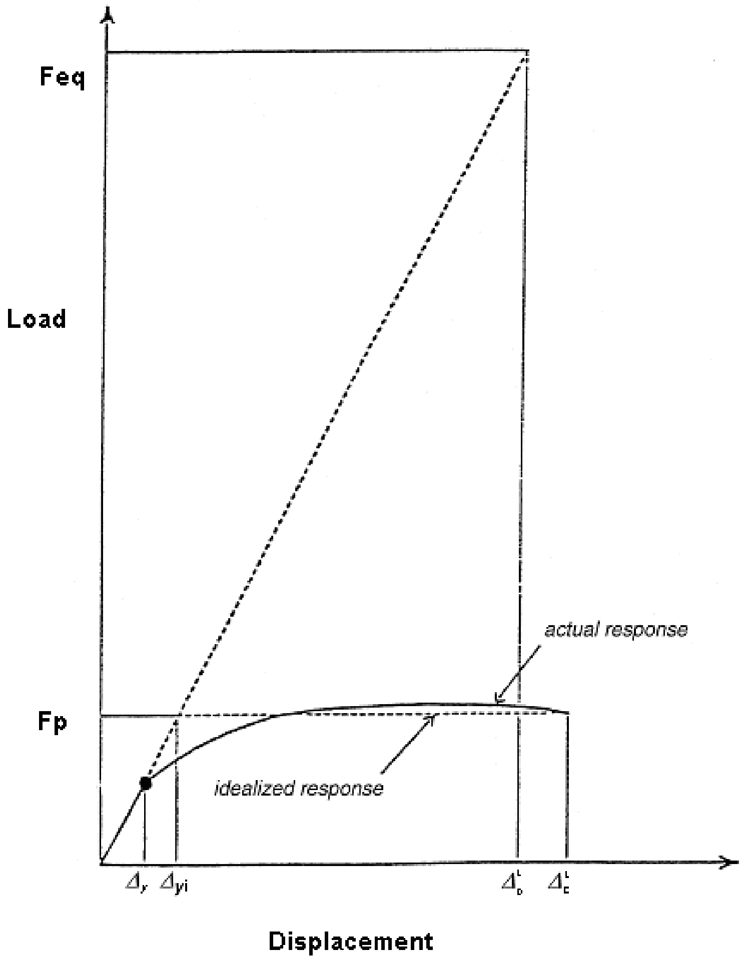


Figure C3.3-1—Design Using Strategy Type 1

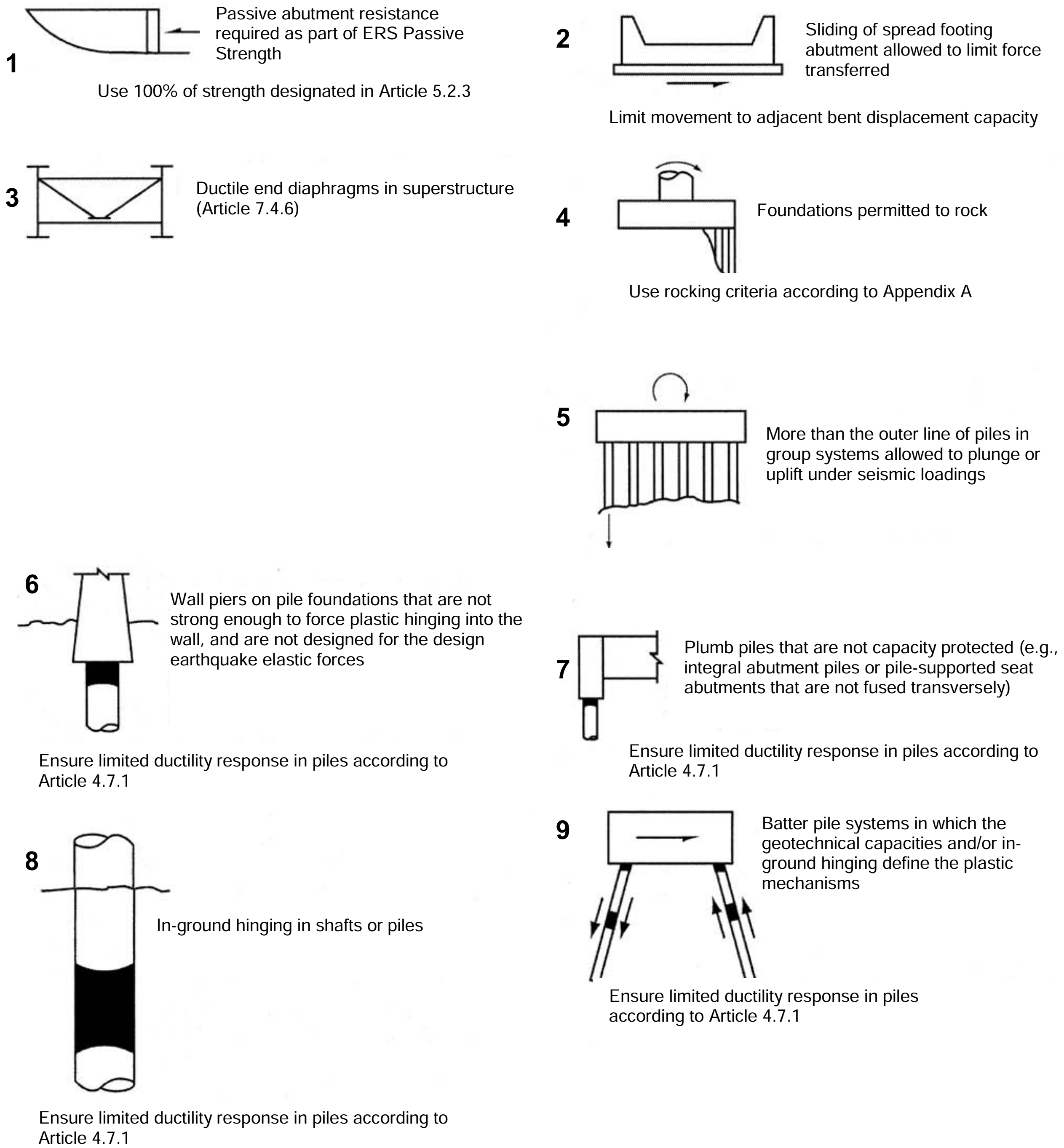


Figure 3.3-2—Permissible Earthquake-Resisting Elements that Require Owner's Approval

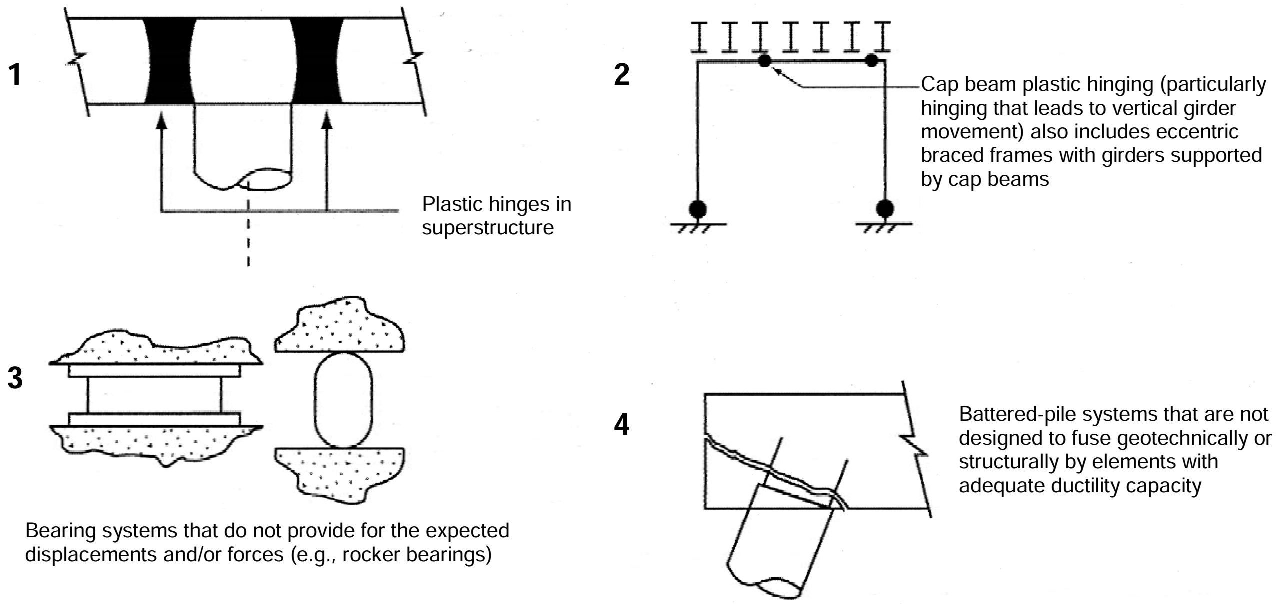


Figure 3.3-3—Earthquake-Resisting Elements that Are Not Recommended for New Bridges

3.4—SEISMIC GROUND SHAKING HAZARD

The seismic ground shaking hazard shall be characterized using an acceleration response spectrum. The acceleration response spectrum shall be determined in accordance with the general procedure of Article 3.4.1 or the site-specific procedure of Article 3.4.3.

In the general procedure, the spectral response parameters shall be determined using the USGS/AASHTO Seismic Hazard Maps, produced by the U.S. Geological Survey depicting the probabilistic ground motion and spectral response for seven percent probability of exceedance in 75 yr.

The site-specific procedure shall consist of a site-specific hazard analysis, a site-specific ground motion response analysis, or both. A site-specific hazard analysis should be considered if any of the following apply:

- The bridge is considered to be critical or essential according to Article 4.2.2, for which a higher degree of confidence of meeting the seismic performance objectives of Article 3.2 is desired.
- Information about one or more active seismic sources for the site has become available since the development of the 2002 USGS data that were used to develop the 2006 USGS/AASHTO Seismic Hazard Maps, and the new seismic source information will result in a significant change of the seismic hazard at the site.

A site-specific ground motion response analyses should be performed if any of the following apply:

- The site consists of Site Class F soils, as defined in Article 3.4.2.1.

C3.4

In the general procedure, the spectral response parameters are determined using the USGS/AASHTO Seismic Hazard Maps produced by the U.S. Geological Survey (USGS). These ground motion hazard maps depict probabilistic ground motion and spectral response for seven percent probability of exceedance in 75 yr. Spectral parameters from the USGS/AASHTO Seismic Hazard Maps are for a soft rock/stiff soil condition, defined as Site Class B (see Article 3.4.2), and should be adjusted for local site effects following the methods given in Article 3.4.2. Either site coefficients in Article 3.4.2.3 or site-specific ground motion response analyses (Article 3.4.3.2) can be used to account for local site effects.

Site-specific procedures consist of a site-specific hazard analysis, a site-specific ground motion response analysis, or both.

- *Site-Specific Hazard Analysis*: A site-specific hazard analysis consists of either a deterministic seismic hazard analysis (DSHA) or a probabilistic seismic hazard analysis (PSHA). A DSHA involves evaluating the seismic hazard at a site for an earthquake of a specific magnitude occurring at a specific location, considering the attenuation of the ground motions with distance. The DSHA is usually conducted without regard for the likelihood of occurrence. The product of the DSHA is an estimate of ground motion parameters at a site for each potential source. The PSHA involves evaluation of the probability of seismic shaking considering all possible sources. The USGS conducted a PSHA in the development of the USGS/AASHTO Seismic Hazard Maps. A PSHA consists of completing numerous deterministic seismic hazard analyses for all

- The bridge is considered critical or essential according to Article 4.2.2, for which a higher degree of confidence of meeting the seismic performance objectives of Article 3.2 is desired.

If the site is located within 6 mi of a known active fault capable of producing a magnitude 5 earthquake and near fault effects are not modeled in the development of national ground motion maps, directivity and directionality effects should be considered as described in Article 3.4.3.1 and its commentary.

feasible combinations of earthquake magnitude, source-to-site distance, and seismic activity for each earthquake source zone located in the vicinity of the site. The result of a PSHA is a relationship of the mean annual rate of exceedance of the ground motion parameter of interest with each potential seismic source considered. See Kramer (1996) for further discussions of the types and methods used to conduct DSHAs and PSHAs.

- *Site-Specific Ground Motion Response Analysis*: A site-specific ground response analysis is used to determine the influence of local ground conditions on the design ground motions. The analysis is generally based on the assumption of a vertically propagating shear wave though more complex analyses can be conducted if warranted. A site-specific ground motion response analysis is typically used to evaluate the influence of "non-standard" soil profiles on ground response to the seismic hazard level. Site-specific ground motion response analyses may also be used to assess the effects of pore-water pressure build-up on ground response, vertical motions resulting from compression wave propagation, laterally non-uniform soil conditions, incoherence, and the spatial variation of ground motions.

In these provisions, an active fault is defined as a near-surface or shallow fault whose location is known or can reasonably be inferred and which has exhibited evidence of displacement in Holocene (or recent) time (in the past 11,000 yr, approximately). Active fault locations can be found from maps showing active faults prepared by state geological agencies or the U.S. Geological Survey. The manner in which an active fault is used in a DSHA and a PSHA is different and should be appropriately treated when conducting each type of analysis.

Article C3.4.3 describes near-fault ground-motion effects that are not included in national ground-motion mapping and could potentially increase the response of some bridges. Normally, site-specific evaluation of these effects would be considered only for essential or very critical bridges.

3.4.1—Design Spectra Based on General Procedure

If a site-specific hazard analysis is not conducted, design response spectra shall be constructed using response spectral accelerations taken from national ground motion maps described in this Article and site factors described in Article 3.4.2. The construction of the response spectra shall follow the procedures described below and illustrated in Figure 1.

C3.4.1

National ground-motion maps are based on probabilistic national ground motion mapping conducted by the U.S. Geological Survey (USGS) having a seven percent chance of exceedance in 75 yr. Values for PGA, S_s and S_1 can be obtained from the maps in these Guide Specifications or from the USGS seismic parameters CD-ROM accompanying these Guide Specifications. The CD-ROM provides the coefficients by the latitude and longitude of the bridge site, or by ZIP code for the site. Use of the latitude and longitude is the preferred procedure when using the CD-ROM.

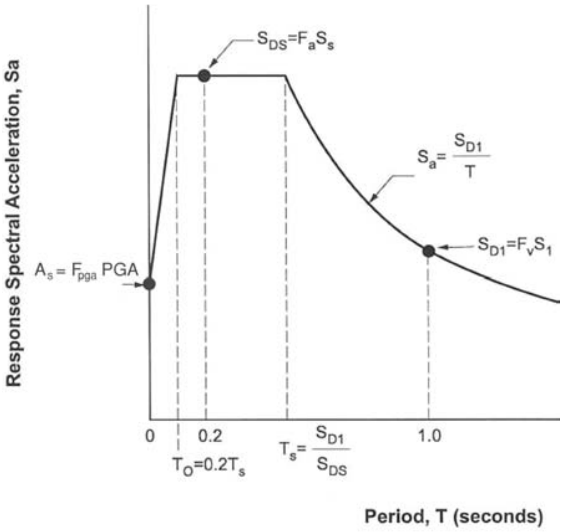


Figure 3.4.1-1—Design Response Spectrum, Construction Using Three-Point Method

Design earthquake response spectral acceleration coefficients for the acceleration coefficient, A_S, the short period acceleration coefficient, S_{DS}, and at the 1-sec period acceleration coefficient, S_{D1}, shall be determined from Eqs. 1 through 3, respectively:

$$A_s = F_{pga}PGA \tag{3.4.1-1}$$

$$S_{DS} = F_aS_s \tag{3.4.1-2}$$

$$S_{D1} = F_vS_1 \tag{3.4.1-3}$$

where:

F_{pga} = site coefficient for peak ground acceleration defined in Article 3.4.2.3

PGA = peak horizontal ground acceleration coefficient on Class B rock

F_a = site coefficient for 0.2-sec period spectral acceleration specified in Article 3.4.2.3

S_s = 0.2-sec period spectral acceleration coefficient on Class B rock

F_v = site coefficient for 1.0-sec period spectral acceleration specified in Article 3.4.2.3

S_1 = 1.0-sec period spectral acceleration coefficient on Class B rock

In lieu of using national ground motion maps referenced in these Guide Specifications, ground motion response spectra may be constructed on the basis of approved state ground motion maps. To be accepted, the development of state maps should conform to the following:

- The definition of design ground motion return period or probability of exceedance should equal or exceed those described in Article 3.2.
- Ground motion maps should be based on a detailed analysis demonstrated to lead to a quantification of ground motion, at a regional scale, that is as accurate or more so as achieved in the national maps. The analysis should include characterization of seismic sources and ground motion that incorporates current scientific knowledge; incorporation of uncertainty in seismic source models, ground motion models, and parameter values used in the analysis; detailed documentation of map development; and detailed peer review as deemed appropriate by the Owner. The peer review process should preferably include individuals from the USGS, other organizations, or both who have expertise in developing probabilistic seismic hazard maps on a regional basis.

The design response spectrum includes the short-period transition from acceleration coefficient, A_s, to the peak response region, S_{DS}, unlike the AASHTO *Standard Specifications for Highway Bridges,* Division I-A. This transition is effective for all modes, including the fundamental vibration modes. Use of the peak response down to zero period is felt to be overly conservative, particularly for displacement-based designs. The use of R_d (see Article 4.3.3) to magnify displacements in the short-period range also offsets the reductions in conservatism when using the transition from A_s to S_{DS}.

For periods exceeding approximately 3 sec, depending on the seismic environment, Eq. 8 may be conservative because the ground motions may be approaching the constant spectral displacement range for which S_a decays with period as $1/T^2$. The long-period transition to constant displacement has been incorporated into recent maps used in the building industry (e.g., *International Building Code* (ICC, 2006). However, the constant displacement portion of the response spectrum has not been included herein. Typical structures for which this region would apply are either long-span non-conventional structures and thus beyond the scope of these Guide Specifications, or they are structures that would warrant a site-specific response analysis. In the latter case, constant displacement attributes of the response spectrum should be considered during the development of the site-specific ground motion hazard. The long-period transition identified in IBC (2006) is for a design earthquake with a two percent probability of exceedance in 50 yr (i.e., 2475-yr return period), and therefore should not be used.

Linear interpolation shall be used to determine the ground motion parameters PGA, S_s, and S_1 for sites located between contour lines or between a contour line and a local maximum or minimum.

The design response spectrum curve shall be developed as follows and as indicated in Figure 1:

- For periods greater than or equal to T_o and less than or equal to T_S, the design response spectral acceleration coefficient, S_a, shall be defined as follows:

$$S_a = \left(S_{DS} - A_s\right)\frac{T}{T_o} + A_s \qquad (3.4.1\text{-}4)$$

in which:

$$T_o = 0.2T_S \qquad (3.4.1\text{-}5)$$

$$T_S = \frac{S_{D1}}{S_{DS}} \qquad (3.4.1\text{-}6)$$

where:

A_s = acceleration coefficient

S_{D1} = design spectral acceleration coefficient at 1.0-sec period

S_{DS} = design spectral acceleration coefficient at 0.2-sec period

T = period of vibration (sec)

- For periods greater than or equal to T_o and less than or equal to T_S, the design response spectral acceleration coefficient, S_a, shall be defined as follows:

$$S_a = S_{DS} \qquad (3.4.1\text{-}7)$$

- For periods greater than T_S, the design response spectral acceleration coefficient, S_a, shall be defined as follows:

$$S_a = \frac{S_{D1}}{T} \qquad (3.4.1\text{-}8)$$

Response spectra constructed using maps and procedures described in Article 3.4.1 are for a damping ratio of five percent and do not include near field ground motion adjustments. See Article 3.4.3.1 for near-field adjustments.

The coefficient obtained for the USGS/AASHTO Seismic Hazard Maps are based on a uniform seismic hazard. The probability that a coefficient will not be exceeded at a given location during a 75-yr period is estimated to be about 93 percent, i.e., seven percent probability of exceedance. The use of a 75-yr interval matches the design life prescribed by the *AASHTO LRFD Bridge Design Specifications.*

It can be shown that an event with a seven percent probability of exceedance in 75 yr has a return period of about 1,000 yr. This earthquake is called the design earthquake.

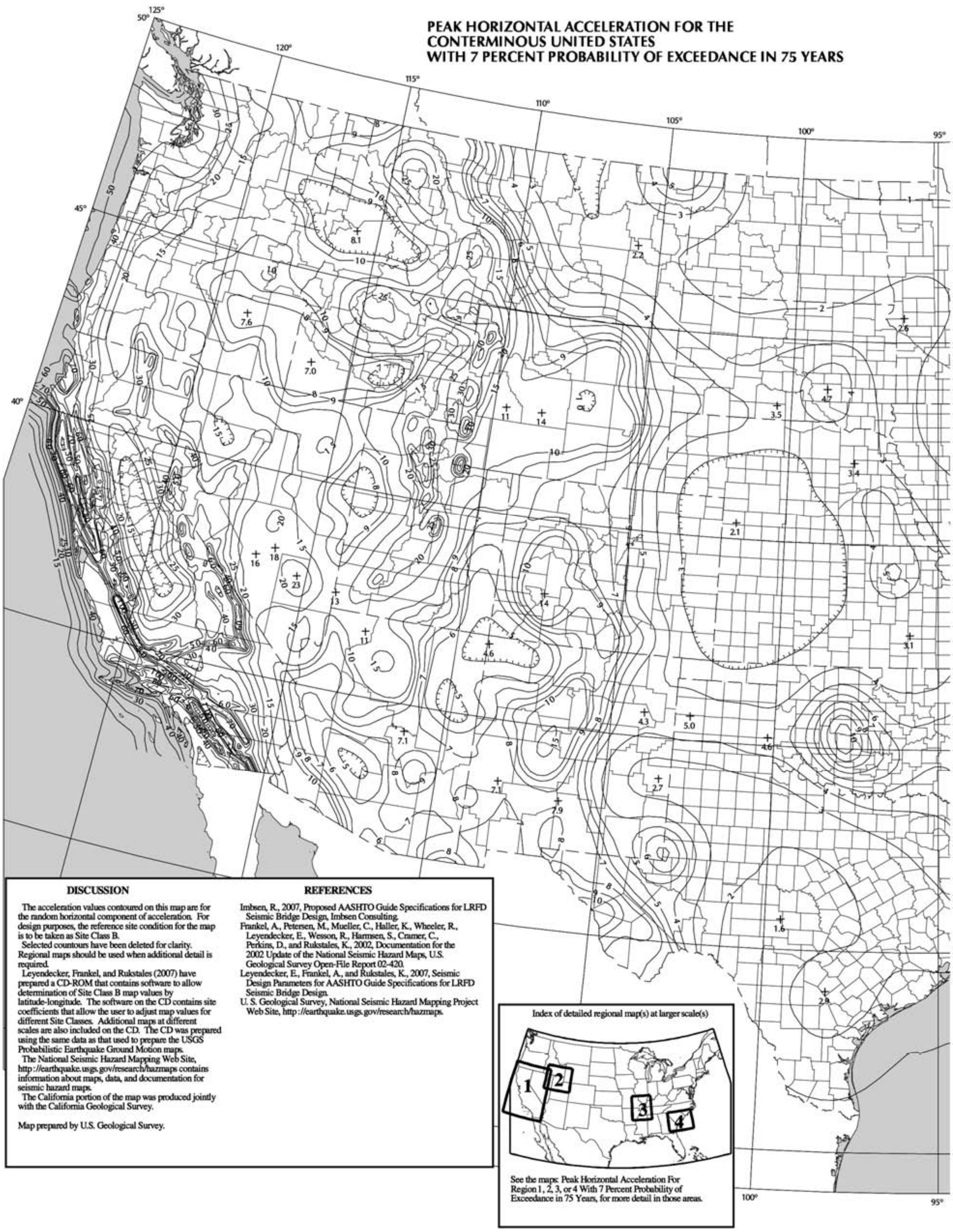


Figure 3.4.1-2a—Horizontal Peak Ground Acceleration Coefficiant for the Conterminous United States (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

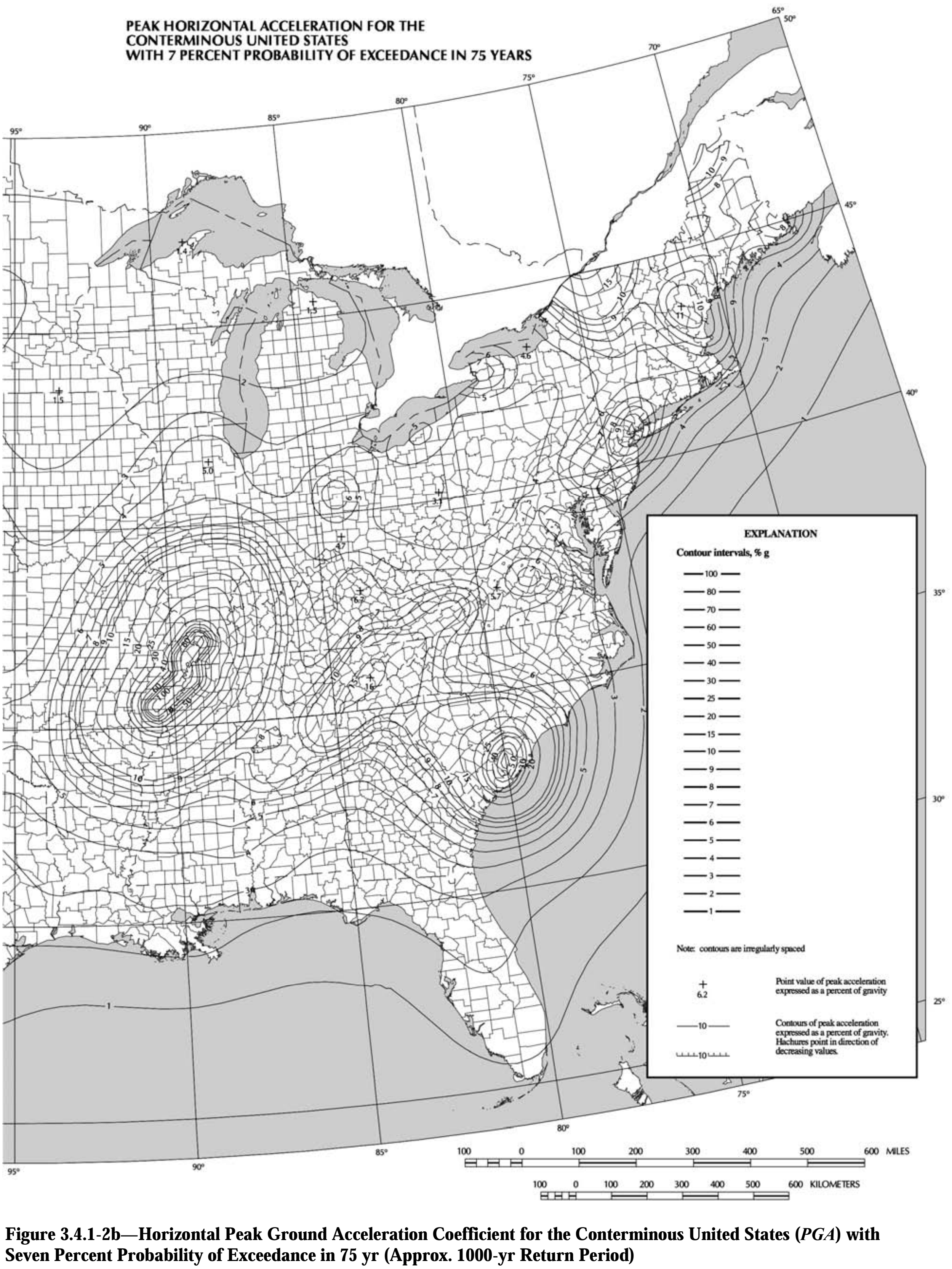


Figure 3.4.1-2b—Horizontal Peak Ground Acceleration Coefficient for the Conterminous United States (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

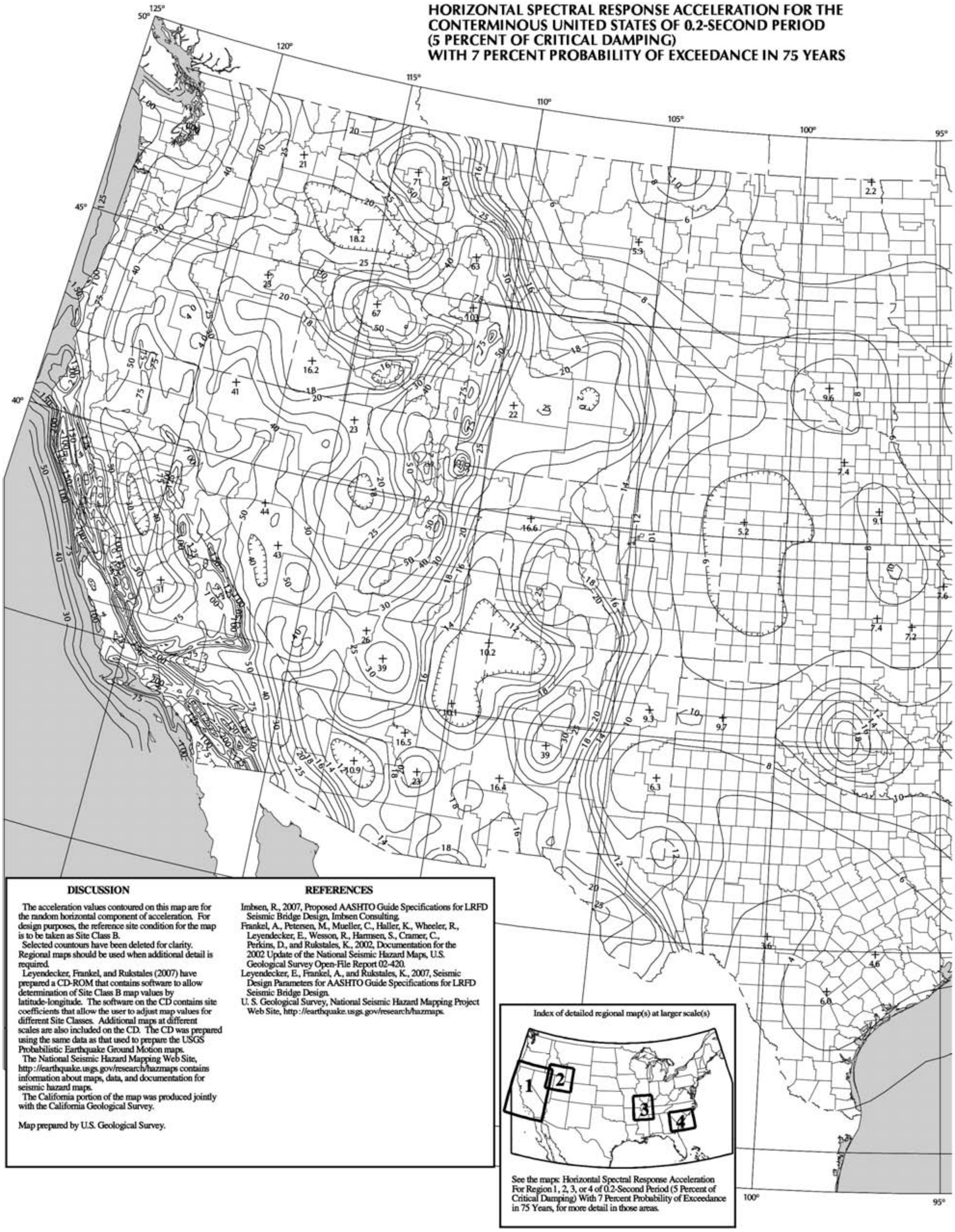


Figure 3.4.1-3a—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 0.2- sec (S_s) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

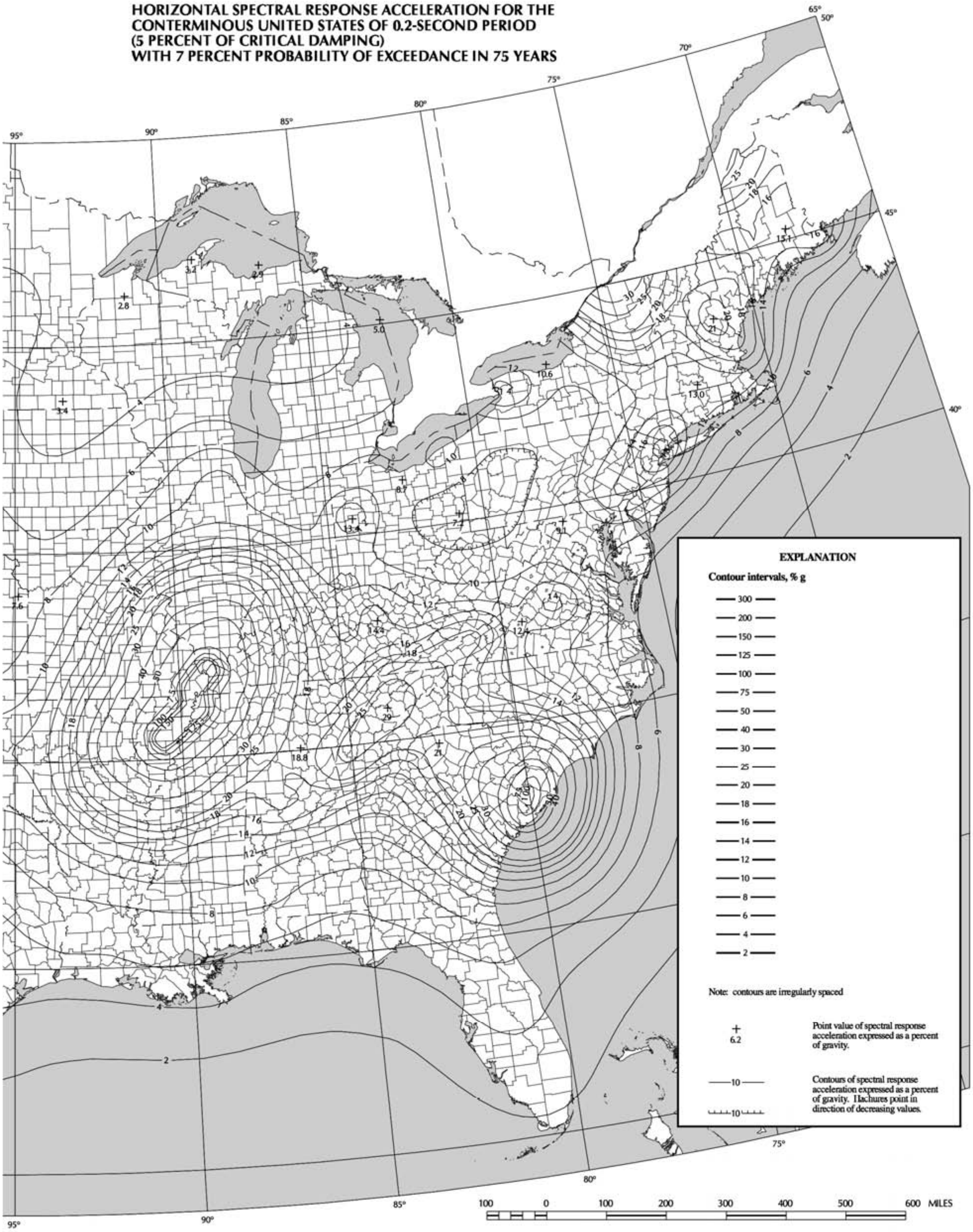


Figure 3.4.1-3b—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 0.2-sec (S_s) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

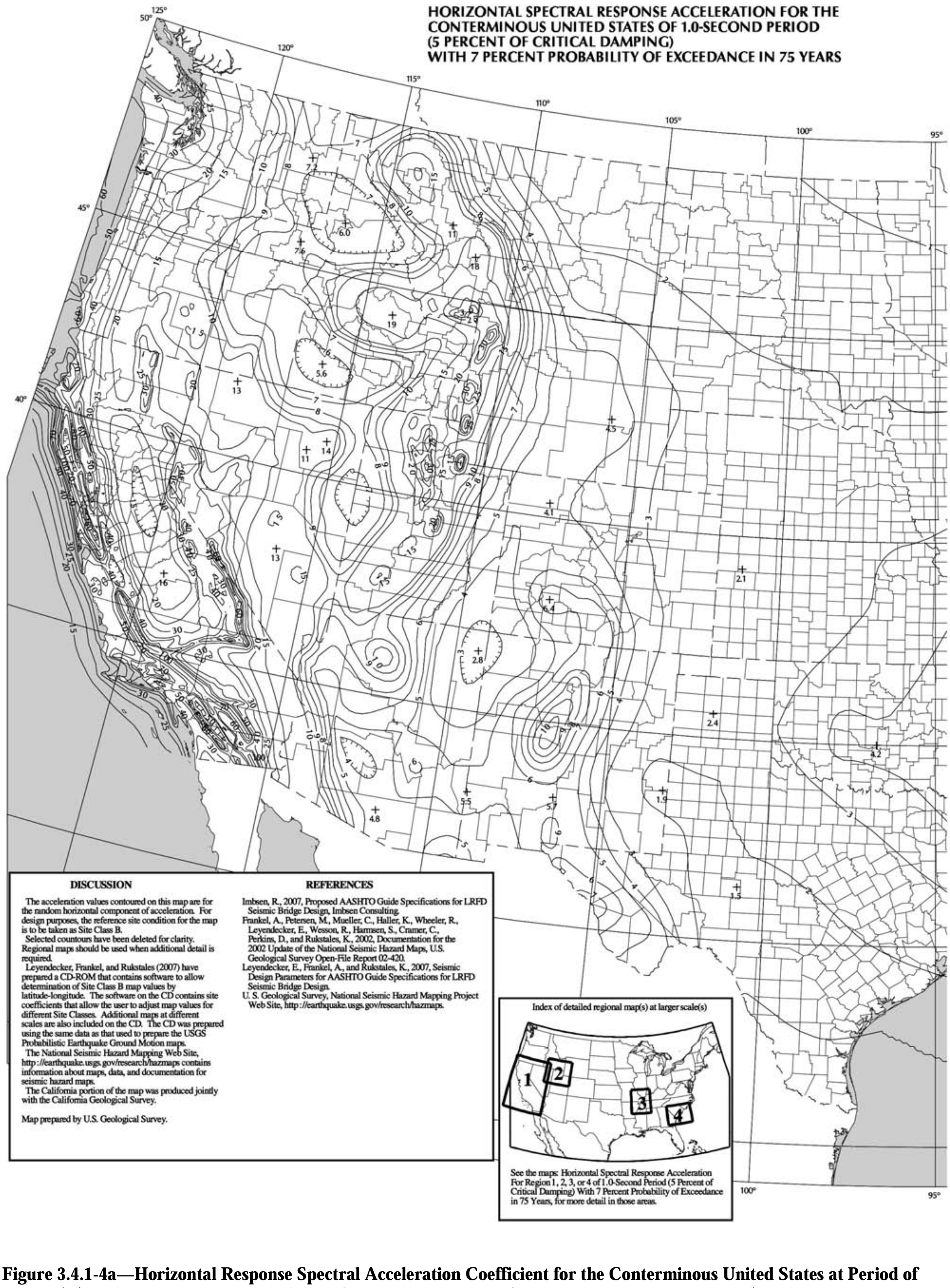


Figure 3.4.1-4a—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 1.0-sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

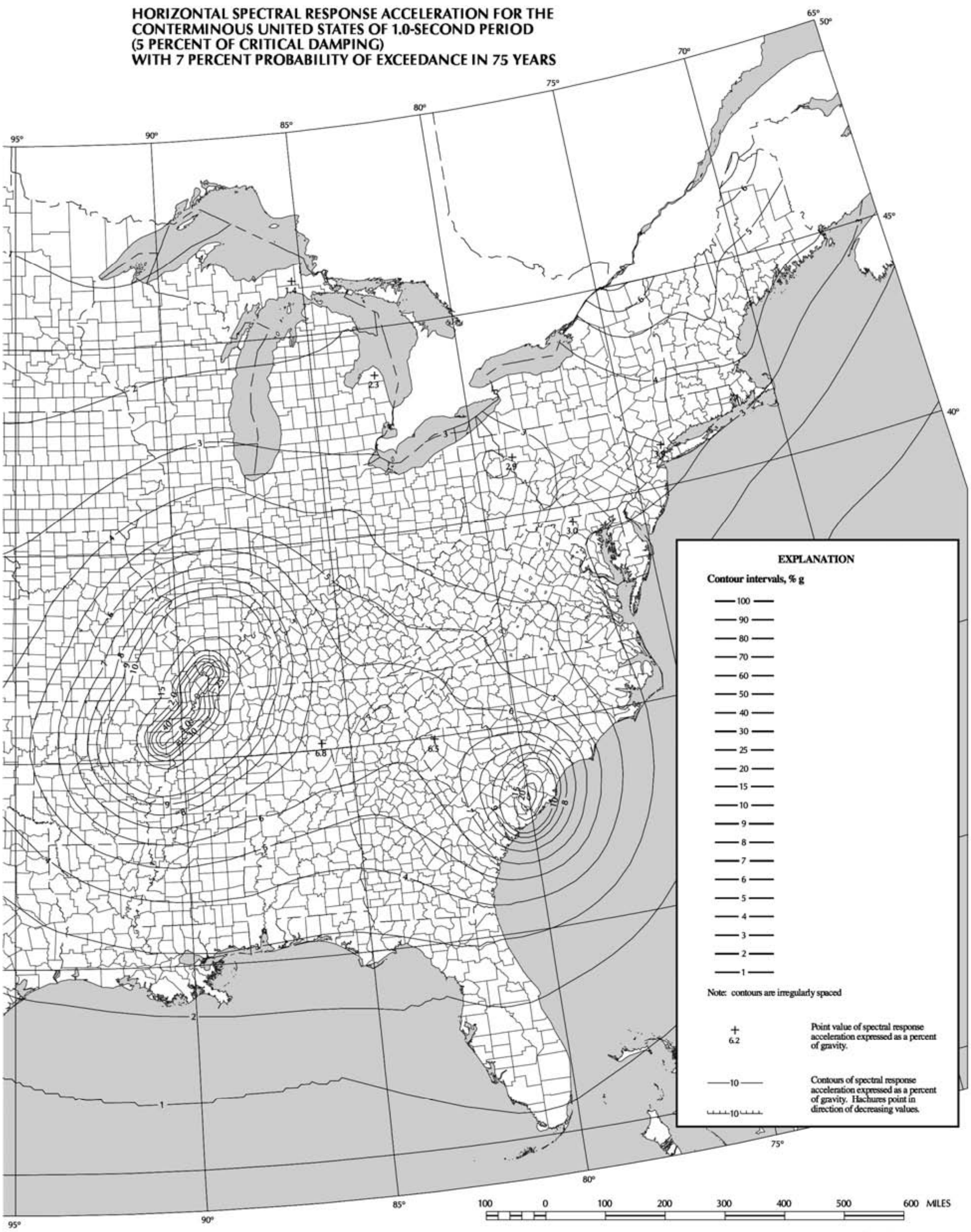


Figure 3.4.1-4b—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 1.0-sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

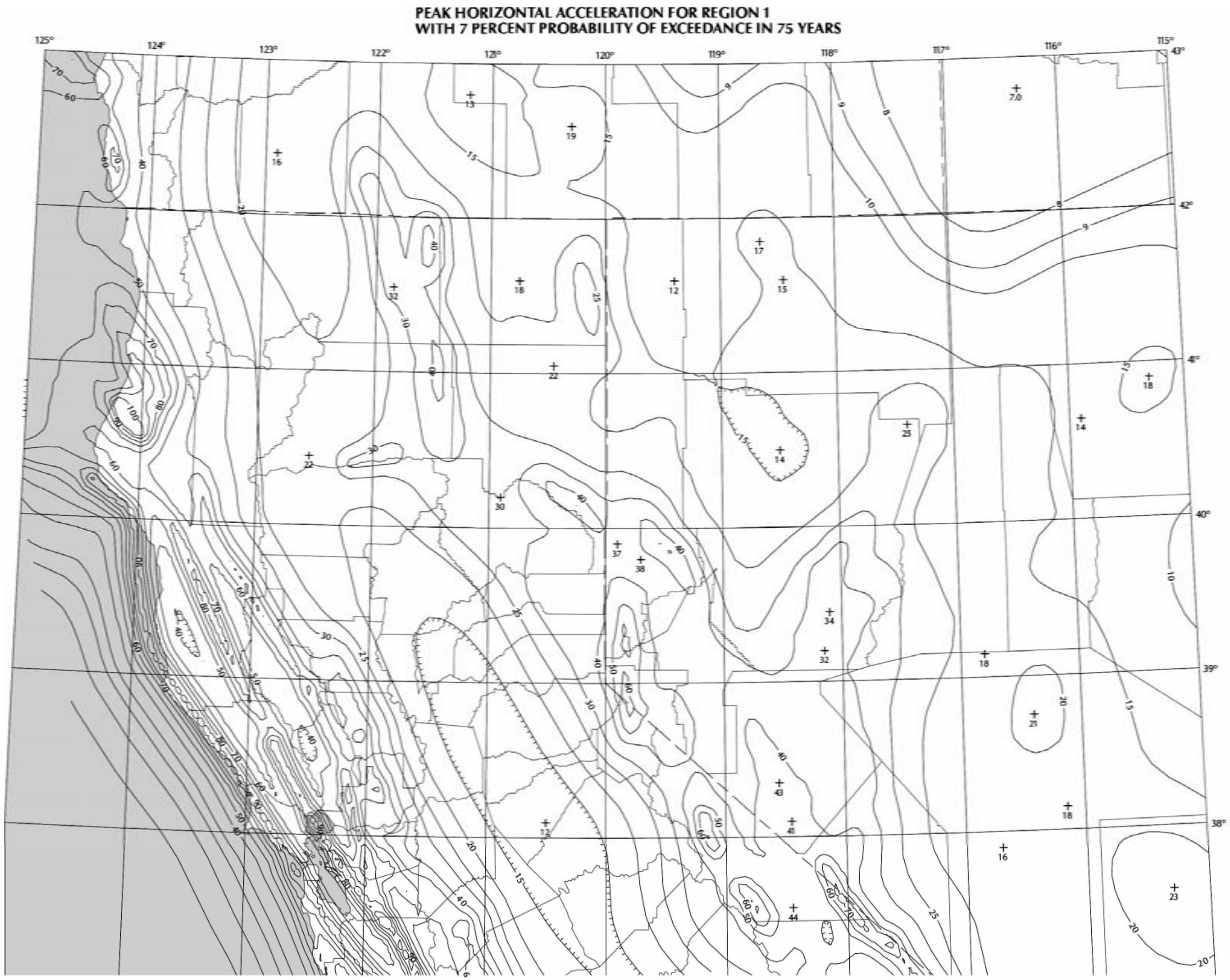


Figure 3.4.1-5a— Horizontal Peak Ground Acceleration Coefficient for Region 1 (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

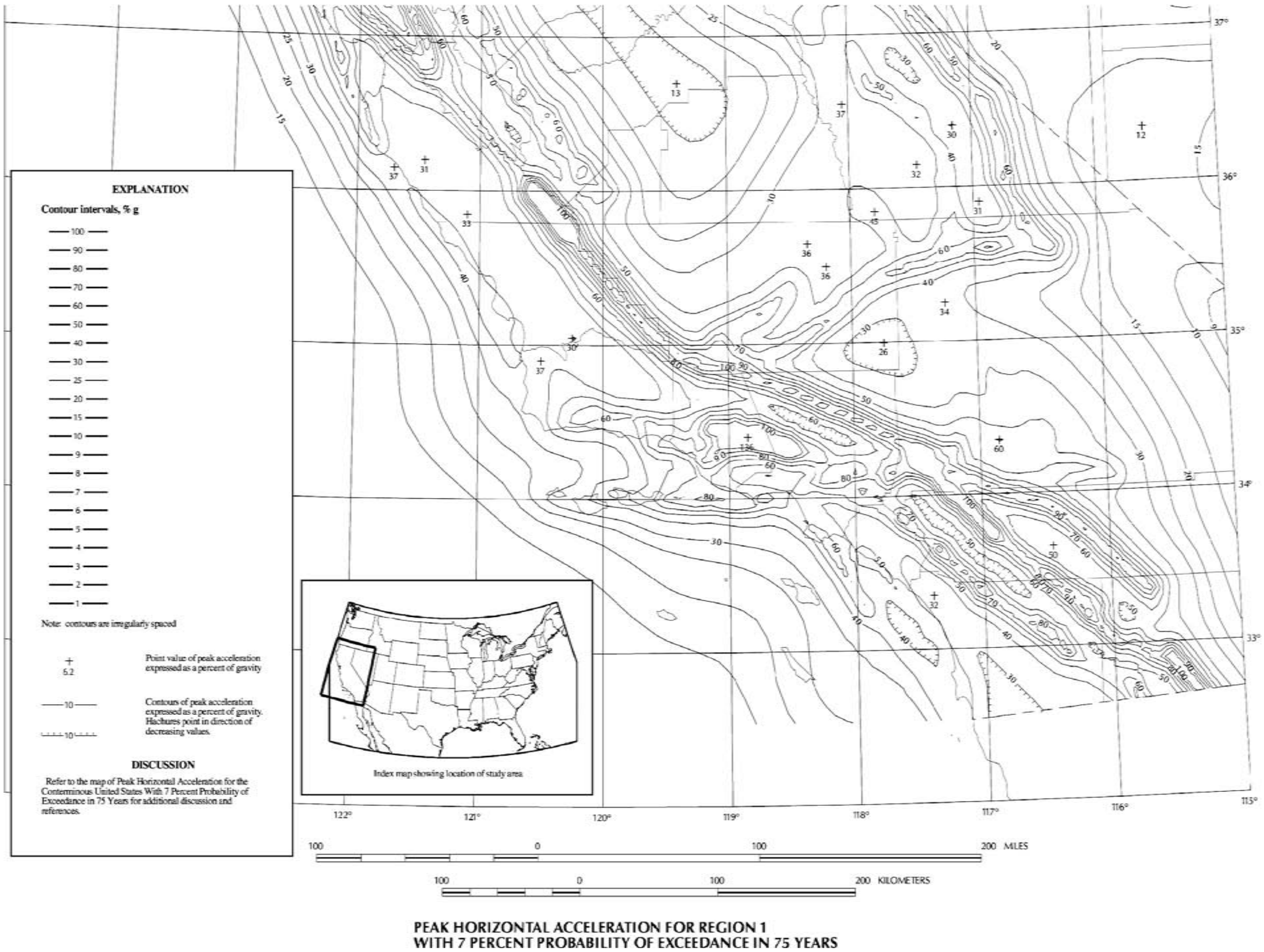


Figure 3.4.1- 5b— Horizontal Peak Ground Acceleration Coefficient for Region 1 (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

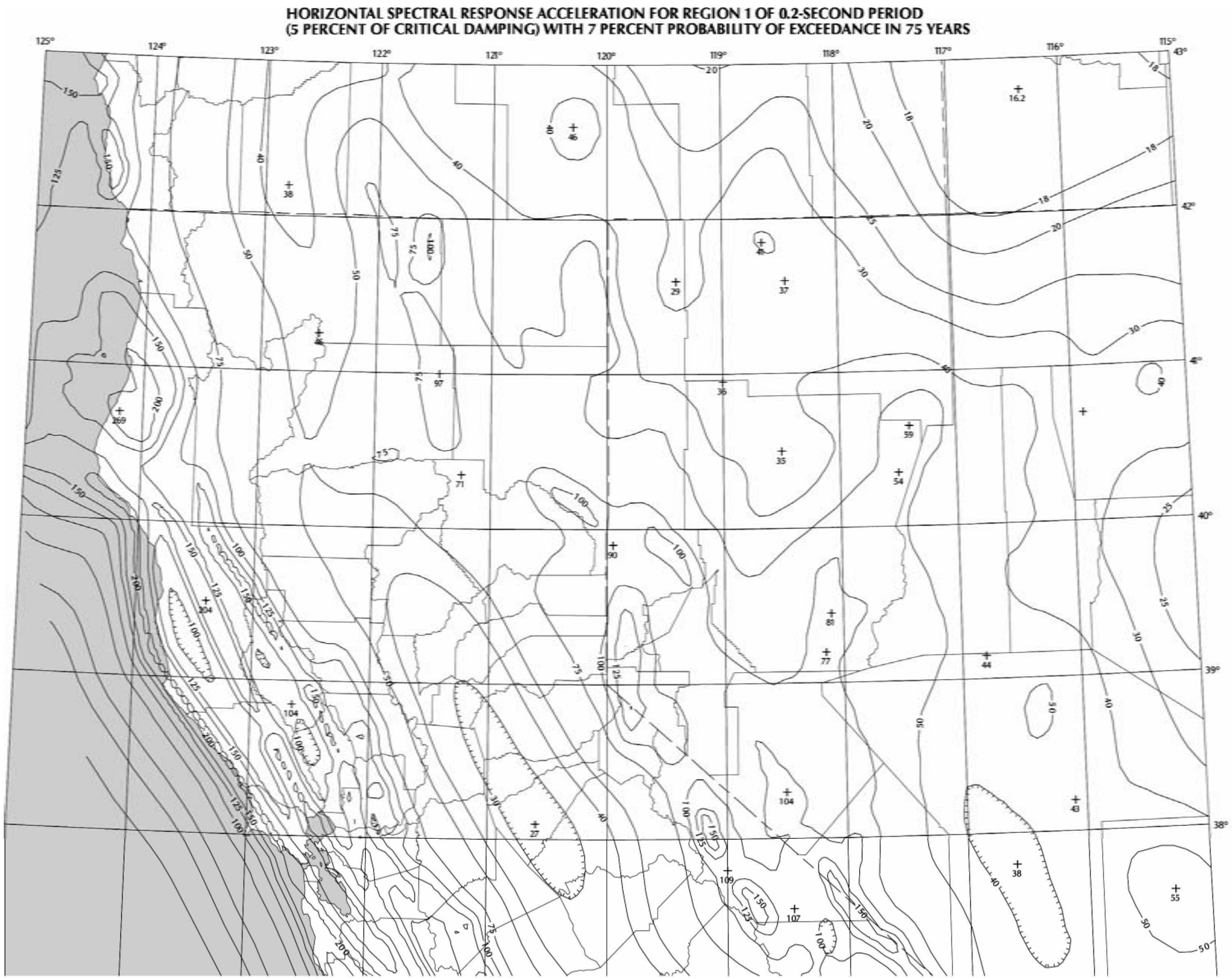


Figure 3.4.1-6a—Horizontal Response Spectral Acceleration Coefficient for Region 1 at Period of 0.2-sec (S_s) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

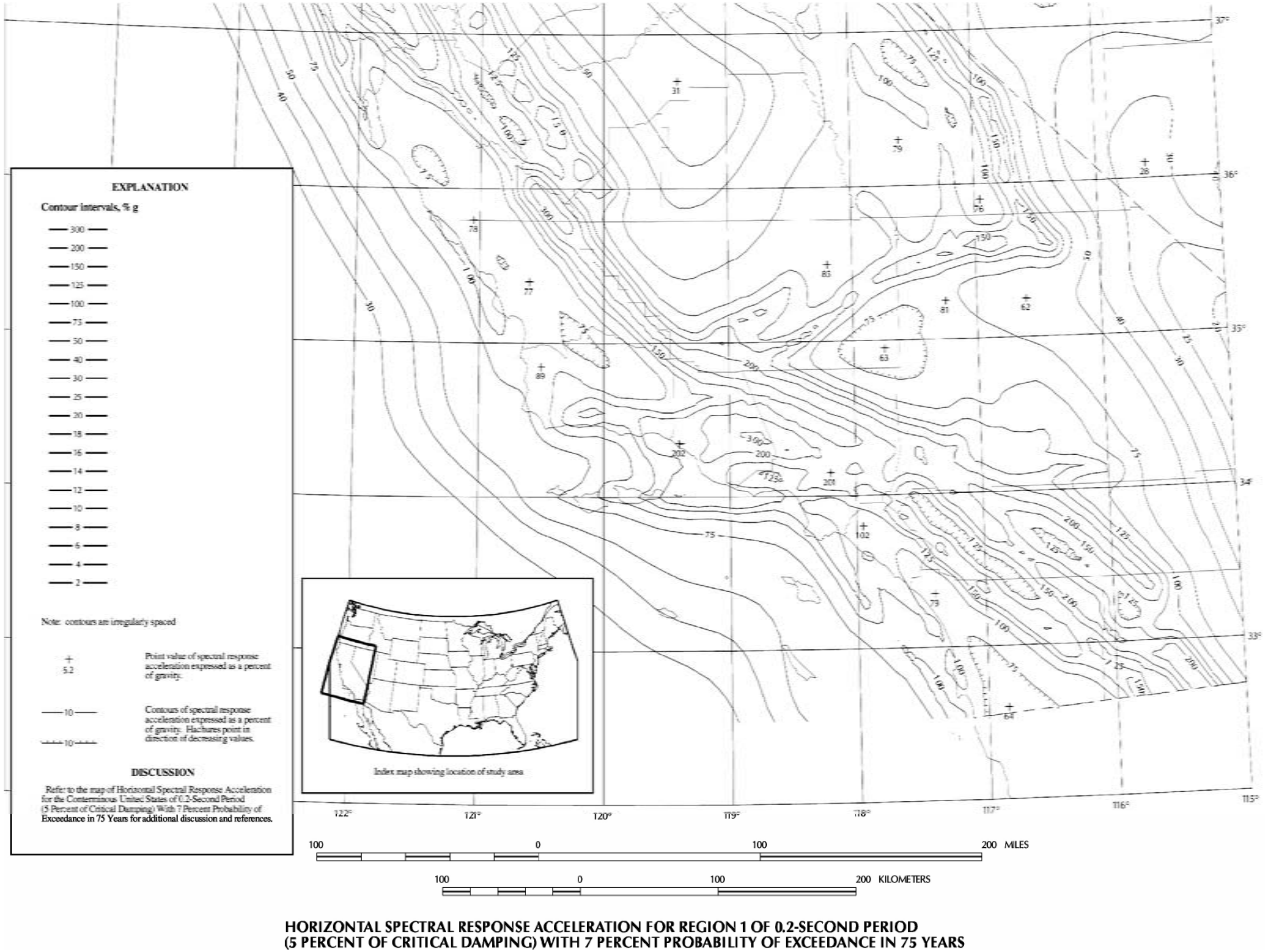


Figure 3.4.1-6b—Horizontal Response Spectral Acceleration Coefficient for Region at Period of 0.2-sec (S_s) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

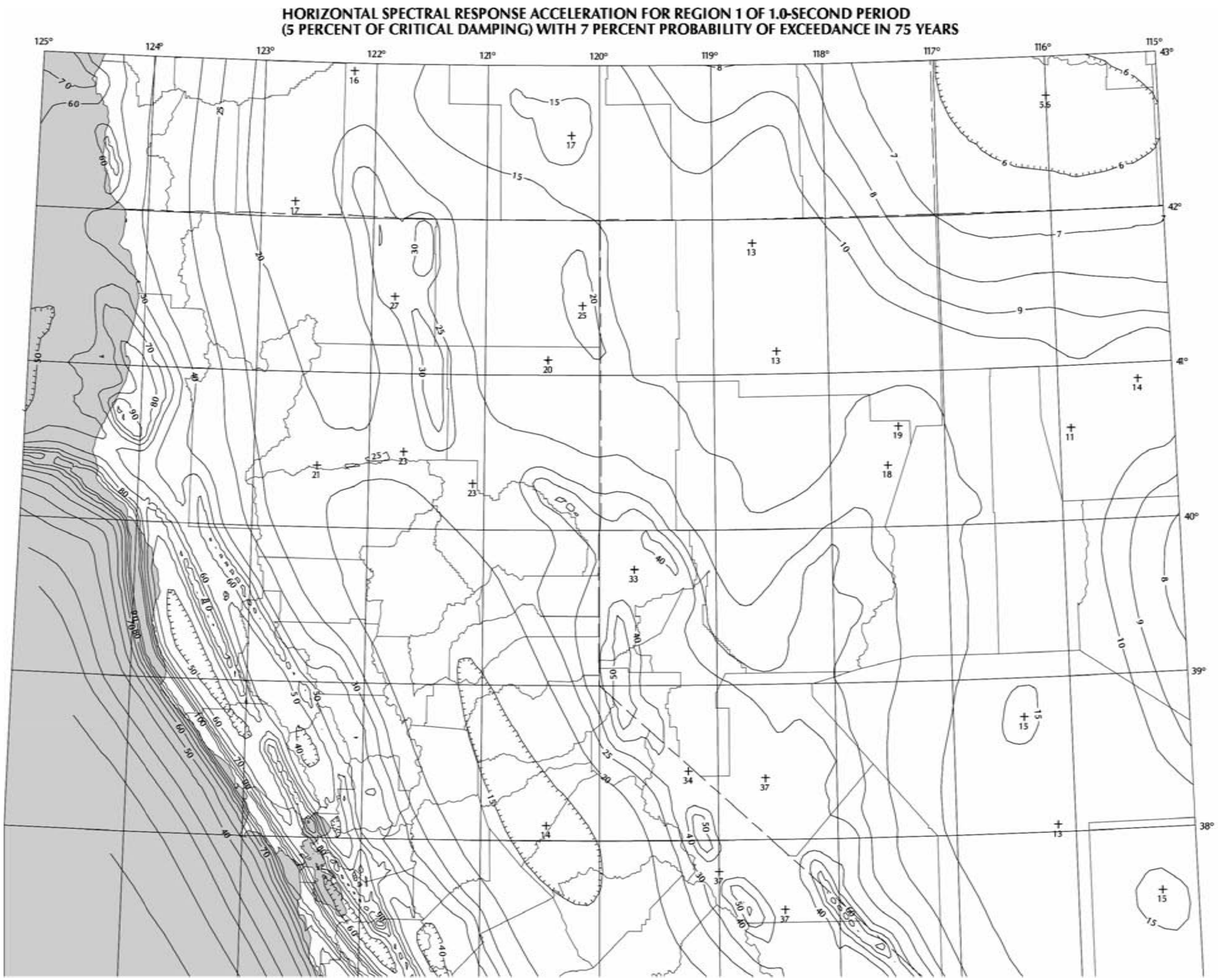


Figure 3.4.1-7a—Horizontal Response Spectral Acceleration Coefficient for Region 1 at Period of 1.0-sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

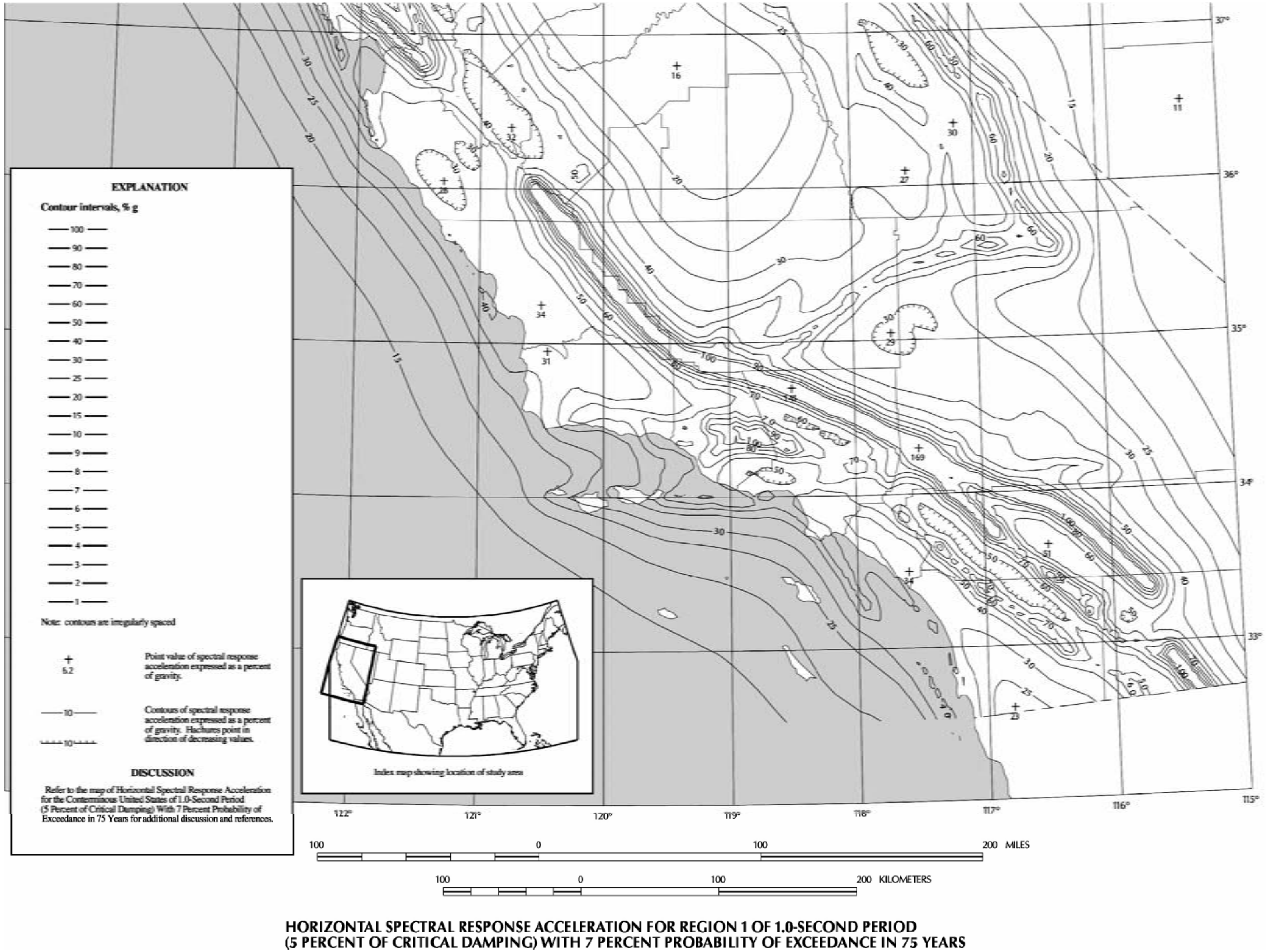


Figure 3.4.1-7b—Horizontal Response Spectral Acceleration Coefficient for Region at Period of 1.0-sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

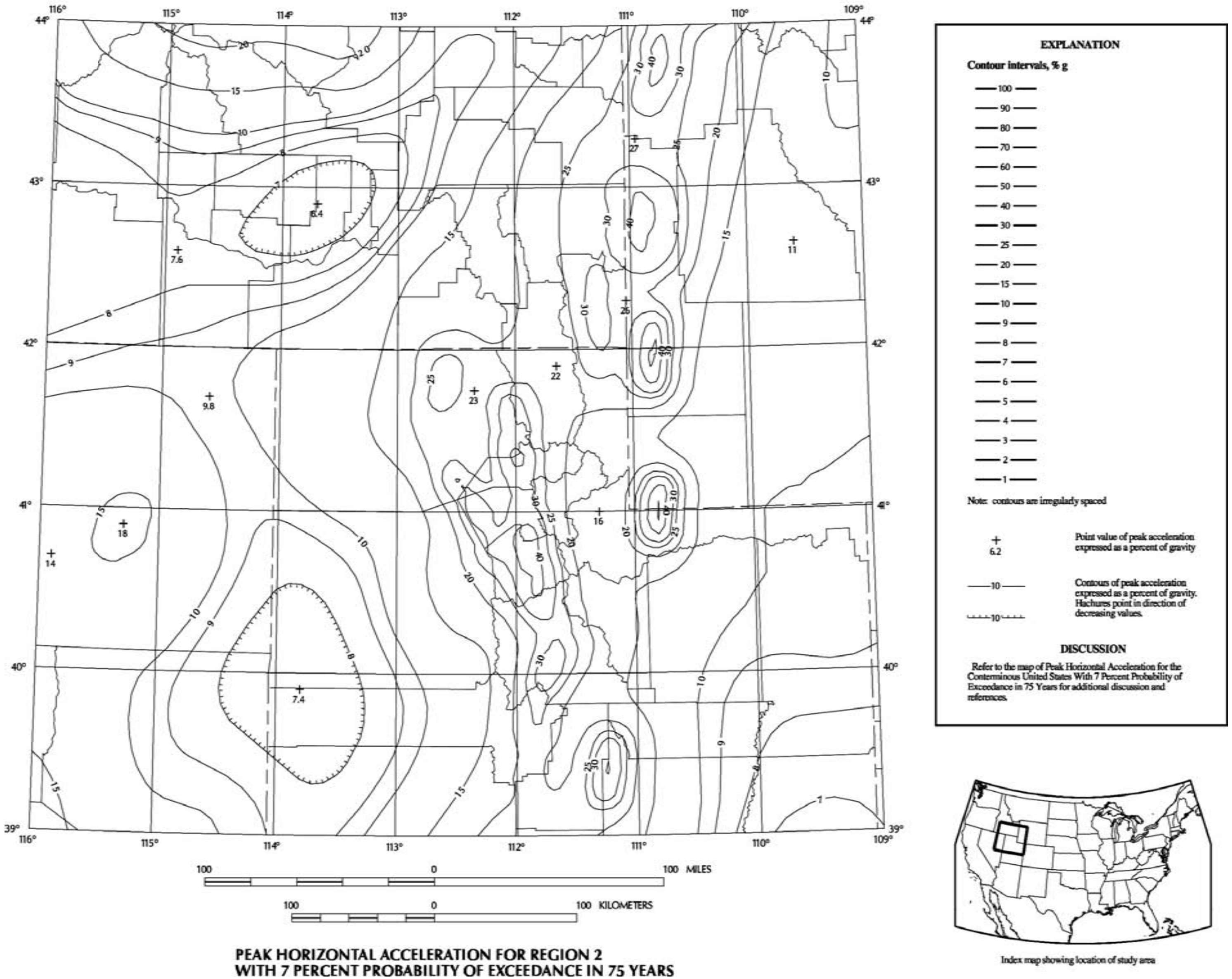


Figure 3.4.1-8—Horizontal Peak Ground Acceleration Coefficient for Region 2 (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

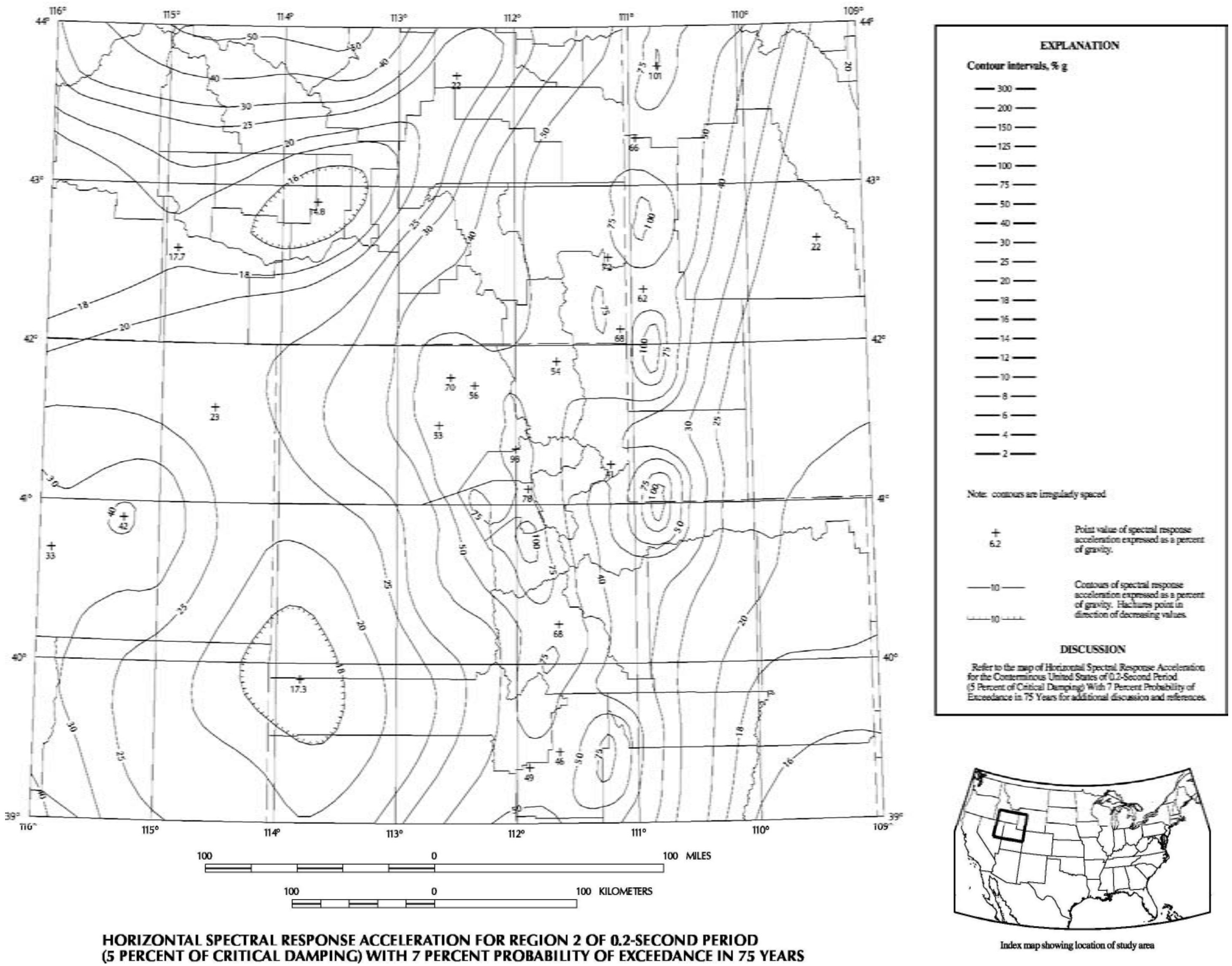


Figure 3.4.1-9—Horizontal Response Spectral Acceleration Coefficient for Region 2 at Period of 0.2-sec (S_s) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

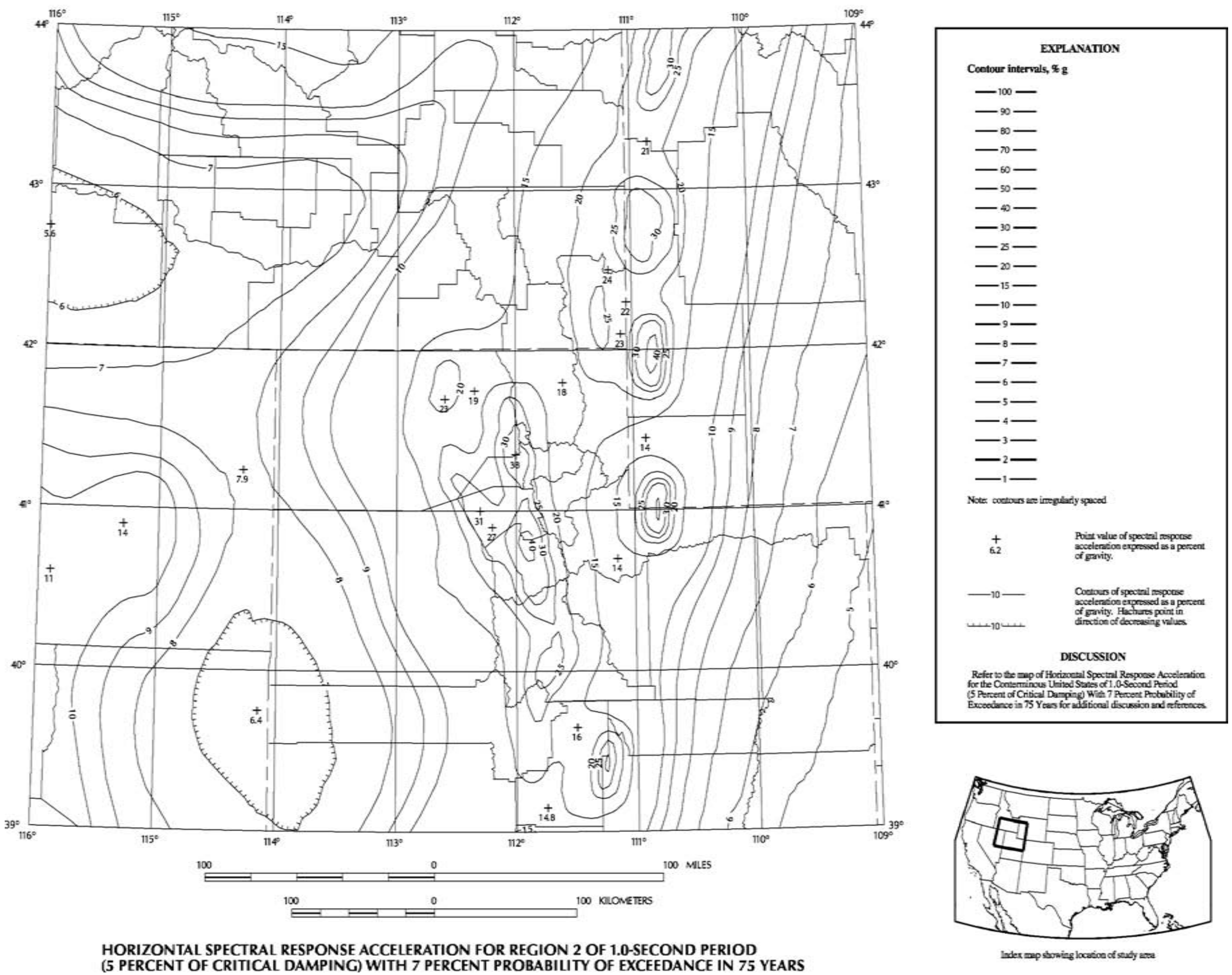
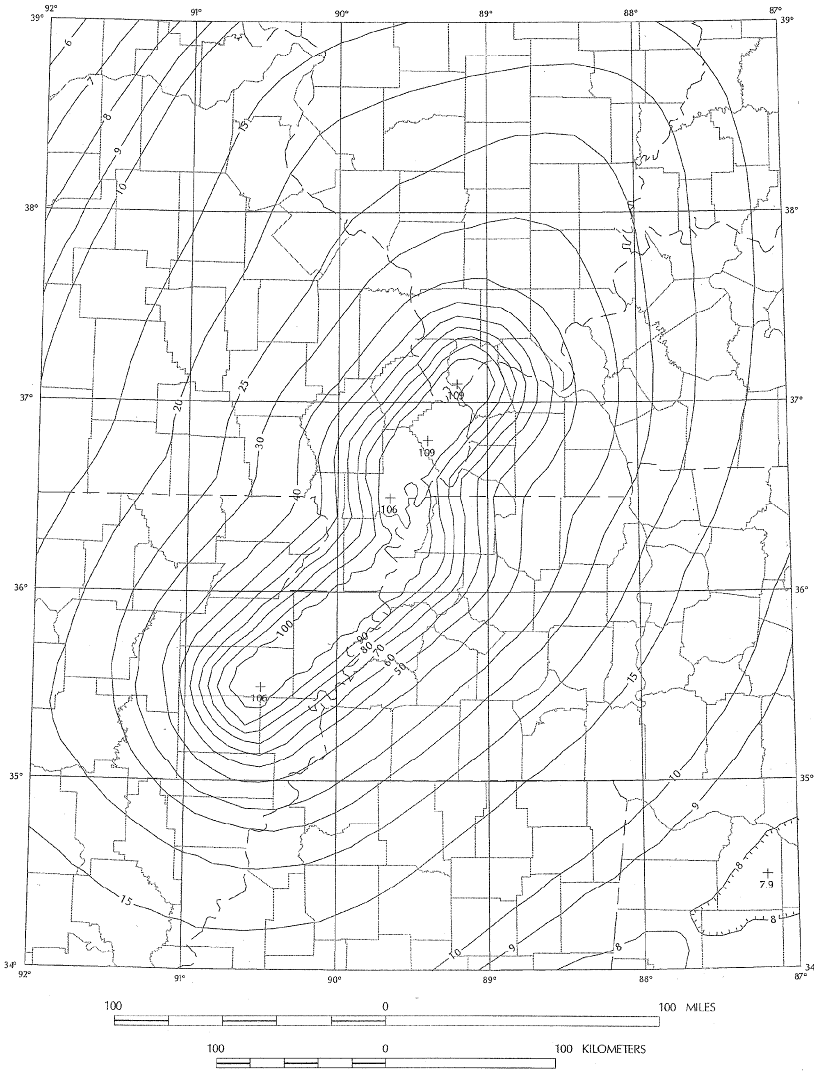


Figure 3.4.1-10—Horizontal Response Spectral Acceleration Coefficient for Region 2 at Period of 1.0-sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping



PEAK HORIZONTAL ACCELERATION FOR REGION 3
WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS

EXPLANATION

Contour intervals, % g

100
90
80
70
60
50
40
30
25
20
15
10
9
8
7
6
5
4
3
2
1

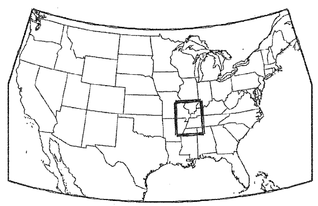
Note: contours are irregularly spaced

+ 6.2 — Point value of peak acceleration expressed as a percent of gravity

—10— Contours of peak acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

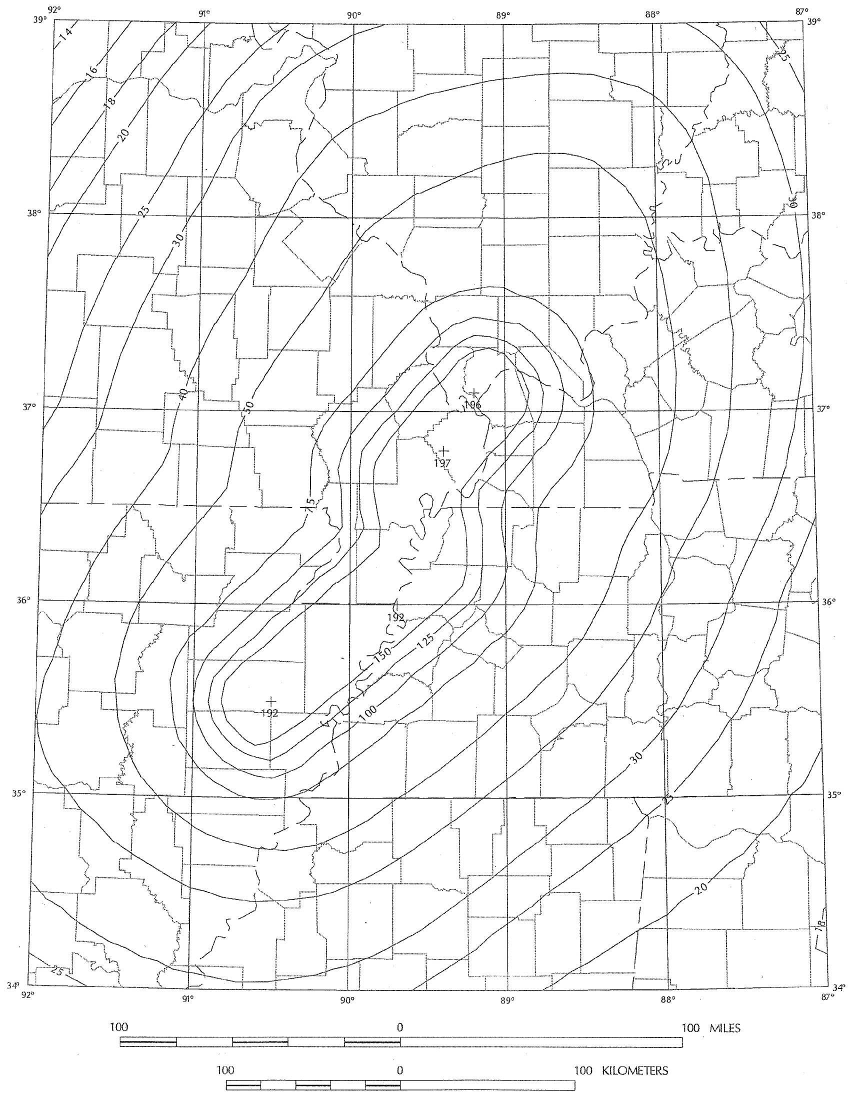
DISCUSSION

Refer to the map of Peak Horizontal Acceleration for the Conterminous United States With 5 Percent Probability of Exceedance in 50 Years for additional discussion and references.



Index map showing location of study area

Figure 3.4.1-11—Horizontal Peak Ground Acceleration Coefficient for Region 3 (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 3 OF 0.2-SECOND PERIOD
(5 PERCENT OF CRITICAL DAMPING) WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS

EXPLANATION

Contour intervals, % g

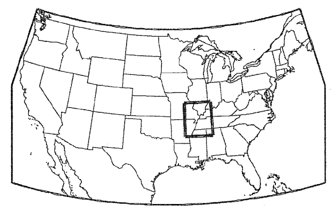
300
200
150
125
100
75
50
40
30
25
20
18
16
14
12
10
8
6
4
2

Note: contours are irregularly spaced

+ 6.2	Point value of spectral response acceleration expressed as a percent of gravity.
—10—	Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

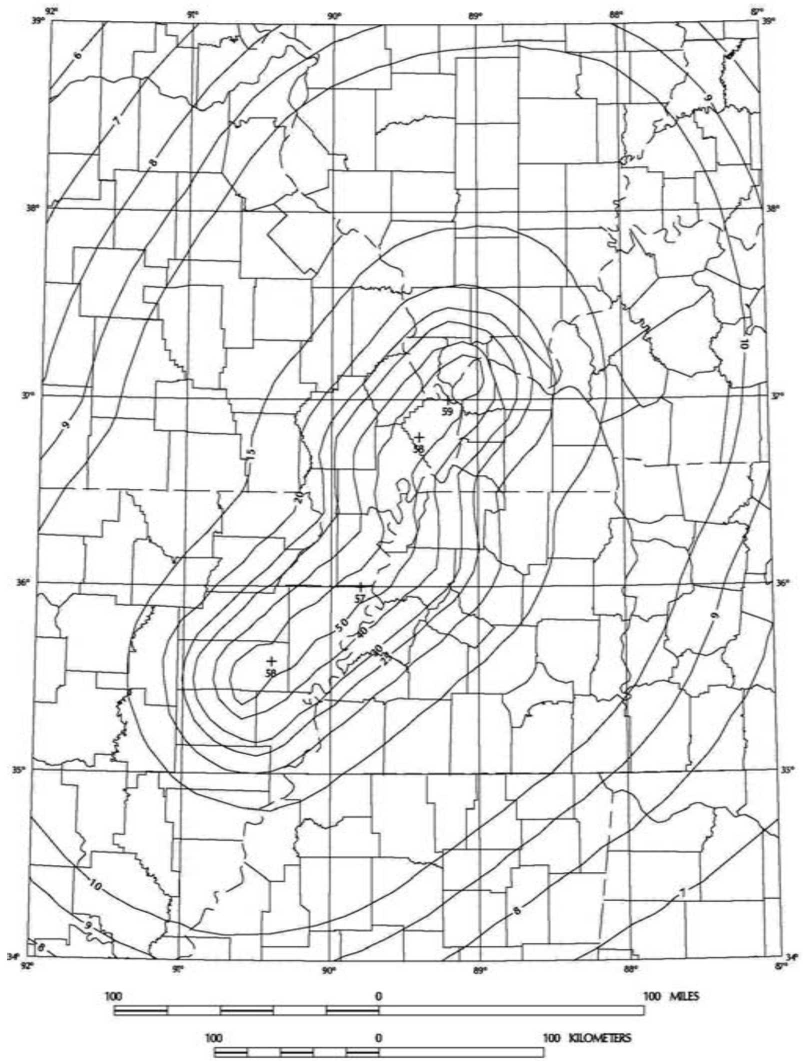
DISCUSSION

Refer to the map of Horizontal Spectral Response Acceleration for the Conterminous United States of 0.2-Second Period (5 Percent of Critical Damping) With 5 Percent Probability of Exceedance in 50 Years for additional discussion and references.



Index map showing location of study area

Figure 3.4.1-12—Horizontal Response Spectral Acceleration Coefficient for Region 3 at Period of 0.2-sec (S_s) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 3 OF 1.0-SECOND PERIOD
(5 PERCENT OF CRITICAL DAMPING) WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS

EXPLANATION

Contour intervals, % g

—100—
—90—
—80—
—70—
—60—
—50—
—40—
—30—
—25—
—20—
—15—
—10—
—9—
—8—
—7—
—6—
—5—
—4—
—3—
—2—
—1—

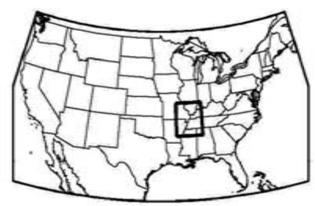
Note: contours are irregularly spaced

+ 6.2 — Point value of spectral response acceleration expressed as a percent of gravity.

—10— — Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

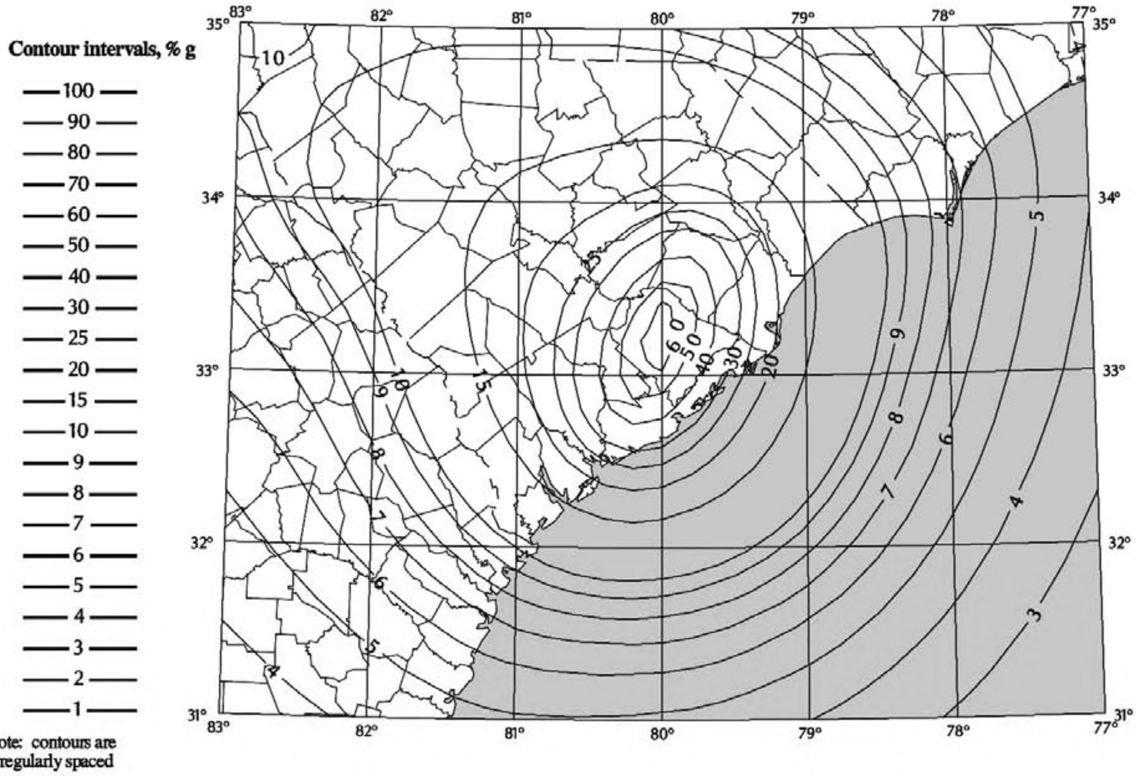
Refer to the map of Horizontal Spectral Response Acceleration for the Conterminous United States of 1.0-Second Period (5 Percent of Critical Damping) With 7 Percent Probability of Exceedance in 75 Years for additional discussion and references.

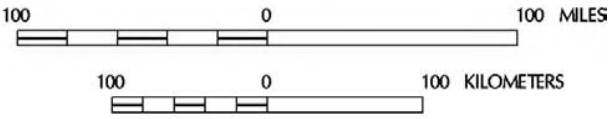


Index map showing location of study area

Figure 3.4.1-13—Horizontal Response Spectral Acceleration Coefficient for Region 3 at Period of 1.0-sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

PEAK HORIZONTAL ACCELERATION FOR REGION 4 WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS





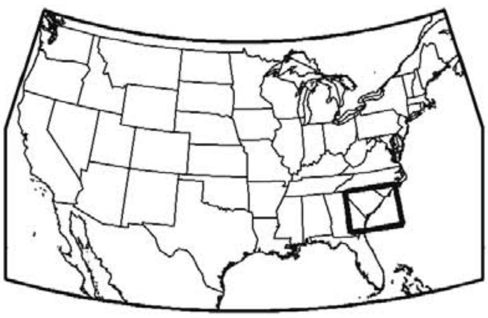
EXPLANATION

+ 6.2 — Point value of peak acceleration expressed as a percent of gravity

10 — Contours of peak acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

Refer to the map of Peak Horizontal Acceleration for the Conterminous United States With 7 Percent Probability of Exceedance in 75 Years for additional discussion and references.



Index map showing location of study area

Figure 3.4.1-14—Horizontal Peak Ground Acceleration Coefficient for Region 4 (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

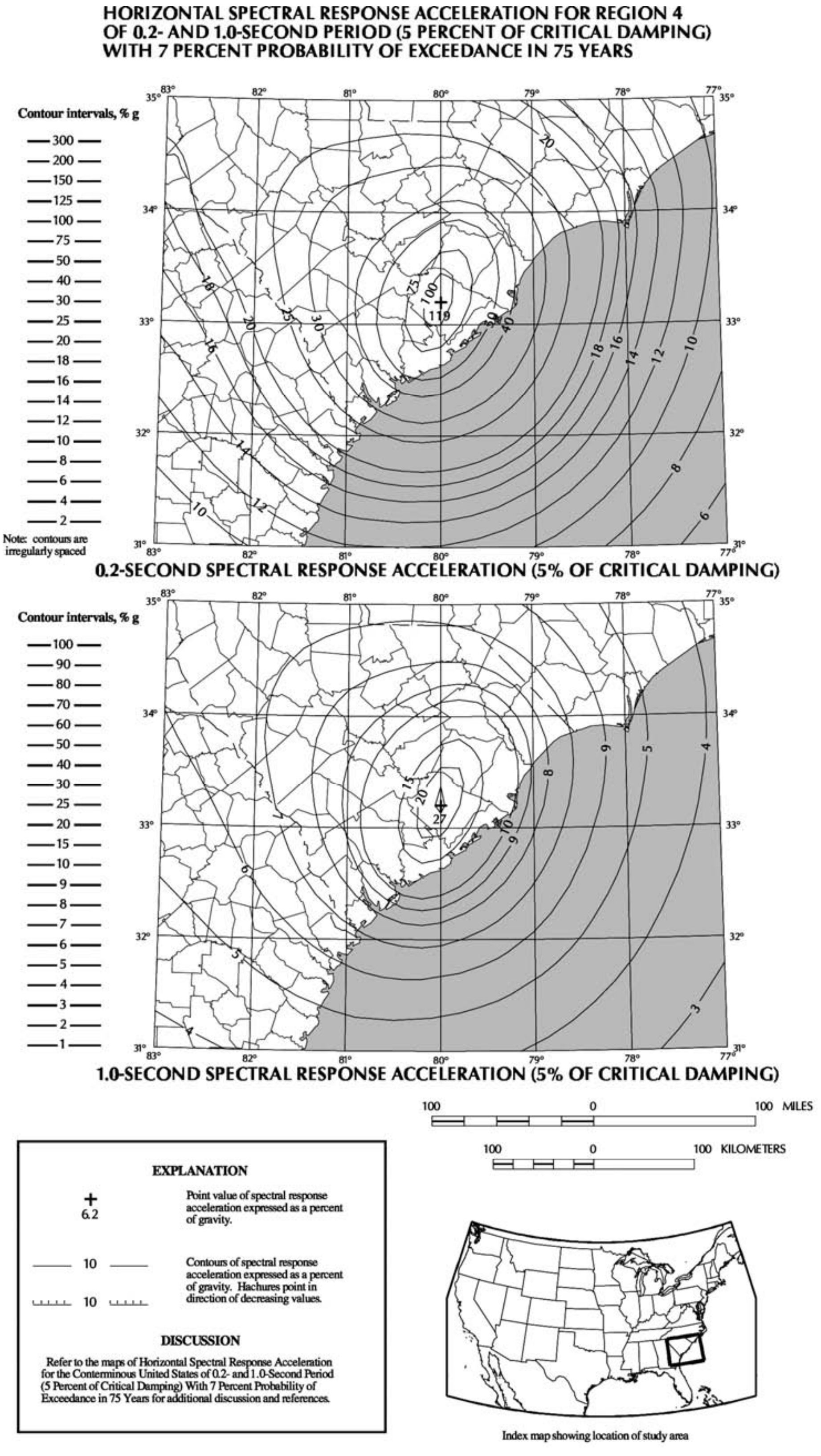


Figure 3.4.1-15—Horizontal Response Spectral Acceleration Coefficients for Region 4 at Periods of 0.2 (S_s) and 1.0 sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

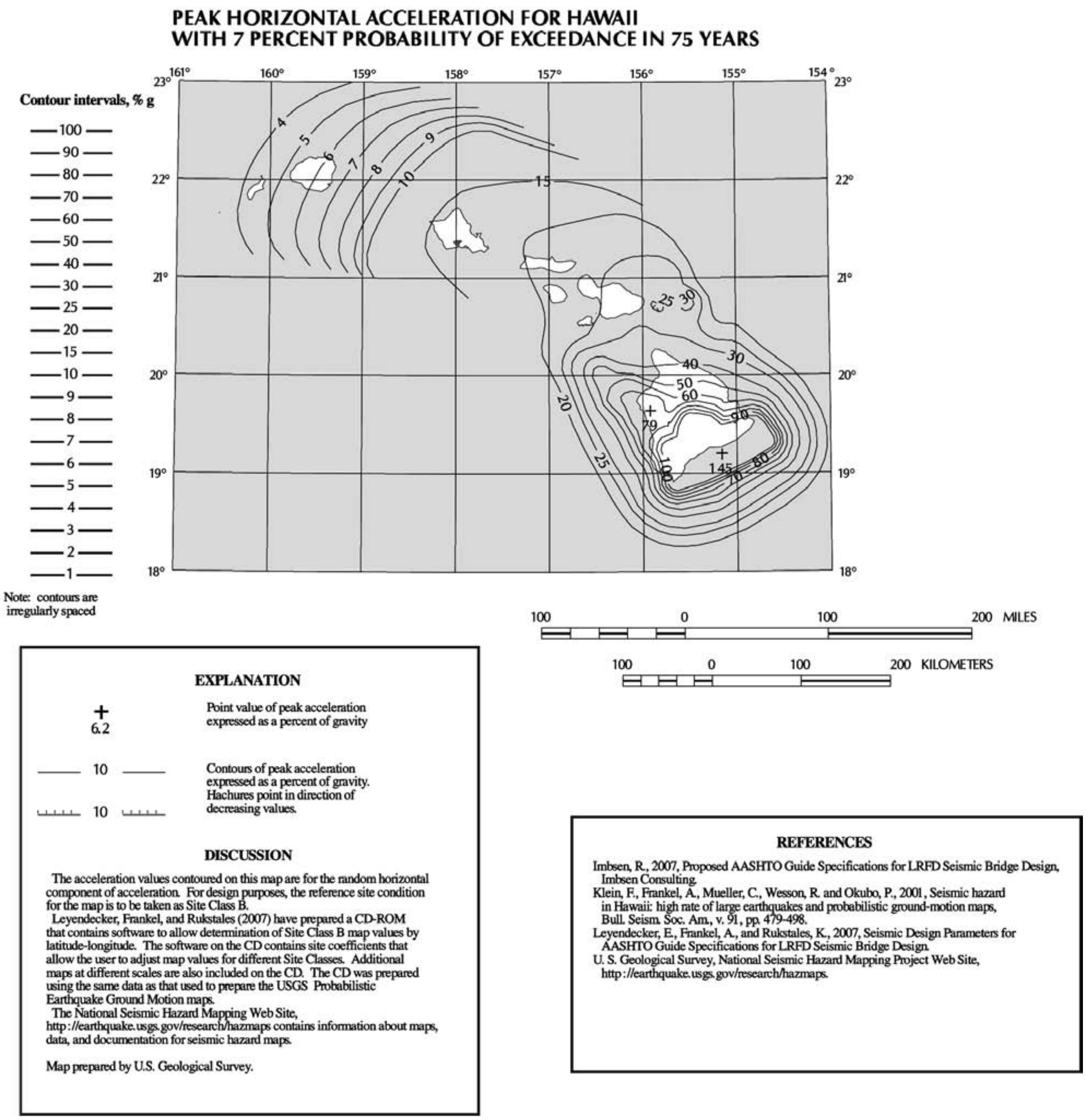
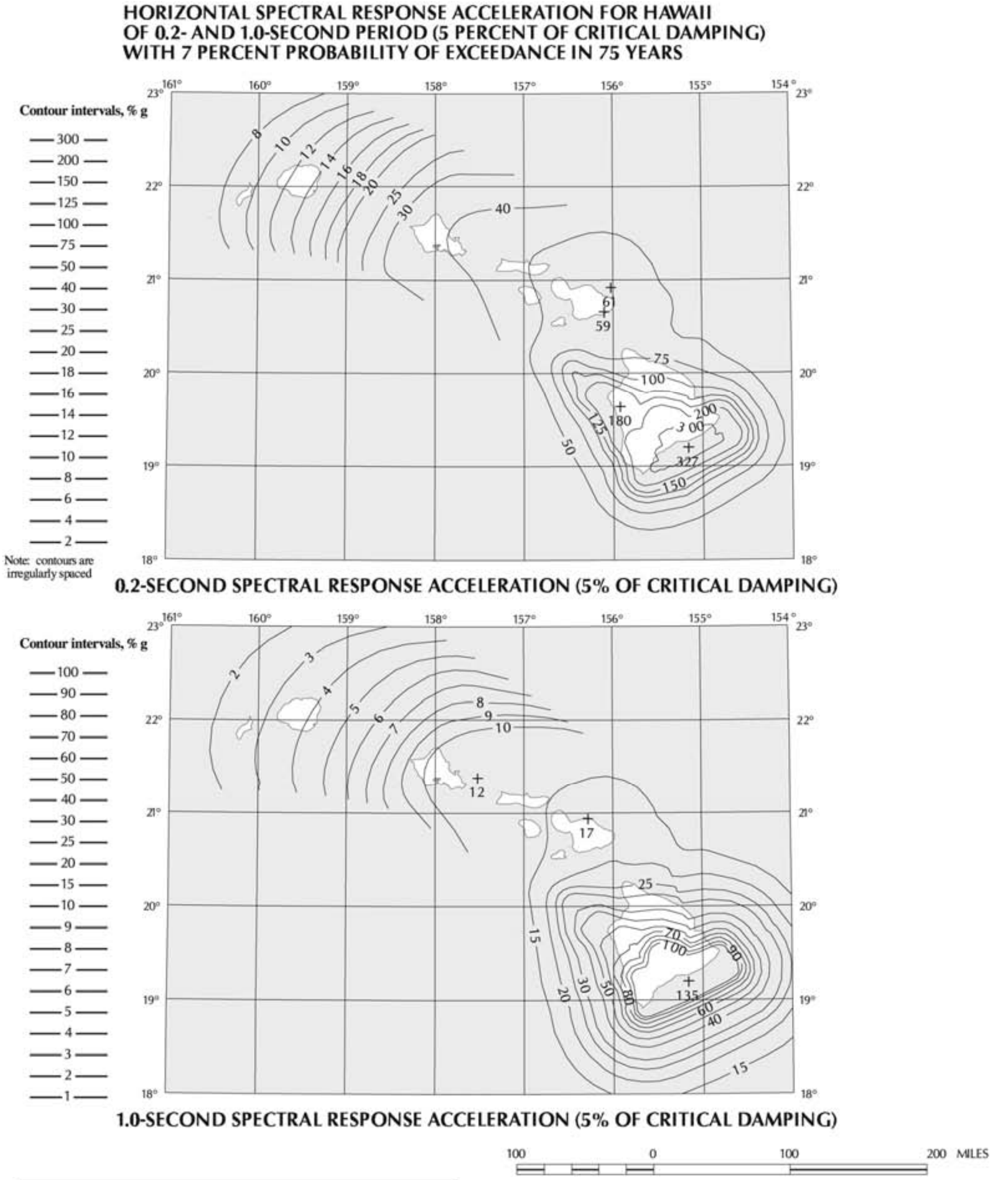


Figure 3.4.1-16—Horizontal Peak Ground Acceleration Coefficient for Hawaii (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)



EXPLANATION

+ 6.2	Point value of spectral response acceleration expressed as a percent of gravity.
10	Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

The acceleration values contoured on this map are for the random horizontal component of acceleration. For design purposes, the reference site condition for the map is to be taken as Site Class B.

Leyendecker, Frankel, and Rukstales (2007) have prepared a CD-ROM that contains software to allow determination of Site Class B map values by latitude-longitude. The software on the CD contains site coefficients that allow the user to adjust map values for different Site Classes. Additional maps at different scales are also included on the CD. The CD was prepared using the same data as that used to prepare the USGS Probabilistic Earthquake Ground Motion maps.

The National Seismic Hazard Mapping Web Site, http://earthquake.usgs.gov/research/hazmaps contains information about maps, data, and documentation for seismic hazard maps.

Map prepared by U.S. Geological Survey.

REFERENCES

Imbsen, R., 2007, Proposed AASHTO Guide Specifications for LRFD Seismic Bridge Design, Imbsen Consulting.

Klein, F., Frankel, A., Mueller, C., Wesson, R. and Okubo, P., 2001, Seismic hazard in Hawaii: high rate of large earthquakes and probabilistic ground-motion maps, Bull. Seism. Soc. Am., v. 91, pp. 479-498.

Leyendecker, E., Frankel, A., and Rukstales, K., 2007, Seismic Design Parameters for AASHTO Guide Specifications for LRFD Seismic Bridge Design.

U. S. Geological Survey, National Seismic Hazard Mapping Project Web Site, http://earthquake.usgs.gov/research/hazmaps.

Figure 3.4.1-17—Horizontal Response Spectral Acceleration Coefficient for Hawaii at Periods of 0.2 (S_s) and 1.0 sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

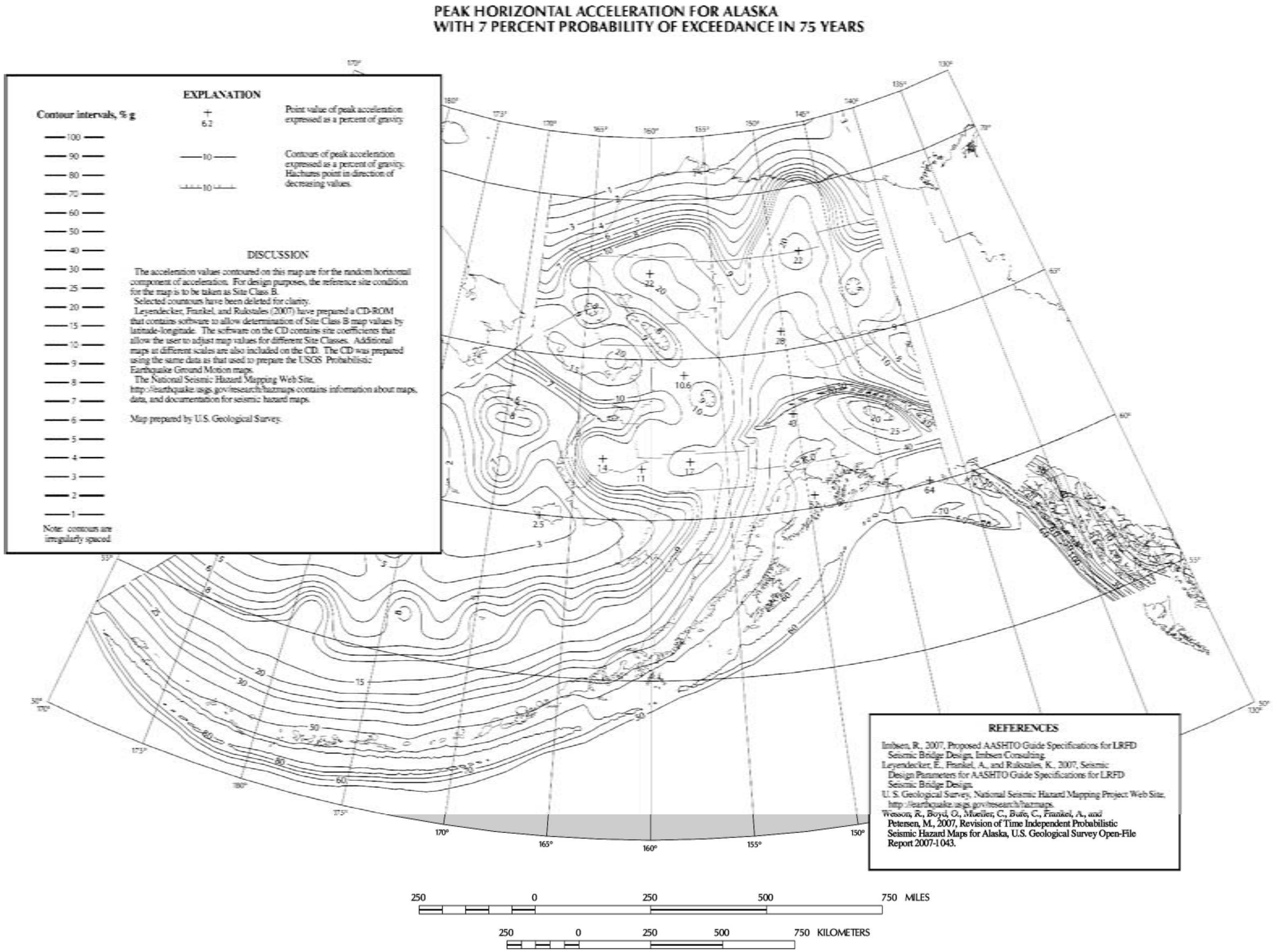


Figure 3.4.1-18—Horizontal Peak Ground Acceleration Coefficient for Alaska with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

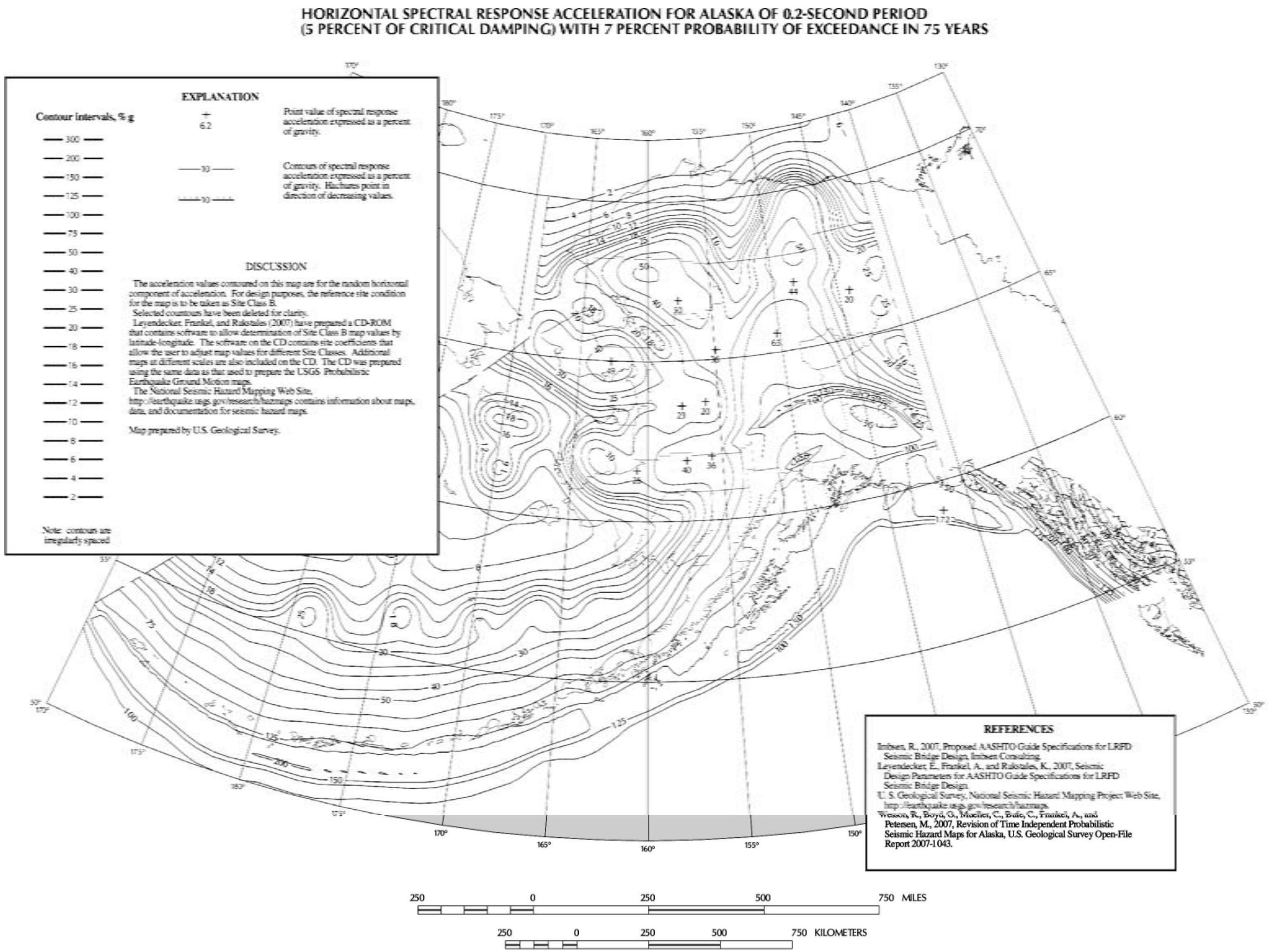


Figure 3.4.1-19—Horizontal Response Spectral Acceleration Coefficient for Alaska at Period of 0.2-sec (S_s) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

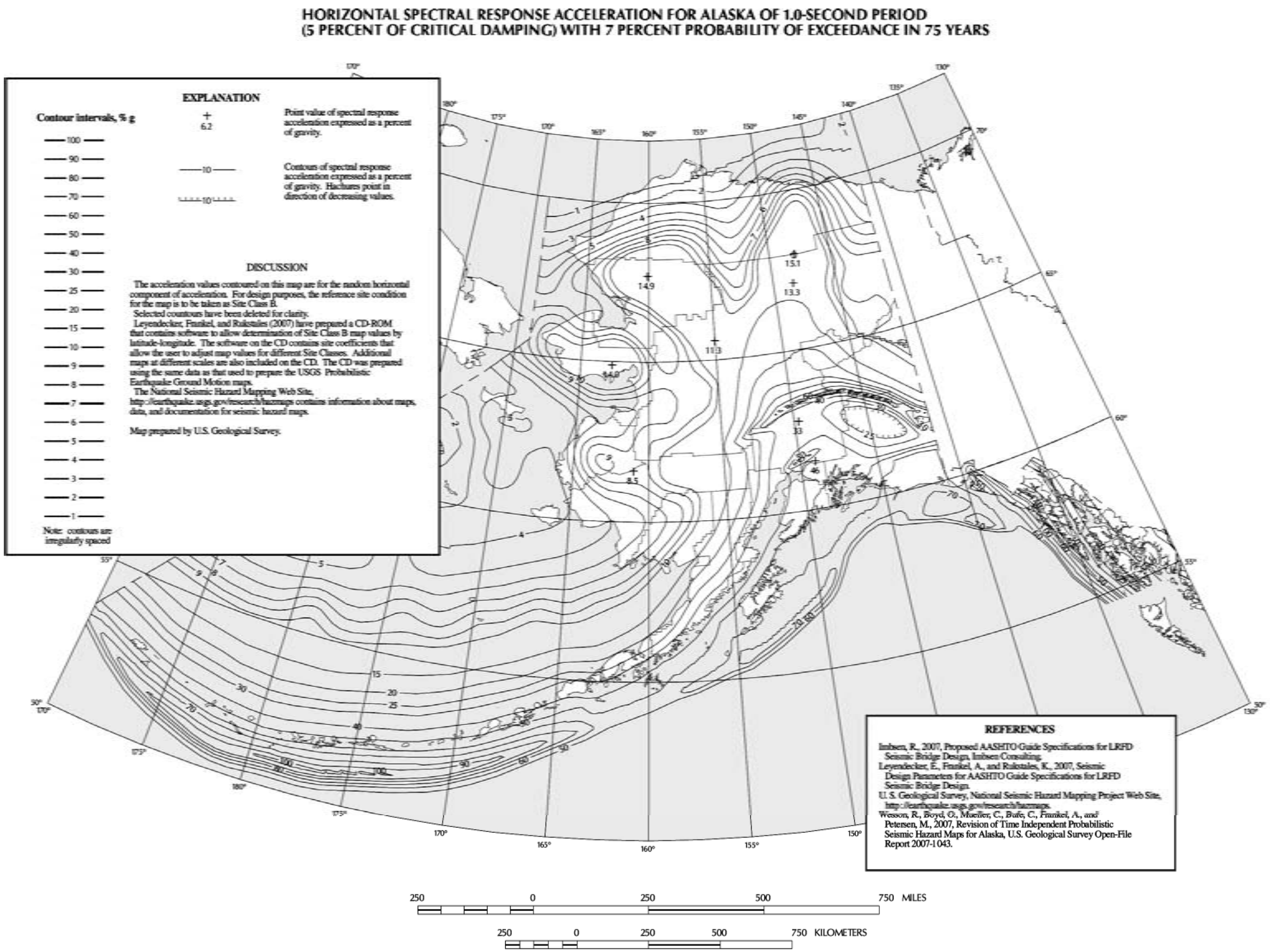


Figure 3.4.1-20—Horizontal Response Spectral Acceleration Coefficient for Alaska at Period of 1.0-sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

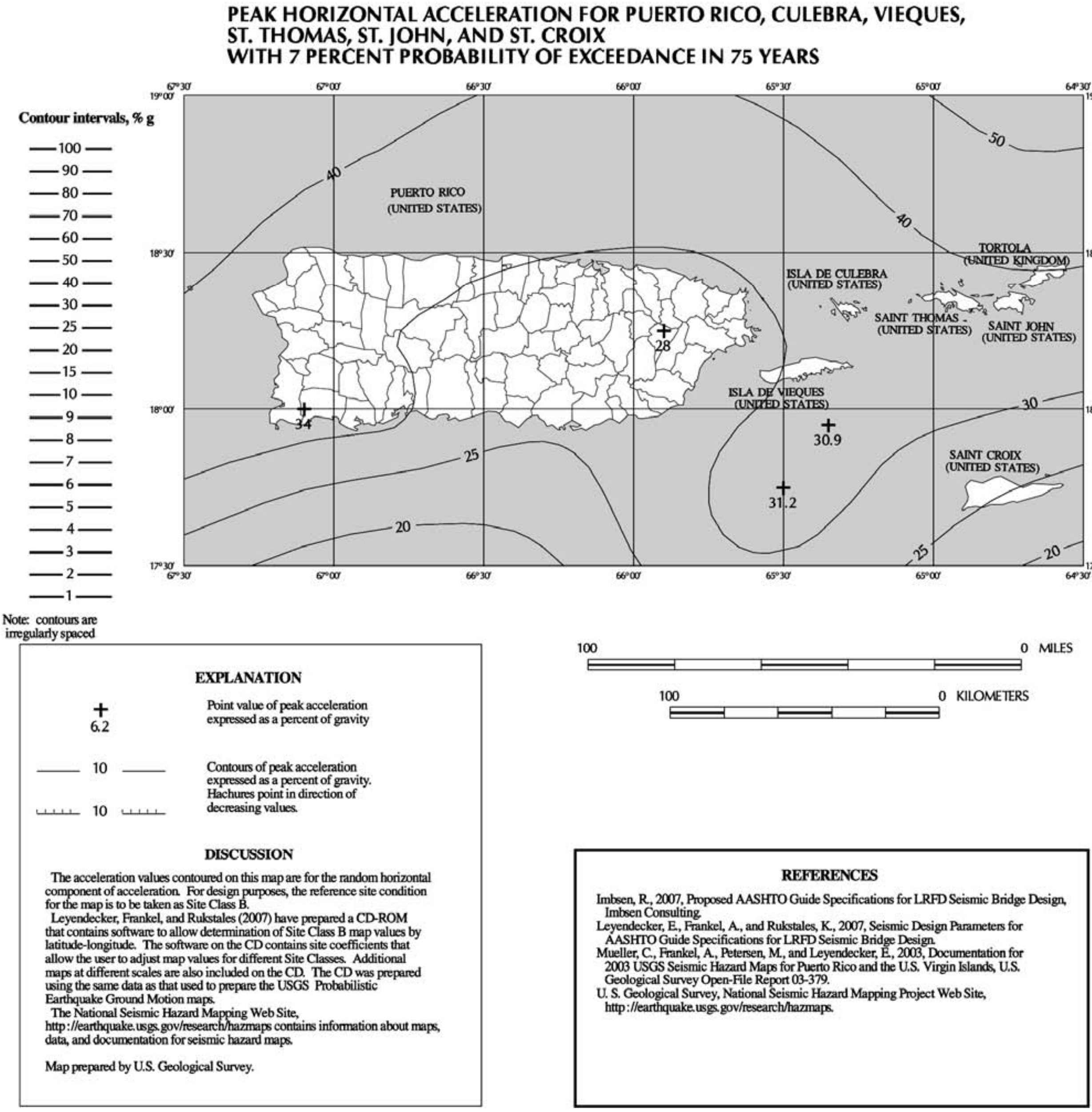


Figure 3.4.1-21—Horizontal Peak Ground Acceleration Coefficient for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

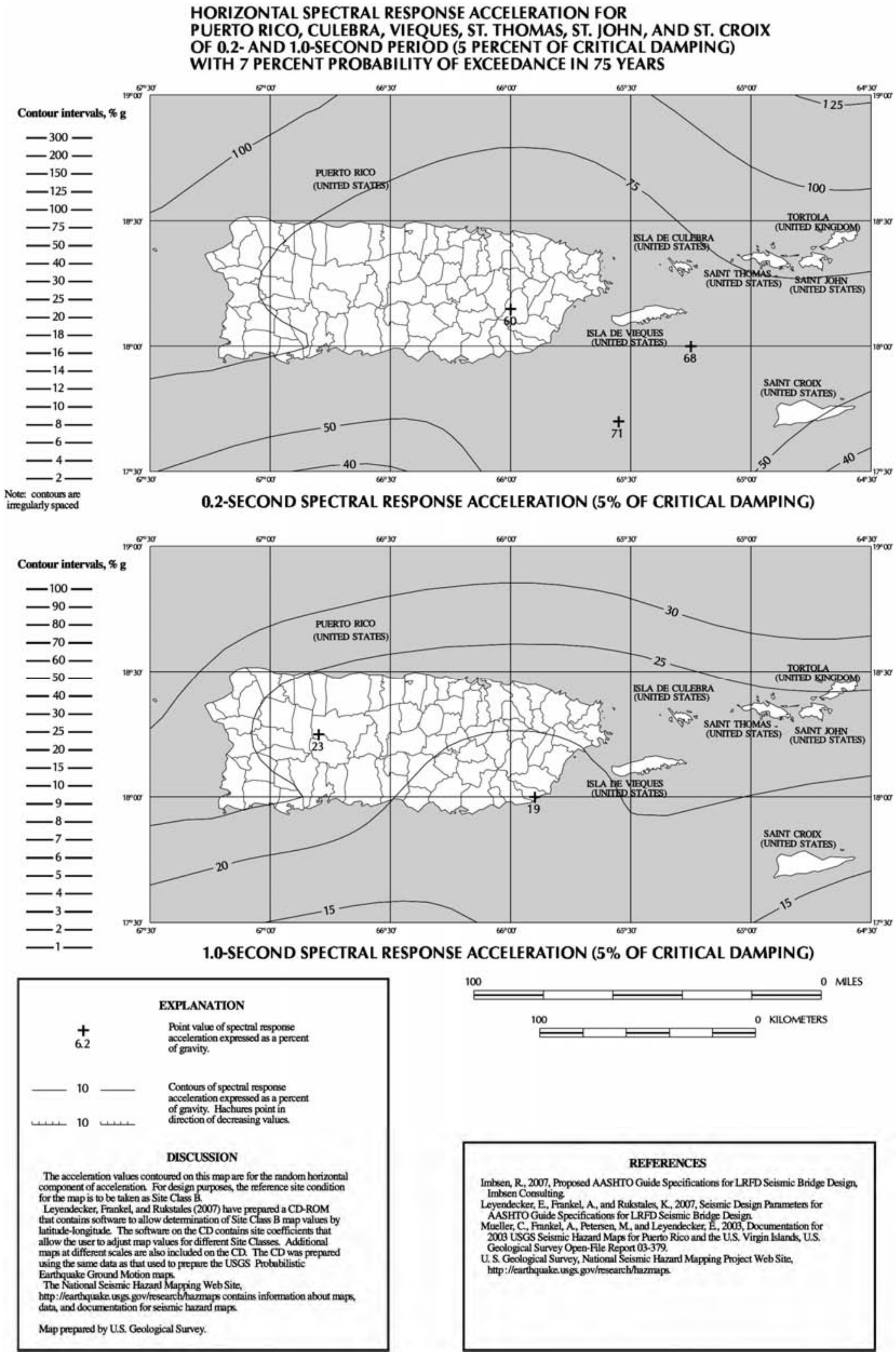


Figure 3.4.1-22—Horizontal Response Spectral Acceleration Coefficients for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix at Periods of 0.2 (S_s) and 1.0 sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

3.4.2—Site Effects on Ground Motions

The generalized site classes and site factors described in this Article shall be used with the general procedure for constructing response spectra described in Article 3.4.1. Site-specific analysis of soil response effects shall be conducted where required by Article 3.4 and in accordance with the requirements in Article 3.4.3 and Table 3.4.2.1-1—Site Class Definitions.

If geological conditions at the abutments and intermediate piers result in different soil classification, then the site factors used to develop the design response spectrum may be determined based upon the site-specific procedures outlined in Article 3.4.3. In lieu of the site-specific procedures and under guidance from the geotechnical engineer, the design response spectrum should be determined by constructing a response spectrum for individual abutments, piers, or groups of piers and then developing a single spectrum based on the higher spectral acceleration coefficient at each period, i.e., an envelope of the spectra.

C3.4.2

The behavior of a bridge during an earthquake is strongly related to the soil conditions at the site. Soils can amplify or deamplify ground motions originating in the underlying rock. The amount of amplification can be a factor of two or more. The extent of amplification or deamplification is dependent on the profile of the soil types at the site and the intensity of shaking in the rock below. Sites are classified by types and profile for the purposes of defining the overall seismic hazard, which is quantified as the product of soil amplification or deamplification and intensity of shaking in the underlying rock.

The site classes and site factors described in this Article were originally recommended at a site response workshop in 1992 (Martin, ed., 1994). Subsequently, they were adopted in the seismic design criteria of Caltrans, the 1994 and 1997 editions of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 1995, 1998), the 1997 *Uniform Building Code* (ICBO, 1997), and subsequently the *International Building Codes* (ICC, 2000, 2003, and 2006). The bases for the adopted site classes and site factors is described by Martin and Dobry (1994) and Rinne (1994).

Procedures described in this Article were originally developed for computing ground motions at the ground surface for relatively uniform site conditions. Depending on the site classification and the level of the ground motion, the motion at the surface will likely be different from the motion at depth. This creates some question as to the location of the motion to use in the bridge design. It is also possible that the soil conditions at the two abutments are different or they differ at the abutments and interior piers. An example would be where one abutment is on firm ground or rock and the other is on a loose fill. These variations are not always easily handled by simplified procedures described in this commentary. For critical bridges, it may be necessary to use more rigorous numerical modeling to represent these conditions. The decision to use more rigorous numerical modeling should be made after detailed discussion of the benefits and limitations of more rigorous modeling between the Bridge and Geotechnical Engineers and the Owner.

3.4.2.1—Site Class Definitions

The site shall be classified as one of the following classes given in Table 1. Procedures given in Article 3.4.2.2 shall be used to determine the average condition for varying profile conditions.

For preliminary design, Site Class E or F should not be assumed unless the authority having jurisdiction determines that Site Class E or F could be present at the site or in the event that Site Class E or F is established by geotechnical data.

The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated on the basis of shear wave velocities in similar competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

The hard rock, Site Class A, category shall be

C3.4.2.1

Steps for classifying a site (also see Table 1):

Step 1: Check the site against the three categories of Site Class F, requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.

Step 2: Categorize the site using one of the following three methods, with $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$ computed in all cases as specified by the definitions in Article 3.4.2.2:

supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft, surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$.

The rock categories, Site Classes A and B, shall not be used if there is more than 100 ft of soil between the rock surface and the bottom of the spread footing or mat foundation.

PI shall be taken as the plasticity index specified in ASTM D 4318. The moisture content, *w*, shall be taken as the moisture content in percent specified in ASTM D 2216.

Method A: $\bar{v}_s$ for the top 100 ft ($\bar{v}_s$ method)

Method B: $\bar{N}$ for the top 100 ft ($\bar{N}$ method)

Method C: $\bar{N}_{ch}$ for cohesionless soil layers ($PI < 20$) in the top 100 ft and average $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft ($\bar{s}_u$ method)

The values $\bar{v}_s$, $\bar{N}_{ch}$, and $\bar{s}_u$ are averaged over the respective thickness of cohesionless and cohesive soil layers within the upper 100 ft. Refer to Article 3.4.2.2 for equations for calculating average parameter values for Methods A, B, and C. If Method C is used, the site class is determined as the softer site class resulting from the averaging to obtain $\bar{N}_{ch}$ and $\bar{s}_u$ (for example, if $\bar{N}_{ch}$ were equal to 20 blows/ft and $\bar{s}_u$ were equal to 800 psf, the site would classify as E in accordance with Table 1). Note that when using Method B, $\bar{N}$ values are for both cohesionless and cohesive soil layers within the upper 100 ft.

As described in Article C3.4.2.2, it may be appropriate in some cases to define the ground motion at depth, below a soft surficial layer, if the surficial layer would not significantly influence bridge response. In this case, the site class may be determined on the basis of the soil profile characteristics below the surficial layer.

Within Site Class F (soils requiring site-specific evaluation), one category has been deleted in these Guide Specifications from the four categories contained in the previously cited codes and documents. This category consists of soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible, weakly cemented soils. It was judged that special analyses for the purpose of refining site ground motion amplifications for these soils were too severe a requirement for ordinary bridge design because such analyses would require utilization of effective stress and strength-degrading nonlinear analyses that are difficult to conduct. Also, limited case-history data and analysis results indicate that liquefaction reduces spectral response rather than increases it, except at long periods in some cases. (e.g., $T = 1$ sec) Because of the general reduction in response spectral amplitudes due to liquefaction, the designer may wish to consider special analysis of site response for liquefiable soil sites to avoid excessive conservatism in assessing bridge inertia loads when liquefaction occurs.

Site-specific analyses are required for major or very important structures in some cases (Article 3.4), so that appropriate analysis techniques would be used for such structures. The deletion of liquefiable soils from Site Class F only affects the requirement to conduct site-specific analyses for the purpose of determining ground motion amplification through these soils. It is still required to evaluate liquefaction occurrence and its effect on a bridge as specified in Article 6.8.

Table 3.4.2.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/sec
B	Rock with 2500 ft/sec < $\bar{v}_s$ < 5000 ft/sec
C	Very dense soil and soil rock with 1200 ft/sec < $\bar{v}_s$ < 2500 ft/sec, or with either $\bar{N} > 50$ blows/ft or $\bar{s}_u > 2.0$ ksf
D	Stiff soil with 600 ft/sec < $\bar{v}_s$ < 1200 ft/sec, or with either 15 blows/ft < $\bar{N}$ < 50 blows/ft or 1.0 ksf < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/sec, or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with $PI > 20$, $w > 40\%$, and $\bar{s}_u < 0.5$ ksf
F	Soils requiring site-specific ground motion response evaluations, such as: Peats or highly organic clays ($H > 10$ ft of peat or highly organic clay, where H = thickness of soil) Very high plasticity clays ($H > 25$ ft with $PI > 75$) Very thick soft/medium stiff clays ($H > 120$ ft)

Exceptions:

Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site Class E or F should not be assumed unless the authority having jurisdiction determines that Site Class E or F could be present at the site or in the event that Site Class E or F is established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{N}$ = average standard penetration test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D 2166 or D 2850) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

PI = plasticity index (ASTM D 4318)

w = moisture content (ASTM D 2216)

3.4.2.2—Definitions of Site Class Parameters

The definitions presented below shall be taken to apply to the upper 100 ft of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft.

The average $\bar{v}_s$ for the site profile shall be taken as:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad (3.4.2.2\text{-}1)$$

where:

C3.4.2.2

If the site profile is particularly nonuniform, or if the average velocity computed in this manner does not appear reasonable, or if the project involves special design issues, it may be desirable to conduct shear wave velocity measurements. In all evaluations of site classification, the shear wave velocity should be viewed as the fundamental soil property, as this was used when conducting the original studies defining the site categories.

Use of Empirical v_{si} Relations: An alternative to applying Eqs. 2, 3, and 4 to obtain values for $\bar{N}$, $\bar{N}_{ch}$, and $\bar{s}_u$ is to convert the N values or s_u values into estimated shear wave velocities and then to apply Eq. 1. Procedures given in Kramer (1996) can be used for these conversions. The empirical equations identified in Kramer (1996) and in other references can involve significant uncertainty at a

$\sum_{i=1}^{n} d_i$ = thickness of upper soil layers = 100 ft

d_i = thickness of ith soil layer (ft)

n = total number of distinctive soil layers in the upper 100 ft of the site profile below the bridge foundation

v_{si} = shear wave velocity of ith soil layer (ft/sec)

i = any one of the layers between 1 and n

$\bar{N}$ shall be taken as:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \quad (3.4.2.2\text{-}2)$$

where:

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of ith soil layer not to exceed 100 ft (blows/ft).

$\bar{N}_{ch}$ shall be taken as:

$$\bar{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \quad (3.4.2.2\text{-}3)$$

where:

m = total number of cohesionless soil layers in the upper 100 ft of the site profile below the bridge foundation

$\bar{s}_u$ shall be taken as:

$$\bar{s}_u = \frac{\sum_{i=1}^{k} d_i}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \quad (3.4.2.2\text{-}4)$$

where:

k = total number of cohesive soil layers in the upper 100 ft of the site profile below the bridge foundation

s_{ui} = undrained shear strength of ith soil layer not to exceed 5 ksf

specific site, and this should be considered during the use of the empirical equations. The preferred approach is to calibrate the empirical procedure using in-situ velocity measurements when the empirical equations are to be used.

Depth of Motion Determination: For short bridges that involve a limited number of spans, the motion at the abutment will generally be the primary mechanism by which energy is transferred from the ground to the bridge superstructure. If the abutment is backed by an earth approach fill, the site classification should be determined at the base of the approach fill. The potential effects of the approach fill overburden pressure on the shear wave velocity of the soil should be accounted for in the determination of site classification.

For long bridges it may be necessary to determine the site classification at an interior pier. If this pier is supported on spread footings, then the motion computed at the ground surface is appropriate. However, if deep foundations (i.e., driven piles or drilled shafts) are used to support the pier, then the location of the motion will depend on the horizontal stiffness of the soil-cap system relative to the horizontal stiffness of the soil-pile system. If the pile cap is the stiffer of the two, then the motion should be defined at the pile cap. If the pile cap provides little horizontal stiffness or if there is no pile cap (i.e., pile extension), then the controlling motion will likely be at some depth below the ground surface. Typically this will be approximately 4 to 7 pile diameters below the pile cap or where a large change in soil stiffness occurs. The determination of this elevation requires considerable judgment and should be discussed by the geotechnical and bridge engineers.

For cases where the controlling motion is more appropriately specified at depth, site-specific ground response analyses can be conducted to establish ground motions at the point of fixity. This approach or alternatives to this approach should be used only with the Owner's approval.

3.4.2.3—Site Coefficients

Site coefficients for the peak ground acceleration F_{pga}, short-period range F_a, and for the long-period range F_v shall be taken as specified in Tables 1 and 2. Application of these coefficients to determine elastic seismic response coefficients of ground motion shall be as specified in Article 3.4.1.

C3.4.2.3

Site Class B (soft rock) is taken to be the reference site category for USGS and IBC ground motion site factors. Site Class B rock is therefore the site condition for which the site factor is 1.0. Site Classes A, C, D, and E have separate sets of site factors for zero-period (F_{pga}), the short-period range (F_a), and the long-period range (F_v), as indicated in Tables 1 and 2. These site factors generally increase as the soil profile becomes softer (in going from Site Class A to E). Except for Site Class A (hard rock), the factors also decrease as the ground motion level increases, due to the strongly nonlinear behavior of soil. For Site Classes C, D, or E, these nonlinear site factors increase the ground motion more in areas having lower rock ground motions than in areas having higher rock ground motions.

Table 3.4.2.3-1—Values of F_{pga} and F_a as a Function of Site Class and Mapped Peak Ground Acceleration or Short-Period Spectral Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration or Spectral Response Acceleration Coefficient at Short Periods				
	$PGA \leq 0.10$ $S_s \leq 0.25$	$PGA = 0.20$ $S_s = 0.50$	$PGA = 0.30$ $S_s = 0.75$	$PGA = 0.40$ $S_s = 1.00$	$PGA \geq 0.50$ $S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	[a]	[a]	[a]	[a]	[a]

Note: Use straight line interpolation for intermediate values of PGA and S_s, where PGA is the peak ground acceleration and S_s is the spectral acceleration coefficient at 0.2 sec obtained from the ground motion maps.

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	[a]	[a]	[a]	[a]	[a]

Note: Use straight line interpolation for intermediate values of S_1, where S_1 is the spectral acceleration coefficient at 1.0 sec obtained from the ground motion maps.

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

3.4.3—Response Spectra Based on Site-Specific Procedures

A site-specific procedure should be used to develop design response spectra of earthquake ground motions when required by Article 3.4 and may be performed for any site subject to the Owner's approval. The site-specific procedure can involve a site-specific hazard analysis, a site-specific ground motion response analysis, or both.

Unless otherwise approved by the Owner, where the response spectrum is developed using a site-specific hazard analysis, a site-specific ground motion response analysis, or both, the spectrum shall not be lower than two-thirds of the response spectrum at the ground surface determined using the general procedure of Article 3.4.1 adjusted by the site coefficients in Article 3.4.2.3 in the region of $0.5\,T_F$ to $2\,T_F$ of the spectrum, where T_F is the bridge fundamental period. For other analyses, such as liquefaction assessment and retaining wall design, the free-field acceleration at the ground surface should not be less than two-thirds of A_s determined from the general procedure.

C3.4.3

When estimating the minimum ground surface response spectrum using two-thirds of the response spectrum from the general procedure in Article 3.4.1 and the site coefficients in Article 3.4.2.3, there are no site coefficients for liquefiable sites or for sites that fall in Site Class F. No consensus currently exists regarding the appropriate site coefficients for these cases. Unless the Owner directs otherwise, the following approach should be used:

- For liquefiable sites, use the site coefficient based on soil conditions without any modifications for liquefaction. This approach is believed to be conservative for higher frequency motions, and the Owner may decide to use a minimum spectrum lower than the two-thirds value. However, when accepting a spectrum lower than two-thirds of the spectrum identified in the above discussions, the uncertainties in the analysis method should be carefully reviewed, particularly for longer periods (i.e., $T > 1.0$ sec) where increases in the spectral ordinate may occur. If a lower factor than two-thirds is being considered, it is suggested that an independent peer review of the results of the site-specific analyses be performed.
- For Site Class F locations, the recommended approach is to accept the results of a site-specific study subject to the concurrence of the Owner and an independent peer review panel. In previous guidance documents (ATC and MCEER, 2003), the suggestion was made to use a Site Class E site coefficient for Site Class F soils. This approach appears to be overly conservative and is not suggested.

3.4.3.1—Site-Specific Hazard Analysis

If the probabilistic seismic hazard analysis (PSHA) is used, the site-specific analysis shall be conducted in a manner to generate a uniform-hazard acceleration response spectrum considering a seven percent probability of exceedance in 75 yr for spectral values over the entire period range of interest. This analysis shall establish the following:

- The contributing seismic sources,
- An upper-bound earthquake magnitude for each source zone,
- Median attenuation relations for acceleration response spectral values and their associated standard deviations,
- A magnitude-recurrence relation for each source zone, and

C3.4.3.1

The intent in conducting a site-specific hazard study is to develop ground motions that are more accurate for the local seismic and site conditions than can be determined from national ground motion maps and the procedure of Article 3.4.1. Accordingly, such studies should be comprehensive and incorporate current scientific interpretations at a regional scale. Because there are typically scientifically credible alternatives for models and parameter values used to characterize seismic sources and ground-motion attenuation, it is important to incorporate these uncertainties formally in a site-specific hazard analysis. Examples of these uncertainties include seismic source location, extent, and geometry; maximum earthquake magnitude; earthquake recurrence rate; and ground-motion attenuation relationship.

- A magnitude fault-rupture length or source area relation for each contributing fault or source area.

If the deterministic seismic hazard analysis (DSHA) method is used, the analysis shall establish all of the items listed above, except the magnitude-recurrence relation for each seismic source. The site-specific deterministic spectrum at the ground surface, adjusted by the site coefficients in Article 3.4.2.3, shall be no less than the seven percent probability of exceedance in 75 yr response spectrum determined using the general procedure in Articles 3.4.1 and 3.4.2 in the region of $0.5\,T_F$ to $2\,T_F$ of the spectrum, where T_F is the bridge fundamental period. The same would also apply to the free field ground acceleration A_s.

Where use of a deterministic spectrum is appropriate, the spectrum shall be either:

- The envelope of median spectra calculated for characteristic maximum magnitude earthquakes on known active faults, or
- The deterministic spectra for each fault and, in the absence of a clearly controlling spectrum, each spectrum should be used.

Uncertainties in source modeling and parameter values shall be taken into consideration in the PSHA and DSHA. Detailed documentation of seismic hazard analysis shall be provided and shall be peer reviewed as appropriate (Article C3.4.1).

For sites located within 6 mi of an active surface or shallow fault, as depicted in the USGS Active Fault Map, near-fault effects on ground motions should be considered to determine if these could significantly influence the bridge response.

The most up-to-date attenuation relationships should be used when developing the hazard model. Attenuation relationships used in developing the USGS/AASHTO Seismic Hazard Maps for these Guide Specifications do not include the Next Generation Attenuation (NGA) relationships developed in 2006 and 2007. It is recommended that the NGA relationships be used for any future site-specific studies for West Coast sites.

Near-fault effects on horizontal response spectra include:

- Higher ground motions due to the proximity of the active fault,
- Directivity effects that increase ground motions for periods greater than 0.5 sec if the fault rupture propagates toward the site, and
- Directionality effects that increase ground motions for periods greater than 0.5 sec in the direction normal (perpendicular) to the strike of the fault.

If the active fault is included in the development of national ground motion maps, then the first effect is already included in the national ground motion maps. The second and third effects are not included in the national maps. These effects are significant only for periods longer than 0.5 sec and normally would be evaluated only for essential or critical bridges having natural periods of vibration longer than 0.5 sec. Further discussions of the second and third effects are contained in Somerville (1997) and Somerville et al. (1997). The ratio of vertical-to-horizontal ground motions increases for short-period motions in the near-fault environment.

The fault-normal component of near-field motion ($D < 6$ mi) may contain relatively long-duration velocity pulses that can cause severe nonlinear structural response, predictable only through nonlinear time-history analyses. For this case, the recorded near-field horizontal components of motion shall be transformed into principal components before being modified to be response-spectrum-compatible.

If deterministic methods are used to develop design spectra, the spectral ordinates should be developed using a range of ground motion attenuation relationships consistent with the source mechanisms. At least three and preferably more attenuation relationships should be used.

If the site-specific deterministic hazard analysis is combined with a site-specific ground motion response analysis, the response spectrum may be as low as two-thirds of the response spectrum at the ground surface determined using the general procedure of Article 3.4.1 adjusted by the site coefficients in Article 3.4.2.3 in the region of $0.5\,T_F$ to $2\,T_F$ (see Article C3.4.3). The same would also apply to the free field acceleration A_s in this case.

3.4.3.2—Site-Specific Ground Motion Response Analysis

Where analyses to determine site soil response effects are suggested by Articles 3.4 and 3.4.2.1, the influence of the local soil conditions shall be determined based on site-specific geotechnical investigations and dynamic site response analyses. Site-specific investigations should be conducted in accordance with Article 6.2.

Methods for conducting the site-specific dynamic ground response analyses shall consist of developing a model of the soil profile and then using numerical modeling methods to evaluate the effects of the soils on wave propagation resulting from representative earthquake ground motions.

C3.4.3.2

Site-specific ground motion response analyses are required by these Guide Specifications for Site Class F sites. Site coefficients identified in Article 3.4.2.3 either were not considered or could not be easily generalized for Site Class F soils, and therefore, require site-specific evaluation.

Additional site conditions may also warrant a site-specific ground motion evaluation, particularly for locations that are outside the range of the original site class development. These conditions can include very deep soil deposits or thin soil deposits over rock.

- Deep soil deposits have a pronounced yet not fully understood influence on the amplification or deamplification of ground motions. Areas encountering this effect include central and eastern United States. In general, geotechnical engineers have noted a shift of the design response spectrum to larger periods when conducting site-specific ground motion response analyses. If deep soil deposits are not considered in developing response spectra for these sites, longer-period bridges may be under-designed and shorter period bridges may be over-designed.

- When the thickness of the soil over rock is less than 40 to 50 ft, significant amplification of ground motions can occur at periods of less than 0.5 sec. These large ground motions can lead to very high inertial forces on stiff bridges.

Site-specific ground motion response analyses should include (1) modeling of the soil profile, (2) selecting input motions for input into the soil profile, and (3) conducting a site response analysis.

- *Modeling the Soil Profile* Typically, a one-dimensional soil column extending from the ground surface to some depth below the ground surface is used to capture first order site response characteristics. Two- and three-dimensional models may be considered for critical projects when two- or three-dimensional wave propagation effects may be significant (e.g., in basins). The soil layers in a one-dimensional model are characterized by total unit weight, shear wave velocities from which low-strain (maximum) shear moduli may be obtained, and by relationships defining the nonlinear shear stress–strain relationships of the soil. The required relationships for analysis are often in the form of curves that describe the variation of shear modulus with shear strain (modulus reduction curves) and by curves that describe the variation of material damping with shear strain (damping curves). Typical modulus reduction curves and damping curves should be selected on the basis of published relationships for similar soils. Kramer (1996) provides a summary of these relationships. Site-specific laboratory dynamic tests on soil samples to establish nonlinear soil characteristics can be considered where published relationships are judged to be inadequate for the types of soils present at the site. Uncertainties in soil modulus and damping properties should be considered in the modeling effort.

- *Select Input Ground Motions* Acceleration time histories that are representative of horizontal input motion should be used in the ground response model. Article 3.4.4 provides guidance on the selection of earthquake records. Input motions should be defined either at the top of rock or at the top of a layer where a significant stiffness contrast occurs. Motions selected for input should be from earthquake records obtained on geologic materials with similar shear wave velocity characteristics. Earthquake record databases are available from the Pacific Earthquake Engineering Research (PEER) Center website and the Consortium of Organizations for Strong Motion Observations Survey (COSMOS) website. Earthquake records should be scaled to a target spectrum developed using either the general procedure or the site-specific hazard analysis. Scaling should be done in the period range of interest. Because the spectrum from the general procedure or a site-specific hazard analysis is defined at the ground surface rather than at depth, the input time histories should be input in the analysis as an outcropping motion.

- *Site Response Analysis and Results Interpretation* Either equivalent linear or nonlinear methods may be used to conduct the site-specific ground motion response analysis. Frequently used computer programs for one-dimensional analysis include the equivalent-linear program SHAKE or nonlinear programs DESRA, SUMDES, and DMOD. See Kramer (1996) for additional discussions on these modeling methods. If the soil response is highly nonlinear (e.g., high acceleration levels or soft clay soils), nonlinear programs are generally preferable to equivalent–linear programs. For analysis of liquefaction effects on site response, computer programs incorporating pore-water pressure models may be considered. Response spectra of output motions at the ground surface should be calculated and the ratios of response spectra of ground surface motions to input outcropping motions should be calculated. Typically, an average of the response spectral ratio curves is obtained and multiplied by the design rock response spectrum to obtain a soil response spectrum. This response spectrum should be adjusted to a smooth design soil response spectrum by slightly decreasing spectral peaks and slightly increasing spectral valleys. Sensitivity analyses to evaluate the effects of soil property uncertainties should be conducted and considered in developing the design response spectrum.

Nonlinear, effective stress methods that account for the build-up in pore-water pressure and stiffness degradation have been developed to evaluate site response at sites where liquefiable soils exist. Use of this approach requires considerable skill in terms of selecting input motions and soil model parameters, particularly the pore pressure model. Currently, there is no consensus within the profession regarding the "correct" pore pressure model. The complexity of this approach is such that Owner's approval is mandatory, and it is highly advisable that an independent peer review panel with expertise in nonlinear, effective stress modeling be used to review the methods and the resulting spectrum.

3.4.4—Acceleration Time Histories

Earthquake acceleration time histories will be required for site-specific ground motion response evaluations and for nonlinear inelastic dynamic analysis of bridge structures. The time histories for these applications shall have characteristics that are representative of the seismic environment of the site and the local site conditions, including the response spectrum for the site.

Response-spectrum-compatible time histories shall be developed from representative recorded earthquake motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching. The recorded time histories should be scaled to the approximate level of the design response spectrum in the period range of

C3.4.4

Characteristics of the seismic environment of the site to be considered in selecting time histories include tectonic environment (e.g., subduction zone; shallow crustal faults in western United States or similar crustal environment; eastern United States or similar crustal environment); earthquake magnitude; type of faulting (e.g., strike-slip; reverse; normal); seismic-source-to-site distance; local site conditions; and design or expected ground motion characteristics (e.g., design response spectrum; duration of strong shaking; and special ground motion characteristics such as near-fault characteristics). Dominant earthquake magnitudes and distances, which contribute principally to the probabilistic design response spectra at a site, as determined from national ground motion maps, can be obtained from deaggregation information on the USGS Web significance unless otherwise approved by the Owner. At least three response-spectrum-compatible time histories shall be used for representing the design earthquake (ground motions having seven percent probability of exceedance in 75 yr) when conducting dynamic ground motion response analyses or nonlinear inelastic modeling of bridges.

- For site-specific ground motion response modeling, single components of separate records shall be used in the response analysis. The target spectrum used to develop the time histories is defined at the base of the soil column. The target spectrum is obtained from the USGS/AASHTO Seismic Hazard Maps or from a site-specific hazard analysis as described in Article 3.4.3.1.
- For nonlinear time history modeling of bridge structures, the target spectrum is usually located at or close to the ground surface, i.e., the rock spectrum has been modified for local site effects. Each component of motion shall be modeled. The issue of requiring all three orthogonal components (x, y, and z) of design motion to be input simultaneously shall be considered as a requirement when conducting a nonlinear time history analysis. The design actions shall be taken as the maximum response calculated for the three ground motions in each principal direction.

If a minimum of seven time histories is used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction.

For near-field sites ($D < 6$ mi), the recorded horizontal components of motion selected should represent a near-field condition and they should be transformed into principal components before making them response-spectrum compatible. The major principal component should then be used to represent motion in the fault-normal direction and the minor principal component should be used to represent motion in the fault-parallel direction.

site (http://geohazards.cr.usgs.gov/).

It is desirable to select time histories that have been recorded under conditions similar to the seismic conditions at the site listed above, but compromises are usually required because of the multiple attributes of the seismic environment and the limited data bank of recorded time histories. Selection of time histories having similar earthquake magnitudes and distances, within reasonable ranges, are especially important parameters because they have a strong influence on response spectral content, response spectral shape, duration of strong shaking, and near-source ground motion characteristics. It is desirable that selected recorded motions be somewhat similar in overall ground motion level and spectral shape to the design spectrum to avoid using very large scaling factors with recorded motions and very large changes in spectral content in the spectrum-matching approach. If the site is located within 6 mi of an active fault, then intermediate-to-long-period ground motion pulses that are characteristic of near-source time histories should be included if these types of ground motion characteristics could significantly influence structural response. Similarly, the high short-period spectral content of near-source vertical ground motions should be considered.

Ground motion modeling methods of strong motion seismology are being increasingly used to supplement the recorded ground motion database. These methods are especially useful for seismic settings for which relatively few actual strong motion recordings are available, such as in the central and eastern United States. Through analytical simulation of the earthquake rupture and wave-propagation process, these methods can produce seismologically reasonable time series.

Response spectrum-matching approaches include methods in which time series adjustments are made in the time domain (Lilhanand and Tseng, 1988; Abrahamson, 1992) and those in which the adjustments are made in the frequency domain (Gasparini and Vanmarcke, 1976; Silva and Lee, 1987; Bolt and Gregor, 1993). Both of these approaches can be used to modify existing time histories to achieve a close match to the design response spectrum while maintaining fairly well the basic time-domain character of the recorded or simulated time histories. To minimize changes to the time-domain characteristics, it is desirable that the overall shape of the spectrum of the recorded time history not be greatly different from the shape of the design response spectrum and that the time history initially be scaled so that its spectrum is at the approximate level of the design spectrum before spectrum matching.

When developing three-component sets of time histories by simple scaling rather than spectrum matching, it is difficult to achieve a comparable aggregate match to the design spectra for each component of motion when using a single scaling factor for each time history set. It is desirable, however, to use a single scaling factor to preserve the relationship between the components. Approaches for dealing with this scaling issue include:

- Using a higher scaling factor to meet the minimum aggregate match requirement for one component while exceeding it for the other two,
- Using a scaling factor to meet the aggregate match for the most critical component with the match somewhat deficient for other components, and
- Compromising on the scaling by using different factors as required for different components of a time history set.

Although the second approach is acceptable, it requires careful examination and interpretation of the results and possibly dual analyses for application of the higher horizontal component in each principal horizontal direction.

The requirements for the number of time histories to be used in nonlinear inelastic dynamic analysis and for the interpretation of the results take into account the dependence of response on the time-domain character of the time histories (duration, pulse shape, pulse sequencing) in addition to their response spectral content.

Additional guidance on developing acceleration time histories for dynamic analysis may be found in publications by the Caltrans Seismic Advisory Board Adhoc Committee (CSABAC) on Soil-Foundation-Structure Interaction (1999) and the U.S. Army Corps of Engineers (2000). CSABAC (1999) also provides detailed guidance on modeling the spatial variation of ground motion between bridge piers and the conduct of seismic soil-foundation-structure interaction (SFSI) analyses. Both spatial variations of ground motion and SFSI may significantly affect bridge response. Spatial variations include differences between seismic wave arrival times at bridge piers (wave passage effect), ground motion incoherence due to seismic wave scattering, and differential site response due to different soil profiles at different bridge piers. For long bridges, all forms of spatial variations may be important. For short bridges, limited information appears to indicate that wave passage effects and incoherence are, in general, relatively unimportant in comparison to effects of differential site response (Shinozuka et al., 1999; Martin, 1998). Somerville et al. (1999) provide guidance on the characteristics of pulses of ground motion that occur in time histories in the near-fault region.

3.5—SELECTION OF SEISMIC DESIGN CATEGORY (SDC)

Each bridge shall be assigned to one of four seismic design categories (SDCs), A through D, based on the 1-sec period design spectral acceleration for the design earthquake (S_{D1}, refer to Article 3.4.1) as shown in Table 1.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}.

C3.5

The seismic hazard level is defined as a function of the magnitude of the ground surface shaking as expressed by F_vS_1. However, other factors may affect the SDC selected. For example, if the soil is liquefiable and lateral spreading or slope failure can occur, SDC D should be selected. For assessment of existing structures, the Designer should also consider using SDC D regardless of the magnitude of A_s, even when significant lateral soil movement is not expected, if the structure is particularly weak with regard to its ability to resist the forces and displacements that could be caused by the liquefaction (see Article C6.8).

The SDC reflects the variation in seismic risk across the country and is used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures.

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \le S_{D1} < 0.30$	B
$0.30 \le S_{D1} < 0.50$	C
$0.50 \le S_{D1}$	D

The requirements for each of the proposed SCDs shall be taken as shown in Figure 1 and described below. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6, respectively, and shall also meet minimum support length requirements of Article 4.12.

- SDC A
  - a. No identification of ERS according to Article 3.3
  - b. No demand analysis
  - c. No implicit capacity check needed
  - d. No capacity design required
  - e. Minimum detailing requirements for support length, superstructure/substructure connection design force, and column transverse steel
  - f. No liquefaction evaluation required
- SDC B
  - g. Identification of ERS according to Article 3.3 should be considered
  - h. Demand analysis
  - i. Implicit capacity check required (displacement, P-Δ, support length)
  - j. Capacity design should be considered for column shear; capacity checks should be considered to avoid weak links in the ERS
  - k. SDC B level of detailing
  - l. Liquefaction check should be considered for certain conditions

- SDC C
  - m. Identification of ERS
  - n. Demand analysis
  - o. Implicit capacity check required (displacement, P-Δ, support length)
  - p. Capacity design required including column shear requirement
  - q. SDC C level of detailing
  - r. Liquefaction evaluation required.
- SDC D
  - s. Identification of ERS
  - t. Demand analysis
  - u. Displacement capacity required using pushover analysis (check P-Δ and support length)
  - v. Capacity design required including column shear requirement
  - w. SDC D level of detailing
  - x. Liquefaction evaluation required.

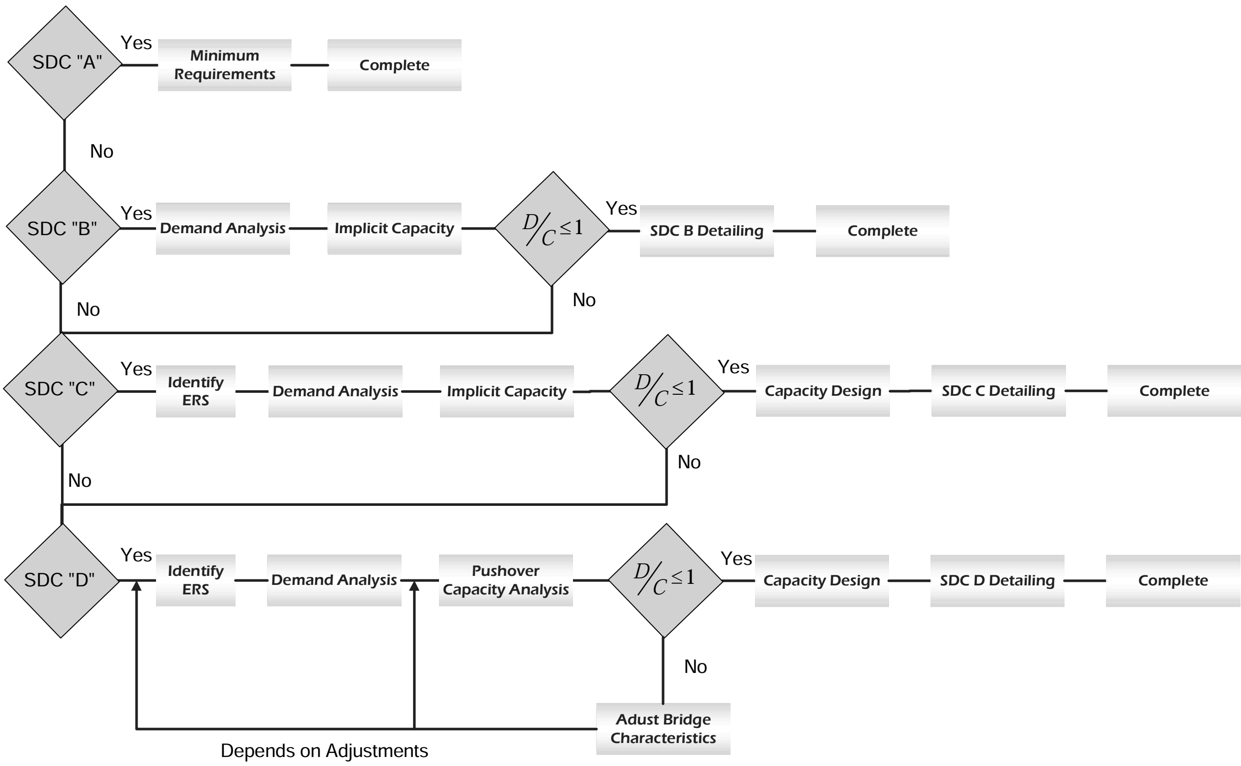


Figure 3.5-1—Seismic Design Category (SDC) Core Flowchart

3.6—TEMPORARY AND STAGED CONSTRUCTION

Any bridge or partially constructed bridge that is expected to be temporary for more than five years shall be designed using the requirements for permanent structures and shall not use the provisions of this Article.

Temporary bridges expected to carry vehicular traffic or pedestrian bridges over roads carrying vehicular traffic shall satisfy the performance criteria defined in Article 3.2. The provisions also apply to those bridges that are constructed in stages and expected to carry traffic and/or pass over routes that carry traffic. The design response spectra given in Article 3.4 may be reduced by a factor of not more than 2.5 to calculate the component elastic forces and displacements. The SDC of the temporary bridge shall be obtained on the basis of the reduced/modified response spectrum except that a temporary bridge classified in SDC B, C, or D based on the unreduced spectrum cannot be reclassified to SDC A based on the reduced/modified spectrum. The requirements for each of the SDCs A through D shall be met as defined in Article 3.5. Response spectra for construction sites that are within 6 mi of an active fault (see Article 3.4) shall be the subject of special study.

C3.6

The option to use a reduced acceleration coefficient is provided to reflect the limited exposure period.

3.7—LOAD AND RESISTANCE FACTORS

Use load factors of 1.0 for all permanent loads. Unless otherwise noted, all ϕ factors shall be taken as 1.0.

C3.7

Historically, the load factor for live load has been taken as zero for the earthquake load combination except where heavy truck traffic, high average daily traffic, or long structure length are anticipated.

SECTION 4: ANALYSIS AND DESIGN REQUIREMENTS

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SECTION 4:

ANALYSIS AND DESIGN REQUIREMENTS

4.1—GENERAL

4.1.1—Application

The requirements of this chapter shall control the selection and method of seismic analysis and design of bridges. The seismic design demand displacements shall be determined in accordance with the procedures of Section 5. Material and foundation design requirements are given in Sections 6, 7, and 8.

Seismic design requirements for single-span bridges shall be taken as specified in Articles 4.5 and 4.12. Design requirements for bridges classified as SDC A are specified in Articles 4.6 and 4.12. Detailed seismic analysis should not be required for a single-span bridge or for bridges classified as SDC A.

Articles 4.1.2, 4.1.3, 4.1.4, and 4.1.5 include recommendations that should be considered for SDC D. Compliance with these recommendations, which are based on past experience, should typically yield preferred seismic performance.

4.1.2—Balanced Stiffness SDC D

It is recommended that the ratio of effective stiffness, as shown in Figure 1 and summarized below, between any two bents within a frame and between any two columns within a bent should satisfy Eq. 1 for frames of constant width and Eq. 2 for frames of variable width. It is also recommended that the ratio of effective stiffness between adjacent bents within a frame and between adjacent columns within a bent should satisfy Eq. 3 for frames of constant width and Eq. 4 for frames of variable width. These recommendations exclude the consideration of abutments. An increase in mass along the length of a frame should be accompanied by a reasonable increase in stiffness. For variable width frames, the tributary mass supported by each bent or column shall be included in the stiffness comparisons as specified in Eqs. 2 and 4.

- Any two bents within a frame or any two columns within a bent

Constant width frames:

$$\frac{k_i^e}{k_j^e} \geq 0.5 \tag{4.1.2-1}$$

C4.1.2

The distributions of stiffness and mass are included in the model for dynamic analysis. The discretization of the model should 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 should be located at the locus of 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 unusual pier configurations, the rotational mass moment of inertia of the superstructure about the longitudinal axis should 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 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.

Variable width frames:

$$\frac{k_i^e m_j}{k_j^e m_i} \geq 0.5 \qquad (4.1.2\text{-}2)$$

- Adjacent bents within a frame or adjacent columns within a bent

Constant width frames:

$$\frac{k_i^e}{k_j^e} \geq 0.75 \qquad (4.1.2\text{-}3)$$

Variable width frames:

$$\frac{k_i^e m_j}{k_j^e m_i} \geq 0.75 \qquad (4.1.2\text{-}4)$$

where:

k_i^e = smaller effective bent or column stiffness kip/in.)

k_j^e = larger effective bent or column stiffness (kip/in.)

m_i = tributary mass of column or bent i (kip)

m_j = tributary mass of column or bent j (kip)

The following considerations shall be taken into account when calculating effective stiffness of concrete components: framing effects, end conditions, column height, percentage of longitudinal and transverse column steel, column diameter, and foundation flexibility.

Some of the consequences of not meeting the relative stiffness recommendations defined above include:

- Increased damage in the stiffer elements,
- An unbalanced distribution of inelastic response throughout the structure, and
- Increased column torsion generated by rigid body rotation of the superstructure.

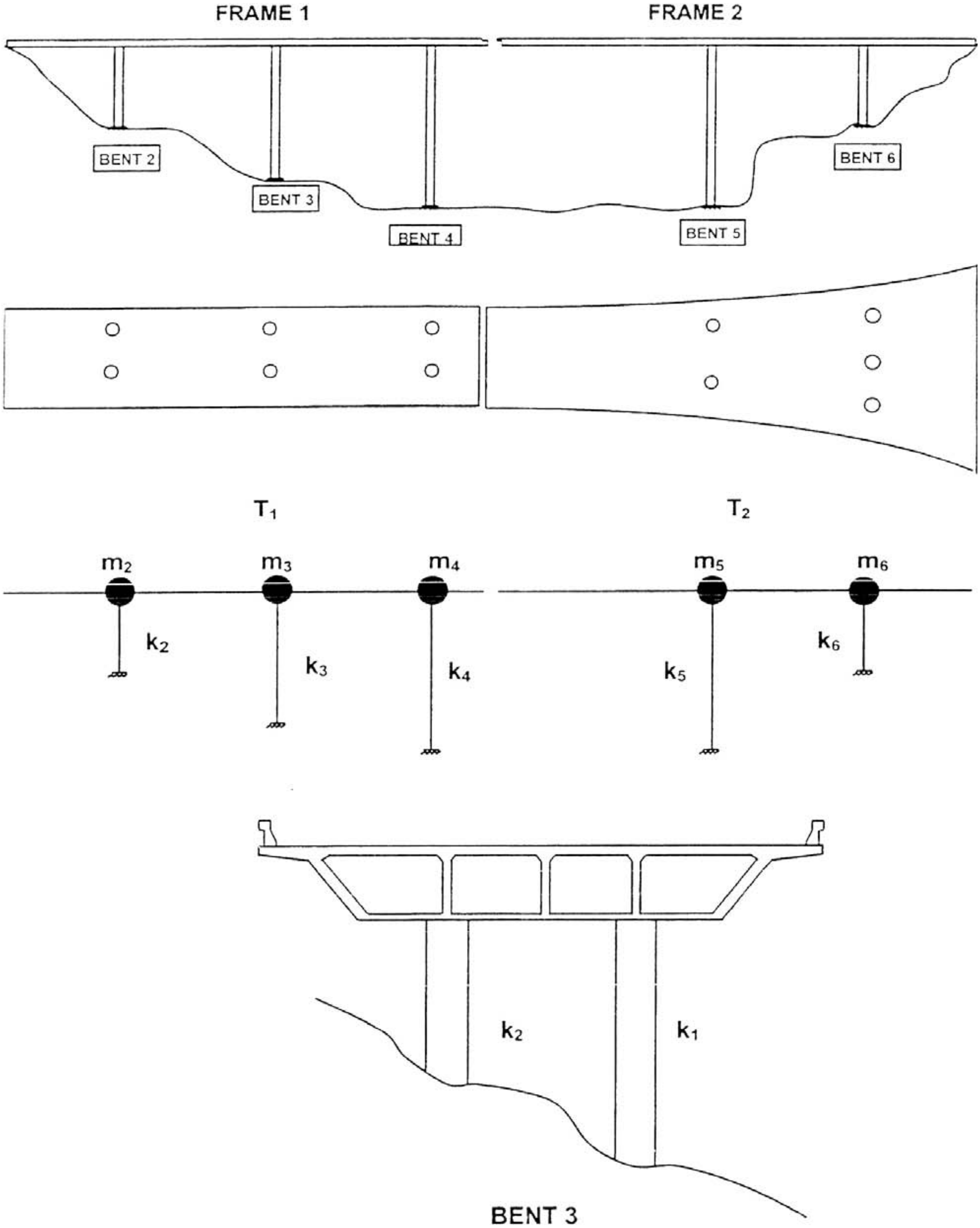


Figure 4.1.2-1—Balanced Stiffness Concepts for Frames, Bents, and Columns

4.1.3—Balanced Frame Geometry SDC D

It is recommended that the ratio of fundamental periods of vibration for adjacent frames in the longitudinal and transverse direction should satisfy:

$$\frac{T_i}{T_j} \geq 0.7 \qquad (4.1.3\text{-}1)$$

where:

T_i = natural period of the less flexible frame (sec)

T_j = natural period of the more flexible frame (sec)

The consequences of not meeting the fundamental period requirements of Eq. 1, including a greater likelihood of out-of-phase response between adjacent frames leading to large relative displacements between the frames that increase the probability of longitudinal unseating and pounding between frames at the expansion joints, shall be considered.

C4.1.3

For bridges with multiple frames, which are separated by expansion bearings or hinges, it is unnecessary to model and analyze the entire bridge for seismic loads. Each frame should 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.

The pounding and relative transverse translation of adjacent frames will transfer the seismic demand from one frame to the next, which can be detrimental to the stand-alone capacity of the frame receiving the additional seismic demand.

4.1.4—Adjusting Dynamic Characteristics

The following list of techniques should be considered for adjusting or tuning the fundamental period of vibration and/or stiffness to satisfy Eqs. 4.1.2-1 to 4.1.2-4 and 4.1.3-1:

- Use oversized pile shafts,
- Adjust effective column lengths (i.e., lower footings, isolation casing),
- Use modified end fixities,
- Reduce and/or redistribute superstructure mass,
- Vary the column cross-section and longitudinal reinforcement ratios,
- Add or relocate columns,
- Modify the hinge/expansion joint layout,
- Incorporate isolation bearings or dampers (i.e., response modification devices), and
- Redesign the articulation.

If project constraints make it impractical to satisfy the stiffness and structure period requirements in Eqs. 4.1.2-1 to 4.1.2-4 and 4.1.3-1, a careful evaluation of the local ductility demands and capacities shall be required for bridges in SDC D.

4.1.5—End Span Considerations

The influence of the superstructure torsional rigidity on the transverse stiffness of single-column bents near the abutment shall be considered.

C4.1.5

This is particularly important when calculating shear demands for single columns where considering single curvature of the column is deemed nonconservative for ensuring adequate shear capacity.

4.2—SELECTION OF ANALYSIS PROCEDURE TO DETERMINE SEISMIC DEMAND

Minimum requirements for the selection of an analysis method to determine seismic demands for a particular bridge type shall be taken as specified in Tables 1 and 2. Applicability shall be determined by the "regularity" of a bridge, which is a function of the number of spans and the distribution of weight and stiffness. Regular bridges shall be taken as those having fewer than seven spans; no abrupt or unusual changes in weight, stiffness, or geometry; and that satisfy the requirements in Table 3. The changes in these parameters for SDC D should be within the tolerances given by Eqs. 4.1.2-1 to 4.1.2-4 from span-to-span or from support-to-support (abutments excluded). Any bridge not satisfying the requirements of Table 3 shall be considered "not regular."

Table 4.2-1—Analysis Procedures

Seismic Design Category	Regular Bridges with 2 through 6 Spans	Not Regular Bridges with 2 or More Spans
A	Not required	Not required
B, C, or D	Use Procedure 1 or 2	Use Procedure 2

Details of the analytical model and procedures mentioned in Table 1 shall be taken as specified in Section 5.

The analysis procedures specified in Table 2 shall be used.

Table 4.2-2—Description of Analysis Procedures

Procedure Number	Description	Article
1	Equivalent static	5.4.2
2	Elastic dynamic analysis	5.4.3
3	Nonlinear time history	5.4.4

Procedure 3 is generally not required unless:

- P-Δ effects are too large to be neglected,
- Damping provided by a base isolation system is large, and
- Requested by the Owner per Article 4.2.2.

Table 4.2-3—Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30º	30º	30º	30º	30º
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

Note: All ratios expressed in terms of the smaller value.

4.2.1—Special Requirements for Curved Bridges

A curved 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 4.2-3 except that for a two-span bridge the maximum span length ratio from span-to-span shall not exceed 2,
- The subtended angle in plan is not greater than 90º, 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 bridges shall be analyzed using the actual curved geometry.

C4.2.1

A common practice is to define the "longitudinal direction" of a curved bridge as that of the chord connecting the ends of the bridge and the transverse direction as orthogonal to the longitudinal direction.

4.2.2—Limitations and Special Requirements

Articles 4.2 and 4.2.1 shall be taken as applicable to normal bridges as specified in Article 3.1.

More rigorous methods of analysis shall be required for certain classes of important bridges that are considered to be critical or essential structures and/or for those that are geometrically complex or close to active earthquake faults (see Article 3.4.3). Nonlinear time history analyses, Procedure 3, should generally be used for critical/essential bridges as approved by the Owner. For bridges that require the use of nonlinear time history analysis, such analysis should meet the requirements of Section 5 of these Guide Specifications.

C4.2.2

Essential or critical bridges within 6 mi of an active fault require a site-specific study and inclusion of vertical ground motion in the seismic analysis. For normal bridges located within 6 mi of an active fault, the procedures in Article 4.7.2 are used to account for the response to vertical ground motion in lieu of including the vertical component in the seismic analysis. 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.

4.3—DETERMINATION OF SEISMIC LATERAL DISPLACEMENT DEMANDS

The global structure displacement demand, Δ_D, shall be taken as the total seismic displacement at a particular location within the structure or subsystem. Components of the global displacement demand that should be considered include components attributed to foundation flexibility, Δ_f, that is, foundation rotation or translation, flexibility of essentially elastic components such as bent caps Δ_b, and the flexibility attributed to elastic and inelastic response of ductile members Δ_y and Δ_{pd}, respectively.

The minimum requirements for superstructure, abutment, and foundation modeling, specified in Section 5, shall be considered.

4.3.1—Horizontal Ground Motions

For bridges classified as SDC B, C, or D, the global seismic displacement demands, Δ_D, shall be determined independently along two perpendicular axes, typically the longitudinal and transverse axes of the bridge, by the use of the analysis procedure specified in Article 4.2 and as modified using Articles 4.3.2 and 4.3.3. The resulting displacements shall then be combined as specified in Article 4.4. The longitudinal axis of a curved bridge may be selected along a chord connecting the two abutments.

4.3.2—Displacement Modification for Other than Five Percent Damped Bridges

Damping ratios on the order of 10 percent may be used with the approval of the Owner for bridges that are substantially influenced by energy dissipation of the soils at the abutments and are expected to respond predominately as a single-degree-of-freedom system. A reduction factor, R_D, may be applied to the five percent damped design spectrum coefficient used to calculate the displacement demand.

The following characteristics may be considered as justification for the use of higher damping:

- Total bridge length is less than 300 ft,
- Abutments are designed for sustained soil mobilization,
- Supports are normal or slight skew (less than 20°), and
- The superstructure is continuous without hinges or expansion joints.

The damping reduction factor, R_D, shall be taken as:

$$R_D = \left(\frac{0.05}{\xi}\right)^{0.4} \quad (4.3.2\text{-}1)$$

where:

ξ = damping ratio (maximum of 0.1)

The displacement demands for bridges with abutments designed to fuse shall be based on a five percent damped spectrum curve unless the abutments are specifically designed for sustained soil mobilization.

C4.3.2

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

In lieu of measurements, the following values may be used for the equivalent viscous damping ratio of time history analysis:

- Concrete construction 5%
- Welded and bolted steel construction.................... 2%

For single-span bridges 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.

End diaphragm and rigid frame abutments typically are effective in mobilizing the surrounding soil. However, abutments that are designed to fuse (seat type) or respond in a flexible manner may not develop enough sustained structure–soil interaction to rely on the higher damping ratio.

4.3.3—Displacement Magnification for Short-Period Structures

The displacement demand, Δ_D, calculated from elastic analysis shall be multiplied by the factor R_d specified in Eq. 1 or 2 to obtain the design displacement demand specified in Article 4.3

$$R_d = \left(1 - \frac{1}{\mu_D}\right)\frac{T^*}{T} + \frac{1}{\mu_D} \geq 1.0 \text{ for } \frac{T^*}{T} > 1.0 \quad (4.3.3\text{-}1)$$

$$R_d = 1.0 \text{ for } \frac{T^*}{T} \leq 1.0 \quad (4.3.3\text{-}2)$$

in which:

$$T^* = 1.25T_s \quad (4.3.3\text{-}3)$$

C4.3.3

The assumption that displacements of an elastic system will be the same as those of an elasto-plastic system is not valid for short-period structures that are expected to perform inelastically. The adjustment factor, R_d, is a method of correcting for the displacement determined from an elastic analysis for short-period structures.

The displacement magnification, R_d, is >1 in cases in which the fundamental period of the structure, T, is less than the characteristic ground motion period, T^*, corresponding to the peak energy input spectrum.

where:

μ_D = maximum local member displacement ductility demand

= 2 for SDC B

= 3 for SDC C

= determined in accordance with Article 4.9 for SCD D. In lieu of a detailed analysis, μ_D may be taken as 6

T_s = period determined from Article 3.4.1 (sec)

The displacement magnification shall be applied separately in both orthogonal directions prior to obtaining the orthogonal combination of seismic displacements specified in Article 4.4.

4.4—COMBINATION OF ORTHOGONAL SEISMIC DISPLACEMENT DEMANDS

A combination of orthogonal seismic displacement demands shall be used to account for the directional uncertainty of earthquake motions and the simultaneous occurrences of earthquake forces in two perpendicular horizontal directions. The seismic displacements resulting from analyses in the two perpendicular directions as described in Article 4.3 shall be combined to form two independent load cases as follows:

- *Load Case 1*: Seismic demand displacements along each of the principal axes of a member shall be obtained by adding 100 percent of the absolute value of the member seismic displacements resulting from the analysis in one of the perpendicular (longitudinal) directions to 30 percent of the absolute value of the corresponding member seismic displacements resulting from the analysis in the second perpendicular direction (transverse).
- *Load Case 2*: Seismic displacements on each of the principal axes of a member shall be obtained by adding 100 percent of the absolute value of the member seismic displacements resulting from the analysis in the second perpendicular direction (transverse) to 30 percent of the absolute value of the corresponding member seismic displacements resulting from the analysis in the first perpendicular direction (longitudinal).

There are some design procedures that require the development of elastic seismic forces. The procedure for developing such forces is the same as that for displacements.

C4.4

The combination of the vibration modes due to ground motion in one direction (longitudinal, transverse, or vertical) by the CQC method ("complete quadratic combination") provides a good estimate of the maximum displacement, including the correlation of modal responses closely spaced in frequency.

4.5—DESIGN REQUIREMENTS FOR SINGLE-SPAN BRIDGES

A detailed seismic analysis should not be required for single-span bridges regardless of SDC as specified in Article 4.1. However, the connections between the bridge span and the abutments shall be designed both longitudinally and transversely to resist a horizontal seismic force not less than the acceleration coefficient, A_S, as specified in Article 3.4, times the tributary permanent load, except as modified for SDC A in Article 4.6. The lateral force shall be carried into the foundation in accordance with Articles 5.2 and 6.7. The minimum support lengths shall be as specified in Article 4.12.

C4.5

Requirements for single-span bridges are not as rigorous as for multispan bridges because of their favorable response to seismic loads in past earthquakes. As a result, single-span bridges need not be analyzed for seismic loads regardless of the SDC, and design requirements are limited to minimum support lengths and connection forces. Adequate support lengths are required in both the transverse and longitudinal directions. Connection forces are based on the premise that the bridge is very stiff and that the fundamental period of response will be short. This assumption acknowledges the fact that the period of vibration is difficult to calculate because of significant interaction with the abutments.

These reduced requirements are also based on the assumption that there are no vulnerable substructures (i.e., no columns) and that a rigid (or near-rigid) superstructure is in place to distribute the in-plane loads to the abutments. If, however, the superstructure is not able to act as a stiff diaphragm and sustains significant in-plane deformation during horizontal loading, it should be analyzed for these loads and designed accordingly.

Although not covered by these Guide Specifications, single-span trusses may be sensitive to in-plane loads, and the Designer may need to take additional precautions to ensure the safety of truss superstructures.

4.6—DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

For bridges in SDC A, where the acceleration coefficient, A_S, as specified in Article 3.4, is <0.05, the horizontal design connection force in the restrained directions shall not be less than 0.15 times the vertical reaction due to the tributary permanent load.

For all other sites in SDC A, the horizontal design connection force in the restrained directions shall not be <0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads, if applicable, 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 SDC A and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

The minimum support length for bridges in SDC A shall be as specified in Article 4.12.

C4.6

These provisions arise because, as specified in Articles 4.1 and 4.2, seismic analysis for bridges in SDC A is not generally required. These default values are used as minimum design forces in lieu of rigorous analysis. The division of SDC A at an acceleration coefficient of 0.05 recognizes that, in parts of the country with very low seismicity, seismic forces on connections are very small.

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.

4.7—DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORIES B, C, AND D

4.7.1—Design Methods for Lateral Seismic Displacement Demands

For design purposes, each structure shall be categorized according to its intended structural seismic response in terms of damage level (i.e., ductility demand, μ_D, as specified by Eq. 4.9-5). The following design methods are further defined as follows:

- *Conventional Ductile Response (i.e., Full-Ductility Structures)*: For horizontal loading, a plastic mechanism is intended to develop. The plastic mechanism shall be defined clearly as part of the design strategy. Yielding may occur in areas that are not readily accessible for inspection with Owner's approval. Inelastic action is intended to be restricted to flexural plastic hinges in columns and pier walls and inelastic soil deformation behind abutment walls and wingwalls. Details and member proportions shall ensure large ductility capacity, μ_C, under load reversals without significant strength loss with ductility demands ($4.0 \leq \mu_D \leq 6.0$; see Article 4.9). This response is anticipated for a bridge in SDC D designed for the life safety criteria.

- *Limited-Ductility Response*: For horizontal loading, a plastic mechanism as described above for full-ductility structures is intended to develop, but in this case for limited-ductility response, ductility demands are reduced ($\mu_D \leq 4.0$). Intended yielding shall be restricted to locations that are readily accessible for inspection following a design earthquake unless prohibited by the structural configuration. Inelastic action is intended to be restricted to flexural plastic hinges in columns and pier walls and inelastic soil deformation behind abutment walls and wingwalls. Detailing and proportioning requirements are less than those required for full-ductility structures. This response is anticipated for a bridge in SDC B or C.

- *Limited-Ductility Response in Concert with Added Protective Systems*: In this case, a structure has limited ductility with the additional seismic isolation, passive energy-dissipating devices, and/or other mechanical devices to control seismic response. Using this strategy, a plastic mechanism may or may not form. The occurrence of a plastic mechanism shall be verified by analysis. This response may be used for a bridge in SDC C or D designed for an enhanced performance. Nonlinear time history analysis (i.e., Procedure 3) may be required for this design strategy.

C4.7.1

A key element in the design procedure is the flexural capacity of the columns. Philosophically, the lower the flexural capacity of the column, the more economical will be the seismic design because the overstrength flexural capacity of a column drives the cost and capacity of both the foundations and connections to the superstructure. For SDC B, the capacity of the column designed for nonseismic loads is considered to be acceptable for this lower seismic hazard level.

For SDCs C and D, the design procedure provides a trade-off between acceptable design displacements and minimum flexural capacities of columns, which could in turn be governed by P-Δ effects.

4.7.2—Vertical Ground Motion, Design Requirements for SDC D

The effects of vertical ground motions for bridges in SDC D located within 6 mi of an active fault, as described in Article C3.4, should be considered for essential and critical bridges.

C4.7.2

The most comprehensive study (Button et al., 1999) performed to date on the impact of vertical acceleration effects indicates that for some design parameters (superstructure moment and shear and column axial forces) and for some bridge types, the impact can be significant. The study was based on vertical response spectra developed by Silva (1997) from recorded western United State ground motions.

Specific recommendations for assessing vertical acceleration effects are not provided in these Guide Specifications until more information is known about the characteristics of vertical ground motion in the central and eastern United States and those areas affected by subduction zones in the Pacific. However, it is advisable for Designers to be aware that vertical acceleration effects may be important and should be assessed for essential and critical bridges. See Caltrans Seismic Design Criteria (Caltrans, 2006).

4.8—STRUCTURE DISPLACEMENT DEMAND/ CAPACITY FOR SDCS B, C, AND D

For SDCs B, C, and D, each bridge bent shall satisfy:

$$\Delta_D^L < \Delta_C^L \tag{4.8-1}$$

where:

Δ_D^L = displacement demand taken along the local principal axis of the ductile member. The displacement demand may be conservatively taken as the bent displacement inclusive of flexibility contribution from the foundations, superstructure, or both. See Figures 4.8-1 and 4.8-2.

Δ_C^L = displacement capacity taken along the local principal axis corresponding to Δ_D^L of the ductile member as determined in accordance with Article 4.8.1 for SDCs B and C and in accordance with Article 4.8.2 for SDC D (in.).

Eq. 1 shall be satisfied in each of the local axes of every bent. The local axis of a bent typically coincides with the principal axis of the columns in that bent.

The formulas presented below are used to obtain Δ_C^L for SDCs B and C. These formulas are not intended for use with configuration of bents with struts at mid-height. A more detailed pushover analysis is required to obtain Δ_C^L for SDC D, as described in Article 4.8.2. For pier walls, a displacement demand to capacity check in the transverse direction is not warranted, provided requirements of Article 8.6.9 are satisfied.

C4.8

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 from either 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 interaction between axial force and moment 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 (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. Expected material properties are used for the displacement capacity verification.

The definition of displacement demand permits the inclusion of foundation and superstructure flexibilities for SDC B and C as a matter of analytical convenience so that global analysis results can be used directly after resolution to the local substructure axes. This is conservative.

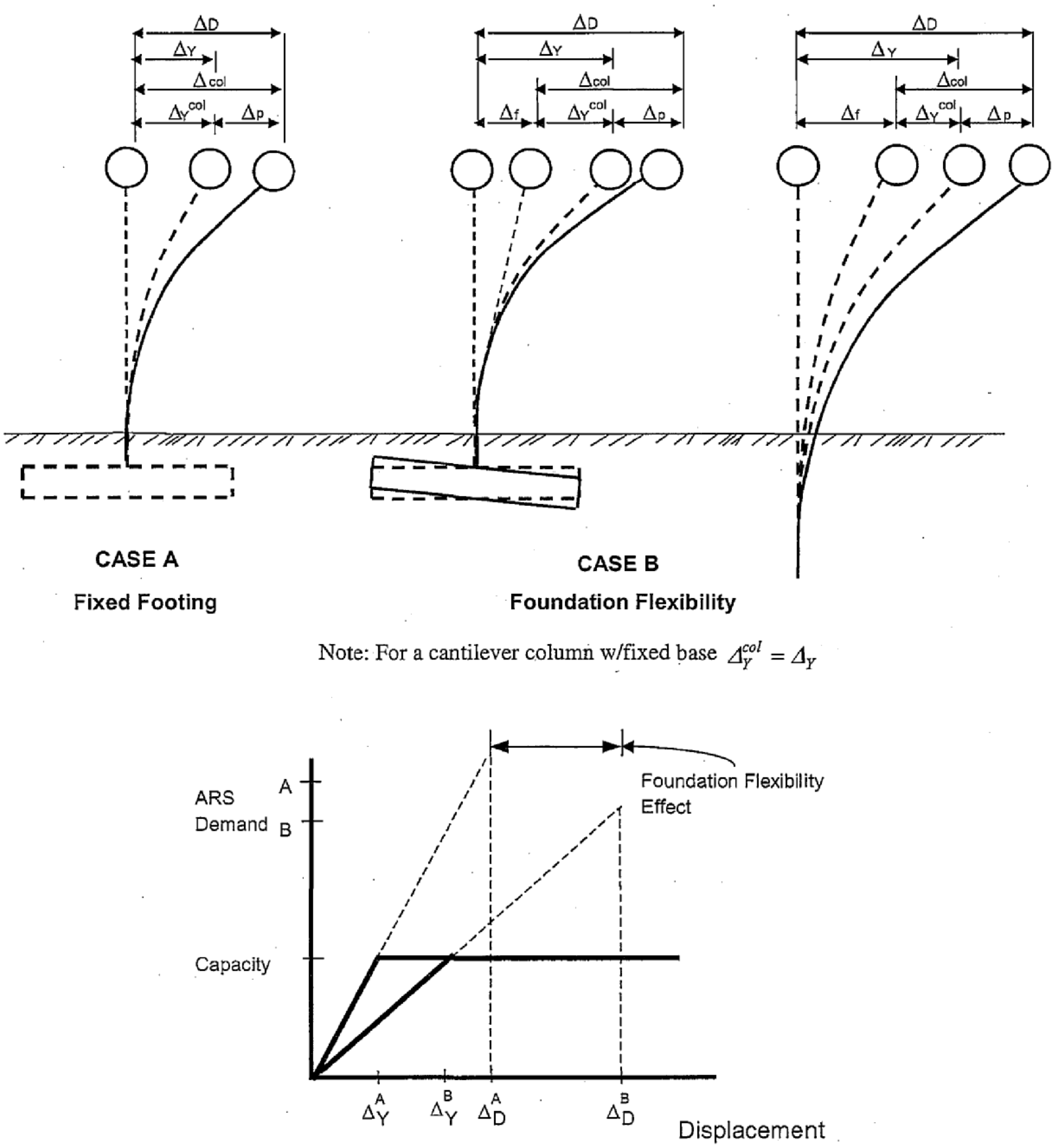


Figure 4.8-1—Effects of Foundation Flexibility on the Force-Deflection Relation for a Single Column Bent (Caltrans, 2006)

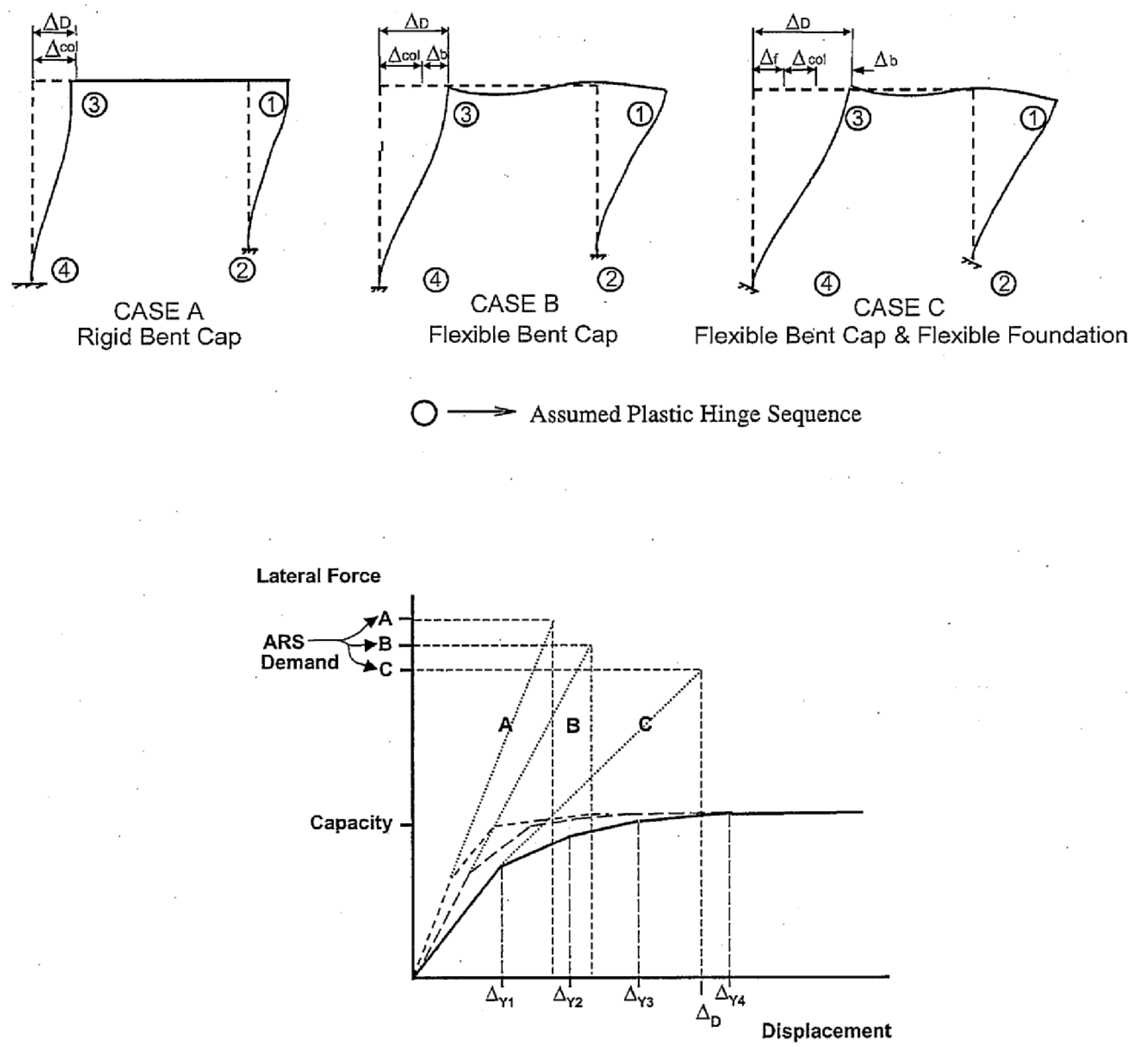


Figure 4.8-2—Effects of Foundation and Bent Cap Flexibilities on the Force-Deflection Relation for a Bent Frame (Caltrans, 2006)

4.8.1—Local Displacement Capacity for SDCs B and C

For Type 1 structures, as specified in Article 3.3, comprising reinforced concrete columns in SDCs B and C, the displacement capacity, Δ_C^L in in., of each bent may be determined from the following approximation:

For SDC B:

$$\Delta_C^L = 0.12 H_o \left(-1.27 \ln(x) - 0.32\right) \geq 0.12 H_o \qquad (4.8.1\text{-}1)$$

For SDC C:

$$\Delta_C^L = 0.12 H_o \left(-2.32 \ln(x) - 1.22\right) \geq 0.12 H_o \qquad (4.8.1\text{-}2)$$

in which:

C4.8.1

Eqs. 1 to 3 are primarily intended for determining displacement capacities of bridges with single- and multiple-column reinforced concrete piers for which there is no provision for fusing or isolation between the superstructure and substructure during design event accelerations. The equations are also calibrated for columns that have clear heights that are greater than or equal to about 15 ft in height and where plastic hinging is anticipated above ground.

For bridges with pier types other than those described above, anomalous results can be obtained. However, if the pier types in a bridge are sufficiently analogous to single- or multiple-column reinforced concrete bents, Eqs. 1 to 3 may be used to compute displacement capacity. An example is bridges with bents comprising single or multiple drilled shaft columns in which plastic hinging may occur below ground such that the clear height dimension would begin at the point of fixity in the soil.

$$x = \frac{\Lambda B_o}{H_o} \tag{4.8.1-3}$$

where:

H_o = clear height of column (ft)

B_o = column diameter or width measured parallel to the direction of displacement under consideration (ft)

Λ = factor for column end restraint condition

= 1 for fixed-free (pinned on one end)

= 2 for fixed top and bottom

For a partially fixed connection on one end, interpolation between 1 and 2 is permitted for Λ. Alternatively, H_o may be taken as the shortest distance between the point of maximum moment and point of contraflexure, and Λ may be taken as 1.0 when determining x using Eq. 3.

For bridge bents or frames that do not satisfy Eq. 4.8-1 or are not Type 1 reinforced concrete structures, the Designer may either:

- Increase the allowable displacement capacity, Δ_C^L, by meeting detailing requirements of a higher SDC as described in Article 3.5, or
- Adjust the dynamic characteristics of the bridge as described in Article 4.1.

If the piers in a bridge are not sufficiently analogous to single- or multiple-column reinforced concrete bents, Article 4.8.2 should be used to compute displacement capacity. An example is bridges with solid wall piers founded on piles with a cap in the strong direction.

Eqs. 1 and 2 were developed by Imbsen (2006) for columns that were either fixed or pinned at their ends. The equations are based on the database results reported by Berry and Eberhard (2003). Therefore, the capacities reported by these equations do not include foundation, capbeam, or superstructure flexibility effects. Capacities from these equations will normally be conservative relative to demands that include such flexibilities. In such cases, removing the demand displacement component attributable to such flexibilities from the local displacement demand may produce an acceptable result, in lieu of making a higher category check or redesigning the system.

Eq. 1 generally corresponds to a limit state of initiation of concrete cover spalling, and Eq. 2 corresponds to an equivalent column member ductility of 3 or less. These limits are generally conservative in terms of estimating the displacement capacity of reinforced concrete elements detailed in accordance with the applicable provisions of either SDC B or C.

Where in-ground hinging is used as an ERE in SDC C, the Designer should consider using Eq. 1 to limit the damage that could occur below grade and that would thus be uninspectable. Alternately, the method and limits for SDC D should be used.

4.8.2—Local Displacement Capacity for SDC D

The Nonlinear Static Procedure (NSP), commonly referred to as "pushover" analysis, shall be used to determine the reliable displacement capacities of a structure or frame as it reaches its limit of structural stability. Displacement capacity determined for SDC C may be used in lieu of pushover analysis. NSP is an incremental linear analysis that captures the overall nonlinear behavior of the elements, including soil effects, by pushing them laterally to initiate plastic action. Each increment of loading pushes the frame laterally, through all possible stages, until the potential collapse mechanism is achieved.

Because the analytical model used in the pushover analysis accounts for the redistribution of internal actions as components respond inelastically, NSP is expected to provide a more realistic measure of behavior than may be obtained from elastic analysis procedures.

Where foundation and superstructure flexibility can be ignored as stipulated in Article 5.3.1, the two-dimensional plane frame "pushover" analysis of a bent or a frame can be simplified to a column model (fixed-fixed or fixed-pinned) if it does not cause a significant loss in accuracy in estimating the displacement capacities.

The effect of seismic load path on the column axial load and associated member capacities shall be considered in the simplified model.

C4.8.2

This design procedure is a key element in the philosophic development of these Guidelines. The pushover method of analysis has seen increasing use throughout the 1990s, especially in Caltrans' seismic retrofit program. This analysis method provides additional information on the expected deformation demands of columns and foundations and as such provides the Designer with a greater understanding of the expected performance of the bridge. The pushover method of analysis is used in two ways. First, it encourages Designers to be as liberal as possible with assessing ductility capacity. Second, it provides a mechanism to allow EREs that need the Owner's approval (Article 3.3). The trade-off was the need for a more sophisticated analysis so that the expected deformations in critical elements could be assessed. Provided the appropriate limits (i.e., plastic rotations for in-ground hinges) are met, the EREs requiring the Owner's approval can be used. This method applies to all the EREs shown in the figures of Article 3.3.

4.9—MEMBER DUCTILITY REQUIREMENT FOR SDC D

In addition to the requirements of Article 4.8, individual member ductility demand, μ_D, shall satisfy:

For single-column bents:

$$\mu_D \leq 5 \tag{4.9-1}$$

For multiple-column bents:

$$\mu_D \leq 6 \tag{4.9-2}$$

For pier walls in the weak direction:

$$\mu_D \leq 5 \tag{4.9-3}$$

For pier walls in the strong direction:

$$\mu_D \leq 1 \tag{4.9-4}$$

in which:

$$\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_{yi}} \tag{4.9-5}$$

where:

Δ_{pd} = plastic displacement demand (in.)

Δ_{yi} = idealized yield displacement corresponding to the idealized yield curvature, φ_{yi}, shown in Figure 8.5-1 (in.)

Pile shafts should be treated similarly to columns. Where in-ground hinging is used for the ERE, the member ductility limits shall satisfy:

- For reinforced concrete members such as drilled shafts, cast-in-place piles, and prestressed piles subject to in-ground hinging:

$$\mu_D \leq 4 \tag{4.9-6}$$

4.10—COLUMN SHEAR REQUIREMENTS FOR SDCS B, C, AND D

For SDC B, C, or D, shear design requirements for reinforced concrete columns shall be satisfied according to Article 8.6. Determination of member ductility demand should be required for SDC D only, as stipulated in Article 8.6.2.

C4.9

Individual member displacements such as column displacements, Δ_{col}, are defined as the portion of global displacement attributed to the elastic column idealized displacement Δ_{yi} and plastic displacement demand Δ_{pd} of an equivalent member from the point of maximum moment to the point of contraflexure. Member section properties are obtained from a moment-curvature analysis and used to calculate Δ_{yi} and the plastic displacement capacity Δ_{pc}.

In lieu of the provided ductility limits, the Designer should consider target ductilites of 4, 5, and 4, respectively, for the cases covered by Eqs. 1 through 3, and a target of 3 for Eq. 6 when designing ductile members such that redesign is not required should small changes be made in subsequent phases of the design.

In calculating the yield displacement and plastic displacement demand, any contribution to the displacements from foundation, capbeam, or superstructure should be removed. Inclusion of such flexibilities in the yield displacement is unconservative, as illustrated in Figures 4.8-1 and 4.8-2.

Where in-ground plastic hinging is part of the plastic mechanism, the effective column length used for yield and plastic deformations should extend to the in-ground hinge. Foundation flexibilities below the expected hinge should then be removed, as described above.

In cases where in-ground hinges could be inspected with excavation—for example, the tops of piles in an integral abutment—the ductility limits provided in Eqs. 1 to 3 may be used at the discretion of the Owner.

4.11—CAPACITY DESIGN REQUIREMENT FOR SDCS B, C, AND D

4.11.1—Capacity Design

Capacity design principles require that those components not participating as part of the primary energy-dissipating system, typically flexural hinging in columns above ground or in some cases flexural hinging of drilled shafts, solid wall encased pile bents, etc., below ground, shall be capacity protected. The components include the superstructure, joints and cap beams, spread footings, pile caps, and foundations. This is achieved by ensuring the maximum moment and shear from plastic hinges in the column, considering overstrength can be resisted elastically by adjoining elements.

For SDC B, forces obtained from capacity design principles should be used when the plastic hinging forces are less than the forces obtained from an elastic analysis. In lieu of full capacity design using overstrength forces, capacity checks should be made to ensure that no weak links exist in the ERS. Joint shear checks are not required.

For SDC C or D, exception to capacity design is permitted for the following:

- The seismic resisting system includes the fusing effects of an isolation device (Type 3 global design strategy),
- A ductile end diaphragm is incorporated into the transverse response of a steel superstructure (Type 2 global design strategy; see Article 7.2.2), and
- A foundation situated in soft or potentially liquefiable soils where plastic hinging is permitted below ground.

C4.11.1

The objective of these provisions for conventional design is that inelastic deformation (plastic hinging) occurs at the location in the columns (top or bottom or both) where they can be readily inspected and repaired. To achieve this objective, all members connected to the columns, the shear capacity of the column, and all members in the load path from the superstructure to the foundation, should be capable of transmitting the maximum (overstrength) force effects developed by plastic hinges in the columns. The exceptions to the need for capacity design of connecting elements are:

- Where seismic isolation design is used, and
- In the transverse direction of columns when a ductile diaphragm is used.

For SDC B, full capacity design is not required. However, it is good practice to identify and check the lateral load path to ensure that no portions of the load path are weaker than the elements that establish the plastic capacity of the substructure (e.g., plastic hinging in the columns). Therefore, checks of the load path are suggested, but full capacity design using plastic overstrength forces is not required. Likewise, joint shear checks are not required. This more liberal practice is appropriate because the ductility demands in SDC B substructure elements are generally expected to be less than about 2. This, in part, is ensured by the lower displacement capacity provided in Article 4.8.1.

The design of substructures using the elastic forces associated with the design ground motion is permitted in SDC B, although not encouraged, because potentially larger ground motions could produce undesirable performance, including collapse or partial collapse of the bridge. Use of ductile design with capacity design principles provides margin against poor performance in ground motions larger than the design level due to the inherent displacement capacity in a suitably ductile structure.

4.11.2—Plastic Hinging Forces

Plastic hinges shall form before any other failure due to overstress or instability in the overall structure and/or in the foundation. Except for pile bents and drilled shafts, and with the Owner's approval, plastic hinges shall be permitted only at locations in columns where they can be readily inspected and/or repaired, as described in Article 3.3.

Superstructure and substructure components and their connections to columns that are designed not to yield shall be designed to resist overstrength moments and shears of ductile columns. Except for the geotechnical aspects for design of foundations, the overstrength moment capacity, M_{po} in kip-in., of column/pier/pile members that form part of the primary mechanism resisting seismic loads shall be assessed as specified below or by using the applicable provisions of Sections 7 and 8.

- For reinforced concrete members:

$$M_{po} = \lambda_{mo} M_p \quad (4.11.2\text{-}1)$$

where:

M_p = plastic moment capacity of column (kip-in.)

λ_{mo} = overstrength factor taken as 1.2 or 1.4 as determined from Article 8.5

- For steel members:

$$M_{po} = \lambda_{mo} M_n \quad (4.11.2\text{-}2)$$

where:

M_n = nominal moment strength for which expected steel strengths for steel members are used (kip-in.)

λ_{mo} = overstrength factor taken as 1.2 as determined from Article 7.3

The plastic moment capacity, M_p, for reinforced concrete columns shall be determined using a moment-curvature section analysis, taking into account the expected yield strength of the materials, the confined concrete properties, and the strain-hardening effects of the longitudinal reinforcement.

In SDC B, it is acceptable to use the moment capacity based on the expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003 as M_p.

The overstrength moments and associated shear forces, calculated on the basis of inelastic hinging at overstrength, shall be taken as the extreme seismic forces that the bridge is capable of resisting. Typical methods of applying capacity design at a bent in the longitudinal and transverse directions are shown in Figure 1 and illustrated in Article 4.11.3 for single-column bents and Article 4.11.4 for multicolumn bents.

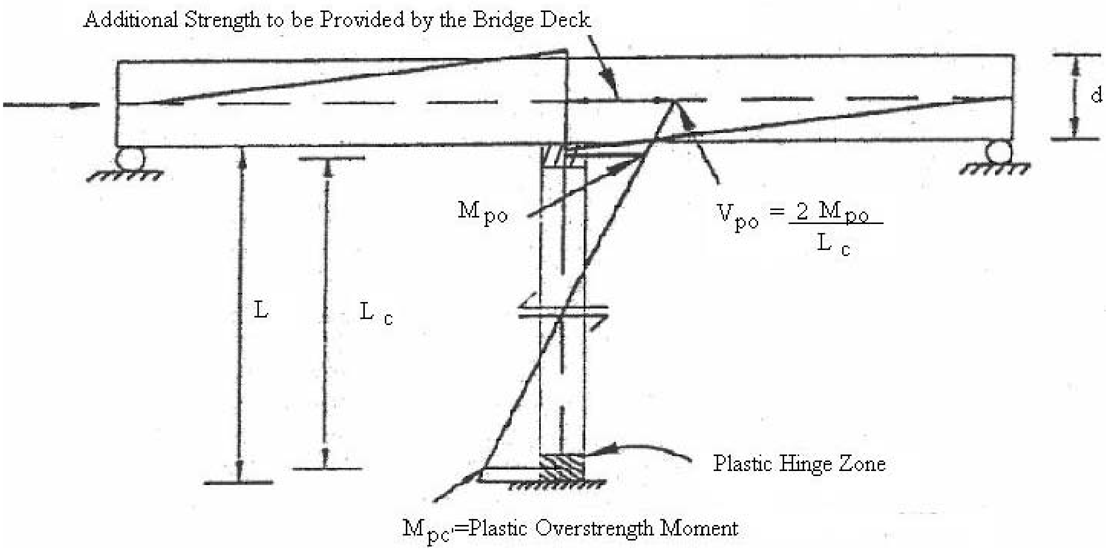
C4.11.2

The principles of capacity design require that the strength of those members that are not part of the primary energy-dissipating system be stronger than the overstrength capacity of the primary energy-dissipating members (i.e., the columns with hinges at their member ends).

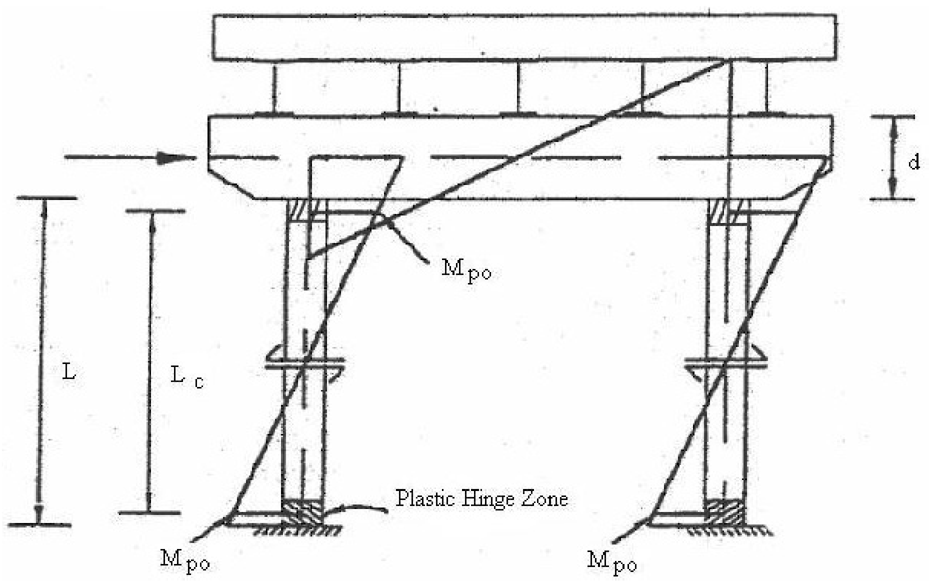
When assessing overstrength capacity of flexural members using compatibility section analysis (i.e., the moment-curvature method), it is important to differentiate between overstrength resulting from the response of the section to high curvature demands and overstrength resulting from upper bound material properties.

For example, for reinforced concrete columns, confined concrete will have enhanced capacity and reinforcing steel will strain-harden at high plastic curvatures. This will result in increased flexural capacity of the column that will be captured by a moment-curvature analysis that considers these factors. In addition, reinforcing steel can have a higher than nominal yield point, and concrete is likely to be stronger than specified and will gain strength with age beyond the 28-day specified strength (ATC, 1996).

Likewise, when assessing structures in which the locations of plastic hinging may not be clearly defined, a conservative estimate of the hinging locations should be used in order to generate conservative forces for the capacity design process. For example, drilled shafts should be designed to resist the plastic shear with hinging at the top of the shaft, even if in-ground hinging is expected under lower bound soil resistances. Similarly, pile bents, where hinging must occur in ground before a complete plastic mechanism forms, should be designed for a conservative estimate of plastic hinging locations.



(a) Longitudinal Response for Nonintegral Abutments



(b) Transverse Response for Dual Column Pier

Figure 4.11.2-1—Capacity Design of Bridges Using Overstrength Concepts

4.11.3—Single Columns and Piers

Column design shear forces and moments in the superstructure, bent caps, and the foundation structure 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 capacities. Use an overstrength factor times the plastic moment capacity or nominal moment as specified in Article 4.11.2. The nominal moment or plastic moment capacity members are calculated using the expected yield strengths and subjected to the applied dead load on the section under consideration. Column overstrength moments should be distributed to the connecting structural elements. (Exception: When calculating the design forces for the geotechnical aspects of foundations, such as determining lateral stability or tip elevation, use an overstrength factor of 1.0 on the nominal moment.)
- *Step 2.* Using the column overstrength moments, calculate the corresponding column shear force assuming a quasi-static condition. For flared columns designed to be monolithic with superstructure or with isolation gaps less than required by Article 8.14, the shear shall be calculated as the greater shear obtained from using:
    - The overstrength moment at both the top of the flare and the top of the foundation with the appropriate column height, or
    - The overstrength moment at both the bottom of the flare and the top of the foundation with the reduced column height.
- *Step 3.* Calculate forces in the superstructure for longitudinal direction loading and forces in the foundation for both longitudinal and transverse loading.

4.11.4—Bents with Two or More Columns

The forces for bents with two or more columns shall be calculated 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 calculated as for single columns in Article 4.11.3. In the plane of the bent, the forces shall be calculated as follows:

- *Step 1.* Determine the column overstrength moment capacities. Use an overstrength factor times the plastic moment capacity or nominal moment as specified in Article 4.11.2. The nominal moment or plastic moment capacity for members is calculated using the expected yield strengths and subjected to the applied dead load on the section under consideration.
- *Step 2.* Using the column overstrength moments, calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the bent. If a partial-height wall exists between the columns, the effective column height is taken from the top of the wall. For flared columns and foundations below ground level, see Article 4.11.3—Step 2.
- *Step 3.* Apply the bent shear force to the top of the bent (center of mass of the superstructure above the bent) and determine the axial forces in the columns due to overturning when the column overstrength moments are developed.
- *Step 4.* Using these column axial forces combined with the dead load axial forces, determine revised column overstrength moments. With the revised overstrength moments, 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 load is the dead load plus or minus the axial load determined from the final iteration of *Step 3.*
- *Moments:* The column overstrength plastic moments or overstrength nominal moment (Article 4.11.2) corresponding to the maximum compressive axial load specified above (in the previously bulleted item).
- *Shear Force:* The shear force corresponding to the final column overstrength moments in *Step 4* above.

Calculate forces in the superstructure for both longitudinal and transverse direction loading and forces in the foundation for both longitudinal and transverse loading.

4.11.5—*P*-Δ Capacity Requirement for SDCs C and D

P-Δ effects may be ignored in the analysis and design of Type 1 structures (see Article 3.3) if the following is satisfied.

- For reinforced concrete columns:

$$P_{d\ell}\Delta_r \le 0.25M_p \qquad (4.11.5\text{-}1)$$

- For steel columns:

$$P_{d\ell}\Delta_r \le 0.25M_n \qquad (4.11.5\text{-}2)$$

where:

$P_{d\ell}$ = unfactored dead load acting on the column (kip)

Δ_r = relative lateral offset between the point of contraflexure and the furthest end of the plastic hinge (in.)

M_p = idealized plastic moment capacity of reinforced concrete column based on expected material properties (kip-in.)

M_n = nominal moment capacity of structural steel column based on nominal material properties (kip-in.)

- For a single pile shaft, Δ_r should be taken as:

$$\Delta_r = \Delta_D - \Delta_S \qquad (4.11.5\text{-}3)$$

where:

Δ_D = displacement demand as determined in accordance with Article 4.3 (in.)

Δ_S = pile shaft displacement at the point of maximum moment developed in ground (in.)

- For a pile cap in Site Classification E, or for cases in which a modal analysis shows out-of-phase movement of the bottom of the column relative to the top of the column, Δ_r shall be taken as:

$$\Delta_r = \Delta_D + \Delta_F \qquad (4.11.5\text{-}4)$$

where:

Δ_D = displacement demand as determined in accordance with Article 4.3 (in.)

Δ_F = pile cap displacement (in.)

C4.11.5

Typical highway bridges should be designed so that *P*-Δ effects can be neglected. For columns that do not satisfy Eq. 1 or 2, the Designer has the option of considering one or more of the following:

- Increasing the column moment capacity by adding longitudinal reinforcement,
- Adjusting the dynamic characteristics of the bridge as discussed in Article 4.1.3,
- Reconfiguring the bridge to reduce the dead load demand acting on the column, or
- Using nonlinear time history analysis to explicitly consider *P*-Δ effects.

At this time, the only rigorous method for considering *P*-Δ effects in combination with seismic demands is to use a nonlinear time history analysis. When using nonlinear time history analysis, post-yield stiffness, stiffness degradation, and unloading stiffness models that are capable of capturing the expected structure response due to seismic-induced cyclic loading are required. Due to the complexity of this type of analysis, it is recommended that the requirements of Eq. 1 and Eq. 2 be satisfied whenever practical.

When the requirements of Eq. 1 or Eq. 2 are not satisfied, P-Δ effects shall be included in the design using a nonlinear time history analysis as specified in Procedure 3 of Article 4.2.

4.11.6—Analytical Plastic Hinge Length

The analytical plastic hinge length for columns, L_p, shall be taken as the equivalent length of column over which the plastic curvature is assumed constant for estimating the plastic rotation. The plastic displacement of an equivalent member from the point of maximum moment to the point of contraflexure shall be determined on the basis of the plastic rotation.

For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, cased shaft, the plastic hinge length, L_p in inches, may be determined as:

$$L_p = 0.08L + 0.15 f_{ye} d_{b\ell} \geq 0.3 f_{ye} d_{b\ell} \tag{4.11.6-1}$$

where:

L = length of column from point of maximum moment to the point of moment contraflexure (in.)

f_{ye} = expected yield strength of longitudinal column reinforcing steel bars (ksi)

$d_{b\ell}$ = nominal diameter of longitudinal column reinforcing steel bars (in.)

For noncased prismatic cast in drilled hole shafts, reinforced concrete piles, and prestressed concrete piles, the below-ground analytical plastic hinge length, L_p in in., may be determined as:

$$L_p = 0.1H' + D^* \leq 1.5D^* \tag{4.11.6-2}$$

where:

D^* = diameter of circular shafts or cross-section dimension in direction under consideration for oblong shafts (in.)

H' = length of shaft from the ground surface to point of contraflexure above ground (in.)

For horizontally isolated flared reinforced concrete columns, the plastic hinge length, L_p in in., may be determined as:

$$L_p = G_f + 0.3 f_{ye} d_{b\ell} \tag{4.11.6-3}$$

C4.11.6

A simplifying assumption used when calculating plastic deformations of columns is that the plastic curvature is constant over the analytical plastic hinge length, L_p. The analytical plastic hinge has been calibrated to give the same plastic hinge rotation as that occurring in the actual structure (Priestley et al., 1996).

Eq. 1 also applies for the upper plastic hinge in prestressed concrete pile bents, because this region is essentially conventionally reinforced until the prestressing strand is fully developed in the pile. Additionally, in SDC C or D, development of prestressing strand into the cap is not permitted.

For columns with longitudinal reinforcing that anchors into members such as footings, bent caps, oversized shafts, or cased shafts, the analytical plastic hinge length is assumed to be composed of two principal components.

The first component accounts for moment gradient, tension shift, and other force effects. For typical bridge columns reinforced with materials addressed in Table 8.4.2-1, the first component is taken to be $0.08L$. Bridge columns reinforced with material other than those addressed in Table 8.4.2-1 are not explicitly addressed in these Guide Specifications.

The second component of the analytical plastic hinge length accounts for the strain penetration of the longitudinal reinforcing steel into the connecting element (footing, cap, etc.).

Chai (2002) developed relationships for below-ground analytical plastic hinge lengths. Chai recommended $1.6D$ as the upper limit of the below-ground analytical plastic hinge length. Due to uncertainties in soil and material properties as well as the desire to limit below-ground damage, an upper limit of $1.5D$ is specified.

where:

G_f = gap between the isolated flare and the soffit of the bent cap (in.)

f_{ye} = expected yield strength of longitudinal column reinforcing steel bars (ksi)

$d_{b\ell}$ = nominal diameter of longitudinal column reinforcing steel bars (in.)

For concrete filled pipe pile extensions that comply with the requirement of Article 7.6, the below-ground plastic hinge length, L_p in in., may be determined as:

$$L_p = 0.1H' + 1.25D \leq 2D \quad (4.11.6\text{-}4)$$

where:

D = diameter of concrete filled pipe (in.)

H' = length of pile from point of the ground surface to point of contraflexure above ground (in.)

The relationships for below-ground analytical plastic hinge lengths of concrete filled pipe piles are similar to those of conventional reinforced concrete shafts. The Port of Long Beach Wharf Design Criteria (2007) and research by others suggests that the below-ground analytical plastic hinge length be taken as $2D$.

4.11.7—Reinforced Concrete Column Plastic Hinge Region

Enhanced lateral confinement shall be provided in a column, pier, or shaft over the plastic hinge region, L_{pr}. L_{pr} shall be taken as the larger of:

- 1.5 times the gross cross-sectional dimension in the direction of bending,
- The region of column where the moment demand exceeds 75 percent of the maximum plastic moment, or
- The analytical plastic hinge length L_p.

4.11.8—Steel Column Plastic Hinge Region

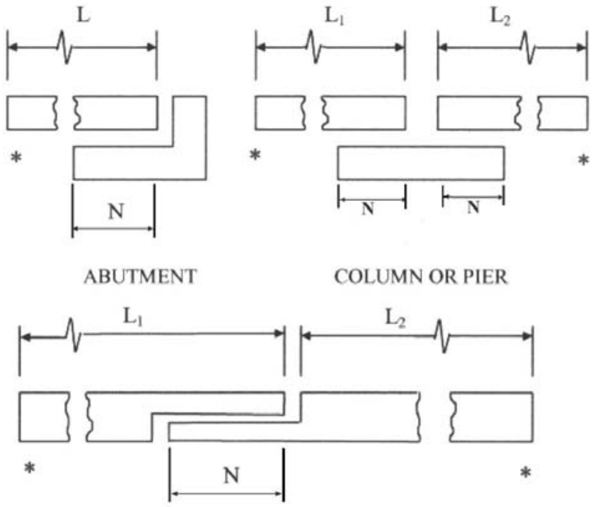
In the absence of any experimental or analytical data that support the use of a plastic hinge length for a particular cross-section, the plastic hinge region length for steel columns shall be taken as:

- the maximum of $^1/_8$ of the clear height of a steel column or
- 1.5 times the gross cross-sectional dimension in the direction of bending

4.12—MINIMUM SUPPORT LENGTH REQUIREMENTS

4.12.1—General

Minimum support length as determined in this Article shall be provided for girders supported on an abutment, bent cap, pier wall, or a hinge seat within a span as shown in Figure 1.



* Expansion Joint or End of Bridge Deck

Figure 4.12.1-1—Support Length, *N*

4.12.2—Seismic Design Categories A, B, and C

Support lengths at expansion bearings without shock transmission units (STUs) or dampers shall be designed to either accommodate the greater of the maximum calculated displacement, except for bridges in SDC A, or a percentage of the empirical support length, *N*, specified by Eq. 1. The percentage of *N*, applicable to each SDC, shall be as specified in Table 1.

$$N = (8 + 0.02L + 0.08H)(1 + 0.000125S^2) \qquad (4.12.2\text{-}1)$$

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)

C4.12.2

Minimum support length provisions provided in this Article are equivalent to the *AASHTO LRFD Bridge Design Specifications* Article 4.7.4.4.

Support lengths are equal to the length of the overlap between the girder and the seat as shown in Figure 4.12.1-1. To satisfy the minimum value for *N* in this Article, the overall seat width will be larger than *N* by an amount equal to movements due to prestress shortening, creep, shrinkage, and thermal expansion and contraction. The minimum values for *N* given in Eq. 1 include an arbitrary allowance for cover concrete at the end of the girder and face of the seat. If above average cover is used at these locations, *N* should be increased accordingly.

H = for abutments, average height of columns supporting the bridge deck from the abutment 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 = angle of skew of support measured from a line normal to span (°)

Table 4.12.2-1—Percentage *N* by SDC and Acceleration Coefficient, A_s

SDC	Acceleration Coefficient, A_s	Percentage N
A	< 0.05	≥ 75
A	≥ 0.05	100
B	All applicable	150
C	All applicable	150

4.12.3—Seismic Design Category D

For SDC D, hinge seat or support length, N, shall be available to accommodate the relative longitudinal earthquake displacement demand at the supports or at the hinge within a span between two frames and shall be determined as:

$$N = \left(4 + 1.65\Delta_{eq}\right)\left(1 + 0.00025S^2\right) \geq 24 \qquad (4.12.3\text{-}1)$$

where:

Δ_{eq} = seismic displacement demand of the long period frame on one side of the expansion joint (in.). The elastic displacement demand shall be modified according to Articles 4.3.2 and 4.3.3.

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

The skew effect multiplier, $(1 + 0.00025S^2)$, may be set equal to 1 when the global model of the superstructure is modeled to include the full width and the skew effects on the displacement demands at the outer face of the superstructure.

C4.12.3

Support length requirements are based on the rigorous analysis required for SDC D. As such, support lengths determined for SDC D may be less than those determined using Article 4.12.2.

4.13—SUPPORT RESTRAINTS

Support restraints may be provided for longitudinal linkage at expansion joints within the space and at adjacent sections of simply supported superstructures. Their use is intended to achieve an enhanced performance of the expansion joint and shall be approved and satisfy Owner requirements. For continuous superstructures spans, restrainers used to minimize displacements (i.e., tune the out-of-phase displacement response) between the frames of a multiframe system shall be considered secondary in reducing the out-of-phase motions at the expansion joints between the frames. Restrainer units shall be designed and detailed as described in the following Articles.

4.13.1—Longitudinal Restrainers

Restrainers shall be designed using criteria prescribed by the Owner.

Friction shall not be considered to be an effective restrainer.

The restrainer shall be 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.

C4.13.1

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.

In lieu of restrainers, shock transmission units may be used and designed for either the elastic force calculated according to Article 4.2 or the maximum force effects generated by inelastic hinging of the substructure as specified in Article 4.11.2.

4.13.2—Simple Span Superstructures

An elastic response analysis or simple equivalent static analysis shall be considered adequate and reliable for the design of restrainers for simple spans. An acceleration coefficient not less than that specified in Article 4.5 shall be used as a minimum.

4.13.3—Detailing Restrainers

- Restrainers shall be detailed to allow for easy inspection and replacement.
- Restrainer layout shall be symmetrical about the centerline of the superstructure.
- Restrainer systems shall incorporate an adequate gap for service conditions.
- Yield indicators may be used on cable restrainers to facilitate post-earthquake investigations.

4.14—SUPERSTRUCTURE SHEAR KEYS

The design of the superstructure and the substructure shall take into consideration the anticipated load path. For slender bents, shear keys on top of the bent cap may function elastically at the design hazard level.

In lieu of experimental test data, the overstrength shear key capacity, V_{ok}, shall be taken as:

C4.14

Shear keys are typically designed to fuse at the design event earthquake level of acceleration. Minimum requirements herein are intended to keep the keys elastic at a lower, more frequent earthquake event.

$$V_{ok} = 2V_n \tag{4.14-1}$$

where:

V_{ok} = overstrength shear key capacity used in assessing the load path to adjacent capacity-protected members (kip)

V_n = nominal interface shear capacity of shear key as defined in Article 5.8.4 of the *AASHTO LRFD Bridge Design Specifications* using the expected material properties and interface surface conditions (kip)

For shear keys at intermediate hinges within a span, the Designer shall assess the possibility of a shear key fusing mechanism, which is highly dependent on out-of-phase frame movements. For bridges in SDC D where shear keys are needed to achieve a reliable performance at the design hazard level (i.e., shear key element is part of the ERS; see Article 3.3), nonlinear analysis should be conducted to derive the distribution forces on shear keys affected by out-of-phase motions.

SECTION 5: ANALYTICAL MODELS AND PROCEDURES

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SECTION 5:
ANALYTICAL MODELS AND PROCEDURES

5.1—GENERAL

A complete bridge system may be composed of a single frame or a series of frames separated by expansion joints, articulated construction joints, or both. A bridge is composed of a superstructure and a supporting substructure.

Individual frame sections are supported on their respective substructures. Substructures consist of piers and single-column or multiple-column bents that are supported on their respective foundations.

The determination of the seismic response of a bridge includes the development of an analytical model followed by the response analysis of the analytical model to predict the resulting dynamic response for component design. Both the development of the analytical model and the selected analysis procedure are dependent on the seismic hazard, selected seismic design strategy, and the complexity of the bridge. There are various levels or degrees of refinement in the analytical model and analytical procedures that are available to the Designer.

C5.1

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. A displacement capacity determination of piers and bents is required for SDCs B, C, and D.

The objective of a displacement capacity determination is to determine the displacement of an individual pier when its deformation capacity (that of the inelastic earthquake-resisting element) is reached. The displacement capacity should 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 Designer 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.

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 defines the requirements for the foundation modeling in the seismic analysis.

When either lateral soil movement due to liquefaction or loss in lateral soil support from liquefaction is determined to be possible, the model should represent the change in support conditions and additional loads on the substructure associated with loss in soil support or soil movement.

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

5.1.1—Analysis of a Bridge ERS

The entire bridge earthquake resistant system (ERS) for analysis purposes is referred to as the “global” model, whereas an individual bent or column is referred to as a “local” model. The term “global response” describes the overall behavior of the bridge system, including the effects of adjacent components, subsystems, or boundary conditions. The term “local response” refers to the behavior of an individual component or subsystem being analyzed to determine, for example, its capacity using a pushover analysis.

Development of both global models and local models is addressed in these Guide Specifications.

Individual bridge components shall have displacement capacities greater than the displacement demands derived from the “global” analysis.

The displacement demands of a bridge system consisting of multiple simple spans may be derived using the equivalent static analysis outlined in Article 5.4.2. Global analysis requirements as specified in Article 5.1.2 need not to be applied in this case.

5.1.2—Global Model

A global model that captures the response of the entire bridge system should be developed. A global model for bridge systems with irregular geometry, in particular curved bridges and skew bridges, should have the actual geometry included. Also, multiple transverse expansion joints, massive substructure components, and foundations supported by soft soil can exhibit dynamic response characteristics that should be included in the models as–their effect on the global response is not necessarily obvious and may not be captured by a separate subsystem analysis.

Linear elastic dynamic analysis shall be used as a minimum for the global response analysis. There are, however, some limitations in a linear elastic analysis approach that should be considered. The nonlinear response of yielding columns, gapped expansion joints, earthquake restrainers, and nonlinear soil properties can only be approximated using a linear elastic approach. Piece-wise linear analysis may be used to approximate nonlinear response. Sensitivity studies using two bounding conditions may be used to approximate the nonlinear effects.

For example, two global dynamic analyses should be developed to approximate the nonlinear response of a bridge with expansion joints because it possesses different characteristics in tension and compression:

- In the tension model, the superstructure joints are permitted to move independently of one another in the longitudinal direction. Appropriate elements connecting the joints may be used to model the effects of earthquake restrainers.
- In the compression model, all of the restrainer elements are inactivated and the superstructure elements are locked longitudinally to capture structural response modes where the joints close up, mobilizing the abutments when applicable.

The determination of whether both a tension model and a compression model are required should be based on consideration of the geometry of the structure. Structures with appreciable superstructure curvature have a bias response to the outside of the curve and may require additional models that combine the characteristics identified for the tension and compression models.

Long multiframe bridges may be analyzed with multiple elastic models. A single multiframe model may not be realistic because it cannot account for out-of-phase movement among the frames.

Each multiframe model may be limited to five frames plus a boundary frame or abutment on each end of the model. Adjacent models shall overlap each other by at least one useable frame, as shown in Figure 1. A massless spring should be attached to the dead end of the boundary frames to represent the stiffness of the adjoining structure. The boundary frames provide some continuity between adjacent models but are considered redundant and their analytical results are ignored.

C5.1.2

Depending on the chosen 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 analyses, grid or finite-element analyses 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 should be included in the model. However, two-dimensional models are adequate for bridges with a skew angle less than 30° and a subtended angle of horizontal curvature less than 20°. When skew is included in a three-dimensional model, the geometry and boundary conditions at the abutments and bearings should be represented 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.

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

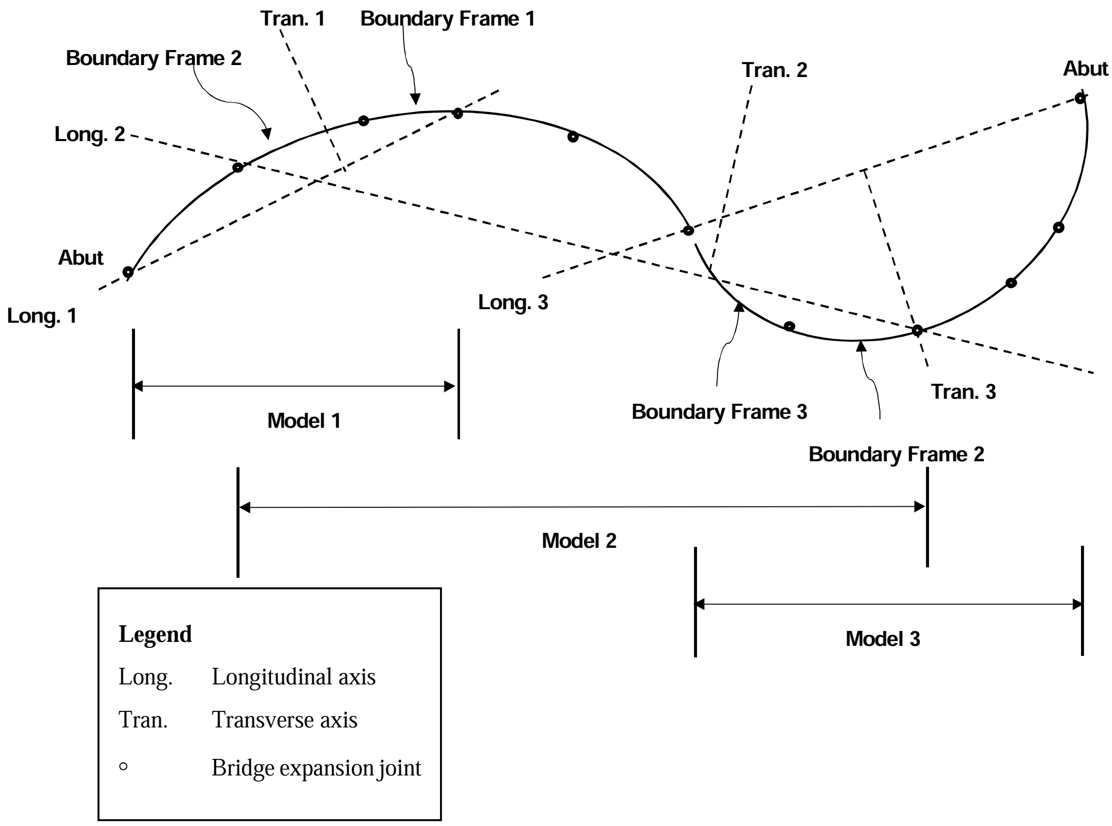


Figure 5.1.2-1—Elastic Dynamic Analysis Modeling Technique

5.2—ABUTMENTS

5.2.1—General

The model of the abutment shall reflect the expected behavior of the abutment with seismic loads applied in each of the two horizontal directions. Resistance of structural components shall be represented by cracked section properties where applicable when conducting an equivalent static analysis (ESA) or an elastic dynamic analysis (EDA).

The resistance from passive pressure of the soil embankment at the abutment wall shall be represented by a value for the secant stiffness consistent with the maximum displacement according to Article 5.2.3. Depending on the bridge configuration, one of two alternatives may be chosen by the Designer:

C5.2.1

Article 5.2 provides requirements for the modeling of abutments in the longitudinal and transverse directions. The iterative procedure with secant stiffness coefficients defined therein is included in the mathematical model 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.

Earthquake-Resisting System (ERS) without Abutment Contribution. ERS is designed to resist all seismic loads without any contribution from abutments in either orthogonal direction.

Earthquake-Resisting System (ERS) with Abutment Contribution. The ERS is designed with the abutments as a key element in one or both of the orthogonal directions. Abutments are designed and analyzed to sustain the design earthquake displacements.

For the displacement capacity verification, the strength of each component in the abutment, including soil, shall be included.

In general, the connections between the superstructure and substructure should be designed for the maximum forces that could be developed. In the spirit of capacity design, this implies that the forces corresponding to the full plastic mechanism (with yielding elements at their overstrength condition) should be used to design the connections. In cases in which the full plastic mechanism might not develop during the design earthquake, the elastic forces for this event are permitted. The minimum specified requirements not withstanding, it is often good practice to design the connections to resist the higher forces corresponding to the full plastic mechanism. It is also good practice to design for the best estimate of forces that might develop in cases such as pile bents with battered piles. In such bents, the connections should be stronger than the expected forces, and these forces may be large and may have large axial components. In such cases, the plastic mechanism may be governed by the pile geotechnical strengths rather than the pile structural strengths.

5.2.2—Wingwalls

The participation of abutment walls and wingwalls in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads may be considered in the seismic design of bridges. Damage to walls shall be considered an acceptable response during earthquakes when considering no collapse criteria. Abutment participation in the overall dynamic response of the bridge system shall reflect the structural configuration, the load-transfer mechanism from the bridge to the abutment system, the effective stiffness and force capacity of the wall–soil system, and the level of expected abutment damage. The capacity of the abutments to resist the bridge inertial load shall be compatible with the structural design of the abutment wall, that is, whether part of the wall will be damaged by the design earthquake, as well as the soil resistance that can be reliably mobilized. The lateral load capacity of walls shall be evaluated on the basis of an applicable passive earth-pressure theory.

C5.2.2

A simplistic approach that may be used is to consider one wall two-thirds effective in acting against the abutment soil fill, and the second wall is considered one-third effective in acting against the outside sloped berm.

5.2.3—Longitudinal Direction

Under earthquake loading, the earth-pressure action on abutment walls changes from a static condition to one of generally two possible conditions:

- The dynamic active pressure condition as the wall moves away from the backfill, or
- The passive pressure condition as the inertial load of the bridge pushes the wall into the backfill.

The governing earth-pressure condition depends on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge/abutment configuration.

C5.2.3

The seismic active pressure can be estimated using the Mononobe–Okabe method given in Appendix A11 of the *AASHTO LRFD Bridge Design Specifications.* The horizontal acceleration coefficient in the Mononobe–Okabe equation is the acceleration coefficient, A_s. If the abutment wall can displace several inches without damaging the bridge structure, the value of A_s can be reduced by 50 percent, as discussed in Article 11.6.5 of the *AASHTO LRFD Bridge Design Specifications*. Otherwise, the A_s should be used without reduction.

For relatively short abutment walls (e.g., effective height less than 10 ft), the passive earth pressure for seismic response analyses can be adequately evaluated using the static passive pressure methods given in Section 3

For seat-type abutments in which the expansion joint is sufficiently large to accommodate both the cyclic movement between the abutment wall and the bridge superstructure (i.e., superstructure does not push against abutment wall), the seismically induced earth pressure on the abutment wall shall be considered to be the dynamic active pressure condition. However, when the gap at the expansion joint is not sufficient to accommodate the cyclic wall/bridge movements, a transfer of forces will occur from the superstructure to the abutment wall. As a result, the active earth-pressure condition will not be valid and the earth pressure approaches a much larger passive pressure load condition behind the backwall, which is the main cause for abutment damage, as witnessed in past earthquakes. For stub or integral abutments, the abutment stiffness and capacity under passive pressure loading are primary design concerns.

of the *AASHTO LRFD Bridge Design Specifications*, as long as the passive pressure zone is compacted granular backfill. If backfill material includes cohesive soil content, the effects of cohesion should be accounted for in the passive earth pressure calculation.

Where abutment walls are greater than 10 ft in height or for critical abutment walls, the inertial response of the resisting soil wedge should be included in the passive pressure computation. Methods developed by Shamsabadi et al. (2007) and documented in the Caltrans computer program CT-Flex can be used in accounting for this effect. The Shamsabadi approach uses the log spiral method similar to static methods given in Section 3 of the *AASHTO LRFD Bridge Design Specifications*, and accounts for both soil friction ϕ and cohesion c in the backfill, interface friction between the wall and soil, and inertial effects within the failure wedge.

The Mononobe–Okabe equation for passive earth pressure given in Section 11 of the *AASHTO LRFD Bridge Design Specifications* should not be used for evaluation of the seismic passive capacity, as the assumptions associated with this equation lead to unconservative capacity estimates.

5.2.3.1—Abutment Longitudinal Response for SDCs B and C

Backwall reinforcement of seat-type abutments or the diaphragm of integral abutments designed primarily for nonseismic load conditions shall be checked for the seismic load path and altered if deemed appropriate.

The provisions of Article 5.2.3.2 may be used for the design of abutments for bridges in SDC B or C.

C5.2.3.1

Abutments designed for bridges in SDC B or C are expected to resist earthquake loads with minimal damage. For seat-type abutments, minimal abutment movement could be expected under dynamic passive pressure conditions. However, bridge superstructure displacement demands may be 4 in. or more and could potentially increase the soil mobilization.

5.2.3.2—Abutment Longitudinal Response for SDC D

For SDC D, passive pressure resistance in soils behind integral abutment walls and backwalls for seat abutments will usually be mobilized because of the large longitudinal superstructure displacements associated with the inertial loads. Two alternatives may be considered by the Designer:

- *Case 1: Earthquake-Resisting System (ERS) without Abutment Contribution.* The bridge ERS shall be designed to resist all seismic loads without any contribution from abutments. Abutments may contribute to limiting displacement, providing additional capacity and better performance that is not directly accounted for in the analytical model. To ensure that the columns will be able to resist the lateral loads, zero stiffness and capacity at the abutments should be assumed. In this case, an evaluation of the abutment that considers the implications of significant displacements from seismic accelerations shall be considered. As appropriate, this evaluation should include overturning for abutments.

- *Case 2: Earthquake-Resisting System (ERS) with Abutment Contribution.* In this case, the bridge shall be designed with the abutments as a key element of the ERS. Abutments are designed and analyzed to sustain the design earthquake displacements. When abutment stiffness and capacity are included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally used for static service load design, as illustrated schematically in Figure 1 (the approach slab shown is for illustration purposes only). Whether presumptive or computed passive pressures are used for design as stated in Article 5.2.3.3, backfill in this zone should be controlled by specifications, unless the passive pressure considered is less than 70 percent of the presumptive value.

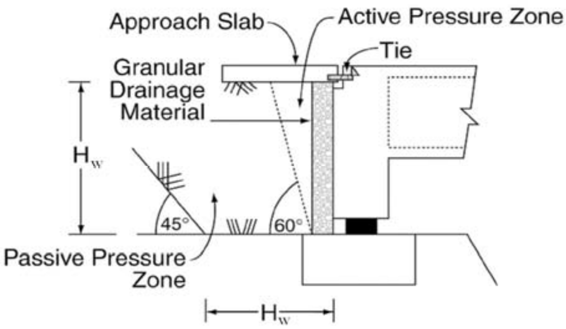


Figure 5.2.3.2-1—Design Passive Pressure Zone

5.2.3.3—Abutment Stiffness and Passive Pressure Estimate

C5.2.3.3

Abutment stiffness, K_{eff} in kip/ft, and passive capacity, P_p in kips, should be characterized by a bilinear or other higher order nonlinear relationship as shown in Figure 1. Passive pressures may be assumed uniformly distributed over the height (H_w) of the backwall or diaphragm. The total passive force may be determined as:

$$P_p = p_p H_w W_w \tag{5.2.3.3-1}$$

where:

p_p = passive lateral earth pressure behind backwall (ksf)

H_w = height of backwall (ft)

W_w = width of backwall (ft)

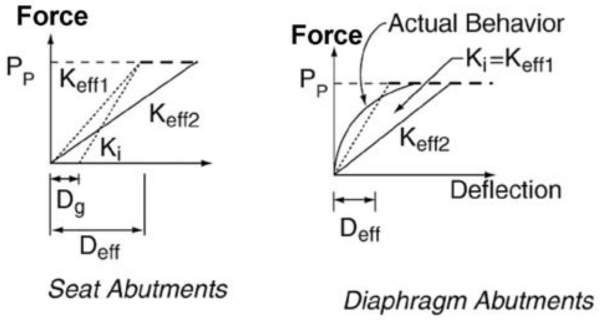


Figure 5.2.3.3-1—Characterization of Abutment Capacity and Stiffness

5.2.3.3.1—Calculation of Best Estimate Passive Pressure p_p

If the strength characteristics of compacted or natural soils in the "passive pressure zone" (total stress strength parameters c and ϕ) are known, then the passive force for a given height, H_w, may be computed using accepted analysis procedures. These procedures should account for the interface friction between the wall and the soil. The properties used shall be those indicative of the entire "passive pressure zone" as indicated in Figure 5.2.3.2-1. Therefore, the properties of backfill that is only placed adjacent to the wall in the active pressure zone may not be appropriate as a weaker failure surface can develop in the embankment.

If presumptive passive pressures are to be used for design, then the following criteria shall apply:

- Soil in the "passive pressure zone" should be compacted to a dry density greater than 95 percent of the maximum per ASTM D 1557 or equivalent.
- For cohesionless, nonplastic backfill (fines content less than 30 percent), the passive pressure p_p may be assumed equal to $2H_w/3$ ksf per ft of wall length.
- For cohesive backfill (clay fraction >15 percent), the passive pressure p_p may be assumed equal to 5 ksf, provided the estimated undrained shear strength is greater than 4 ksf.

The presumptive values given above shall be considered applicable for use in the "permissible earthquake-resisting elements that require Owner's approval," as defined in Article 3.3. If the design is based on presumptive resistances that are not greater than 70 percent of the values listed above, then the structure may be classified in the "permissible earthquake-resisting elements."

5.2.3.3.2—Calculation of Soil Stiffness

An equivalent linear secant stiffness, K_{eff} in kip/ft, is required for analyses. For integral- or diaphragm-type abutments, an initial secant stiffness (Figure 5.2.3.3-1) may be determined as follows:

$$K_{eff1} = \frac{P_p}{(F_w H_w)} \qquad (5.2.3.3.2\text{-}1)$$

where:

P_p = passive lateral earth pressure capacity (kip)

H_w = height of backwall (ft)

F_w = factor taken as between 0.01 to 0.05 for soils ranging from dense sand to compacted clays

If computed abutment forces exceed the soil capacity, the stiffness should be softened iteratively (K_{eff1} to K_{eff2}) until abutment displacements are consistent (within 30 percent) with the assumed stiffness. For seat-type abutments, the expansion gap should be included in the initial estimate of the secant stiffness. As specified in Eq. 2:

$$K_{eff1} = \frac{P_p}{(F_w H_w + D_g)} \qquad (5.2.3.3.2\text{-}2)$$

where:

D_g = width of gap between backwall and superstructure (ft)

For SDC D, where pushover analyses are conducted, values of P_p and the initial estimate of K_{eff1} should be used to define a bilinear load-displacement behavior of the abutment for the capacity assessment.

C5.2.3.3.2

Guidance on the value of F_w to use for a particular bridge may be found in Table C3.11.1-1 of the *AASHTO LRFD Bridge Design Specifications*. The table presents values of F_w for dense sand, medium dense sand, loose sand, compacted silt, compacted lean clay, and compacted fat clay. If the influence of passive pressure extends beyond one particular soil type at an abutment, averaged or weighted average values for F_w may be used at the Engineer's discretion.

5.2.4—Transverse Stiffness

Two alternatives may be considered by the Designer:

Case 1: Earthquake-Resisting System (ERS) without Abutment Contribution. The bridge ERS shall be designed to resist all seismic loads without any contribution from abutments. Concrete shear keys shall be considered sacrificial where they are designed for lateral loads lower than the design earthquake loads. A minimum level of design for shear keys corresponds to lateral loads not including earthquake loads. If sacrificial concrete shear keys are used to protect the piles, the bridge shall be analyzed and designed according to Articles 5.2.4.1 and 5.2.4.2 as applicable. If a fuse is used, then the effects of internal force redistribution resulting from fusing shall be taken into account in the design of the bridge. Limitations on the use of fusing (hinging or failure of a bridge component along the earthquake load path) for SDC C or D are listed below.

C5.2.4

For both Cases 1 and 2, abutment pile foundations may be considered adequate to carry the vertical dead loads following the design event for the no collapse criteria.

- *Case 2: Earthquake-Resisting System (ERS) with Abutment Contribution.* The bridge shall be designed with the abutments as a key element of the ERS. Shear keys at the abutment shall be designed and analyzed to sustain the lesser of the design earthquake forces or sliding friction forces of spread footings. Pile-supported foundations shall be designed to sustain the design earthquake displacements. Inelastic behavior of piles at the abutment shall be considered to be acceptable.

In the context of these provisions, elastic resistance shall be taken to include the use of:

- Elastomeric bearings,
- Sliding, or isolation bearings designed to accommodate the design displacements,
- Soil frictional resistance acting against the base and backwall of a spread footing-supported abutment, or
- Pile resistance provided by piles acting in their elastic range, or passive resistance of soil acting at displacements less that 2 percent of the wall height.

Likewise, fusing includes:

- Breakaway elements, such as isolation bearings with a relatively high yield force;
- Shear keys;
- Yielding elements, such as wingwalls yielding at their junction with the abutment backwall;
- Elastomeric bearings whose connections have failed and upon which the superstructure is sliding;
- Spread footings that are proportioned to slide; or
- Piles that develop a complete plastic mechanism.

The stiffness of the abutment foundation under transverse loading may be calculated on the basis of the procedures given in Article 5.3. Where fusing elements are used, allowance shall be made for the reduced equivalent stiffness of the abutment after fusing occurs.

5.2.4.1—Abutment Transverse Response for SDCs B and C

Shear keys shall be designed to resist a horizontal seismic force not less than the acceleration coefficient, A_s, as specified in Article 3.4, times the tributary permanent load.

Fusing is not expected for SDC B or C; however, if deemed necessary, fusing shall be checked using the procedure applicable to SDC D according to Article 5.2.4.2, taking into account the overstrength effects of shear keys according to Article 4.14.

C5.2.4.1

For bridges in these categories, elastic resistance may be achievable.

5.2.4.2—Abutment Transverse Response for SDC D

For structures in this category, either elastic resistance or fusing shall be used to accommodate transverse abutment loading. The elastic forces used for transverse abutment design shall be determined from an elastic demand analysis of the structure.

Where a shear key fusing mechanism is used for pile-supported abutments, the combined overstrength capacity of the shear keys shall be less than the combined plastic shear capacity of the piles. Soil friction and passive earth pressure shall not be included in the transverse abutment resistance of pile-supported abutments.

For concrete shear keys that are not intended to fuse, the design should consider the unequal forces that may develop in each shear key.

Recessed or hidden shear keys are difficult to inspect and repair and should be avoided if possible.

For pile-supported abutment foundations, the stiffness contribution of piles less than or equal to 18 in. in diameter or width shall be ignored if the abutment displacement is greater than 4 in. unless a displacement capacity analysis of the piles is performed and the piles are shown to be capable of accommodating the demands.

C5.2.4.2

Shear keys are typically classified as either:

- Interior—multiple shear keys placed between adjacent girders such that each individual shear key accommodates some portion of the total load, or
- Exterior—one shear key on each exterior face of the superstructure such that only one shear key is used to accommodate the total load in each direction.

Shear keys are used at abutments to provide transverse restraint for bridge superstructures under seismic and nonseismic loads. For seismic design, sacrificial shear keys can serve as structural fuses to limit the demand and thereby control the damage in the abutments and supporting piles.

Variations in shear key stiffness, shear key-to-superstructure gap size, and skew angle result in the unequal loading of interior shear keys. Unequal loading is further complicated for structures with intermediate piers of unequal stiffness. Consequently, the accurate determination of individual shear key design forces is difficult.

Because of the complications of accurately determining the design forces for interior shear keys, the use of exterior shear keys is recommended. Furthermore, exterior shear keys are usually recommended for new construction because they are easier to inspect and repair.

For interior shear keys that are intended to resist the elastic seismic forces, unequal loading should be considered in the design. For sacrificial shear keys, unequal loading may be of less concern because the shear keys are expected to fuse prior to the formation of plastic response in the piles. Thus, the unequal loading of interior shear keys may result in an “unzipping” response that would effectively reduce the abutment demand. Nonetheless, the combined overstrength capacity of all interior shear keys should be used when designing the abutment.

5.3—FOUNDATIONS

5.3.1—General

The foundation modeling methods (FMMs) defined in Table 1 should be used as appropriate. The requirements for estimating foundation springs for spread footings, pile foundations, and the depth to fixity for drilled shafts shall be as specified in Articles 5.3.2, 5.3.3, and 5.3.4, respectively. For a foundation that is considered as rigid, the mass of the foundation should be ignored in the analytical model. The Designer shall assess the merits of including the foundation mass in the analytical model where appropriate, taking into account the recommendations in this Article.

The required FMM depends on the SDC:

- FMM I is permitted for SDCs B and C provided the foundation is located in Site Class A, B, C, or D. Otherwise, FMM II is required.
- FMM II is required for SDC D.

The foundation models in the multimode dynamic analysis and displacement capacity verification shall be consistent and representative of the footing behavior. FMM II is required in the displacement capacity verification ("pushover") analysis if it is used in the multimode dynamic analysis for displacement demand.

For sites identified as susceptible to liquefaction or lateral spreading, the ERS global model shall consider the nonliquefied and liquefied conditions using the procedures specified in Article 6.8.

C5.3.1

A wide range of methods for modeling foundations for seismic analysis is available. 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 fixity 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 FMM II depends on the contribution of the foundation to the elastic displacement of the pier. More flexible spread and pile footings should be modeled and included in the seismic analysis.

If foundation springs are included in the multimode dynamic analysis, they should be included in the pushover analysis so the two models are consistent for the displacement comparison.

For most spread footings and piles with pile caps, a secant stiffness for the soil springs is adequate. Bilinear soil springs are used for the pushover analysis.

For pile bents and drilled shafts, an estimated depth to fixity 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 on *P-y* curves for the footing and soil. Bilinear springs may be used in the pushover analysis if there is particular concern with depth of the plastic hinge and effective depth of fixity. Equations are provided in the *AASHTO LRFD Bridge Design Specifications* (e.g. Article 10.7.3.13.4) for an equivalent depth to fixity for lateral stiffness. The equations are based upon linear–elastic models and may not be appropriate for large deflections without adjustment of the soil parameters. Other approaches exist, such as those presented in Buckle et al. (1987), ATC-32 (1996), Chai (2002), and Priestley (2007).

Table 5.3.1-1—Definition of Foundation Modeling Methods (FMMs)

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

5.3.2—Spread Footing

Where it is necessary to represent foundation flexibility, spring constants shall be developed for the modeling of spread footings. The spring constants should account for the likely modes of seismic response (i.e., translation and rotation), the embedment and shape of the footing, the stiffness of the soil beneath the footing, and the effects of seismic loading on the stiffness of the soil.

No special computations are required to determine the geometric or radiation damping of the foundation system. Five percent system damping shall be used for design, unless special studies are performed and approved by the Owner.

Spread footings shall not be used at locations where liquefaction or significant strength loss could occur during earthquake loading, unless the footing is located below the maximum depth of liquefaction or strength loss, or the ground has been improved such that liquefaction or strength loss will not occur.

C5.3.2

Procedures given in the FEMA 273 *Guidelines for the Seismic Rehabilitation of Buildings* (ATC/BSSC, 1997) are acceptable for estimating spring constants. Alternate methods of determining response include use of Winkler beam models or more rigorous numerical modeling.

The approach given in FEMA 273 includes equations for computing spring constants based on the shear modulus (G) of the soil. This modulus is determined by the average soil stiffness within one foundation width of the footing base. The uncertainty in determination of shear modulus should be considered when evaluating response. Normally, the evaluation of uncertainty should involve performing analyses for the best-estimated modulus and for a range of ±25 percent above and below the best estimate.

The shear modulus (G) used to compute stiffness values is normally determined by adjusting the low-strain shear modulus (G_{max}) for the level of shearing strain using strain-adjustment factors (G/G_{max}), which are less than 1.0. Strain-adjustment factors for SDC D should be less than those for SDC B or C. FEMA 273 provides guidance on the strain-adjustment factors based on the effective peak acceleration. The effective peak acceleration in this context is the same as A_s (acceleration coefficient at the ground surface) defined in Section 3 of these Guide Specifications. Alternatively, the strain-compatible shear modulus can be obtained as part of the site-specific ground motion response evaluation described in Article 3.4.3.1.

Values of G_{max} can be determined by seismic field methods (e.g., crosshole, downhole, or SASW), by laboratory testing methods (e.g., resonant column with adjustments for time), or by empirical equations. Kramer (1996) provides a discussion of these methods. Section 10 of the *AASHTO LRFD Bridge Design Specifications* identifies procedures for establishing site condition.

Moment-rotation and shear force-displacement can be represented by either an equivalent secant modulus or a bilinear relationship, when modeling footing response. The rotational spring constant of the spread footing can be taken as the rotational stiffness based on G_{max} if a bilinear moment-rotation curve is used. The maximum resisting force (i.e., plastic capacity) on the force-deformation curve is defined for the best-estimate case of geotechnical properties.

Uplift or rocking analysis for spread footings may be considered with the Owner's approval. See Appendix A.

5.3.3—Pile Foundations

The design of pile foundations shall be based on column loads determined by capacity design principles (Article 4.11) or elastic seismic forces, whichever is smaller for SDC B and based on capacity design principles only for SDC C or D. Both the structural and geotechnical elements of the foundation shall be designed accordingly.

Foundation flexibility shall be incorporated into design for SDC D according to Article 5.3.1.

The nonlinear properties of the piles shall be considered in evaluating the lateral response of the piles to lateral loads during a seismic event.

Liquefaction shall be considered using procedures specified in Article 6.8 where applicable during the development of spring constants and capacity values.

C5.3.3

A group reduction factor should be considered in the analysis of pile-group response to seismic loading. Section 10 of the *AASHTO LRFD Bridge Design Specifications* provides guidance on the determination of group reduction factors for horizontal loading. The group reduction factors in Section 10 were determined from results of a limited number of field quasi-static-load tests, model studies, and numerical analyses. There is uncertainty in the applicability of these group reduction factors during seismic loading. In light of this uncertainty, analyzing the structure with and without consideration of a group reduction factor should also be considered because the overall response of the structure for these two cases may vary significantly.

5.3.4—Drilled Shafts

The flexibility of the drilled shaft shall be represented using either the estimated depth of fixity or soil springs in a lateral pile analysis. Procedures identified in Article 5.3.3, including those for liquefaction, may be used except that group reduction factors are typically considered only in the transverse direction of a multishaft bent.

C5.3.4

Section properties of drilled shafts should be consistent with the deformation caused by the seismic loading. In many cases, it is necessary to use the cracked section modulus in the evaluation of lateral load-displacement relationships. In the absence of detailed information regarding reinforcing steel and applied load, an equivalent cracked section can be estimated by reducing the stiffness of the uncracked section by half. In general, the cracked section is a function of the reinforcement ratio and axial load, but it is often adequate to assume as one-half of the uncracked section.

5.4—ANALYTICAL PROCEDURES

5.4.1—General

The objective of seismic analysis is to assess displacement demands of a bridge and its individual components. Equivalent static analysis and linear elastic dynamic analysis are the appropriate analytical tools for estimating the displacement demands for normal bridges. Inelastic static "pushover analysis" is the appropriate analytical tool used to establish the displacement capacities for normal bridges assigned to SDC D.

Nonlinear time history analysis should be used for critical or essential bridges as defined in Article 4.2.2 and in some cases for normal bridges in SDC D using devices for isolation or energy dissipation. In this type of analysis, component capacities are characterized in the mathematical model used for the seismic response analysis. The procedures mentioned above are described in more detail in Article 5.4.4.

C5.4.1

For bridges with a regular configuration, a single-degree-of-freedom model is sufficiently accurate to represent the seismic response. For these types of bridges, the equivalent static analysis (Procedure 1) may be used to establish displacement demands.

For structures that do not satisfy the requirements of regularity for an elastic response spectrum analysis, Procedure 2 should be used to determine the displacement demands.

5.4.2—Procedure 1: Equivalent Static Analysis (ESA)

ESA may be used to estimate displacement demands for structures where a more sophisticated dynamic analysis will not provide additional insight into behavior. ESA should be considered to be best suited for structures or individual frames with well-balanced spans and uniformly distributed stiffness where the response can be captured by a predominant translational mode of vibration.

Both the uniform load method and the single-mode spectral analysis method shall be considered acceptable equivalent static analysis procedures.

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 single-mode spectral analysis method shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction. 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 procedure for using the single-mode spectral analysis method found in *AASHTO LRFD Bridge Design Specifications*, Article 4.7.4.3.2b may be used.

C5.4.2

The equivalent static analysis is suitable for short to medium span structures with regular configuration. Long bridges, or those with significant skew or horizontal curvature, have dynamic characteristics that should be assessed in a multimode dynamic analysis.

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 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. The single-mode spectral analysis method or a multimode dynamic analysis is recommended to avoid unrealistic distributions of seismic forces.

The steps in the uniform load method are as follows:

1. Calculate the static displacements $v_s(x)$ due to an assumed uniform load p_o, as shown in Figure C1. 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:

$$K = \frac{p_o L}{v_{s,max}} \qquad \text{(C5.4.2-1)}$$

$$W = \int_0^L w(x)\,dx \qquad \text{(C5.4.2-2)}$$

where:

p_o = uniform lateral load applied over the length of the structure (kip/ft or kip/in.)

L = total length of the structure (ft or in.)

$v_{s,max}$ = maximum lateral displacement due to uniform load p_o (ft or in.)

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

W = total weight of structure (kip)

K = effective lateral bridge stiffness (kip/ft or kip/in.)

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.

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

$$T_m = 2\pi\sqrt{\frac{W}{Kg}} \quad \text{(C5.4.2-3)}$$

where:

g = acceleration due to gravity (ft/s^2 or in./s^2)

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

$$p_e = \frac{S_a W}{L} \quad \text{(C5.4.2-4)}$$

where:

S_a = design response spectral acceleration coefficient determined in accordance with Article 3.4.1 for $T = T_m$

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

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.

The configuration requirements for ESA (Procedure 1) restrict application to individual frames or units that can be reasonably assumed to respond as a single-degree-of-freedom system in the transverse and longitudinal directions. When abutments do not resist significant seismic forces, the superstructure will respond as a rigid-body mass. The lateral load-resisting piers or bents should be uniform in strength and stiffness to justify the assumption of independent transitional response in the longitudinal and transverse directions.

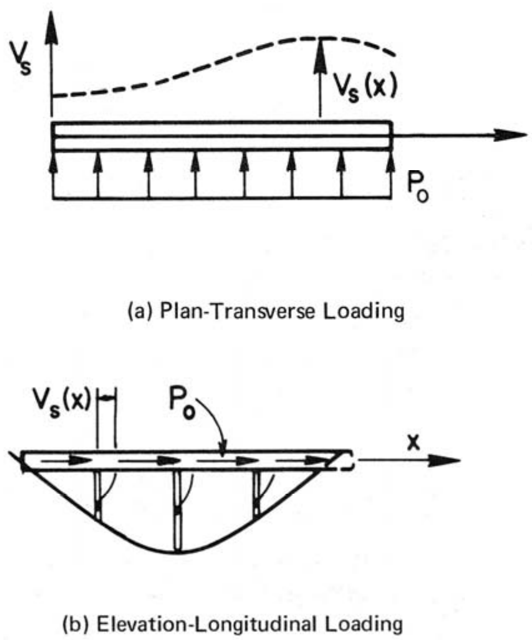


Figure C5.4.2-1—Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading

5.4.3—Procedure 2: Elastic Dynamic Analysis (EDA)

EDA shall be used to estimate the displacement demands for structures where ESA does not provide an adequate level of sophistication to estimate the dynamic behavior. A linear elastic multimodal spectral analysis using the appropriate response spectrum (i.e., 5 percent damping) shall be performed. The number of degrees of freedom and the number of modes considered in the analysis shall be sufficient to capture at least 90 percent mass participation in both the longitudinal and transverse directions. A minimum of three elements per flexible column and four elements per span shall be used in the linear elastic model.

The Engineer should recognize that forces generated by linear elastic analysis could vary, depending on the degree of nonlinear behavior, from the actual force demands on the structure. Displacements are not as sensitive to the nonlinearities and may be considered good approximations. Sources of nonlinear response that are not captured by EDA include the effects of the surrounding soil, yielding of structural components, opening and closing of expansion joints, and nonlinear restrainer and abutment behavior. EDA modal results shall be combined using the complete quadratic combination (CQC) method.

C5.4.3

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 displacement demands. The model should be based on cracked section properties for concrete components and on secant stiffness coefficients for the foundations, abutments, and seismic isolation components. Modeling should be consistent with the expected levels of deformation of all components. 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.

For multiframe analysis, it is recommended to include a minimum of two boundary frames or one frame and an abutment beyond the frame under consideration. (See Article 5.1.2.)

For SDC D, 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 nonductile components should be designed using capacity design principles to avoid brittle failure. For simple piers or bents, the displacement capacity can be evaluated by simple calculations using the geometry of displaced shapes and forces and deformations at the plastic hinges. 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 to assess the behavior beyond the displacement capacity and obtain a better understanding of the limit states.

The displacement capacity is compared to the displacement demand determined from an elastic response spectrum analysis.

Vibration modes are convenient representations of dynamic response for response spectrum analysis. Enough modes should be included to provide sufficient participation for bending moments in columns or other components with inelastic deformation. Dynamic analysis programs, however, usually compute participation factors only for base shear, often expressed as a percentage of total mass. For regular bridges, the guideline of including 90 percent of the modal mass for horizontal components generally provides a sufficient number of modes for accurate estimation 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 percent damping.

5.4.4—Procedure 3: Nonlinear Time History Method

The time histories of input acceleration used to describe the earthquake loads shall be selected following methods given in Article 3.4.4 of these Guide Specifications in consultation with the Owner. Time history analysis shall be performed with no fewer than three data sets (two horizontal components and one vertical component) of appropriate ground motion time histories selected from not less than three recorded or synthetic events. Appropriate time histories shall represent magnitude, fault distances, and source mechanisms that are consistent with those that control the design earthquake ground motion. Each time history shall be modified to be response spectrum compatible using the time domain procedure.

C5.4.4

A nonlinear dynamic analysis is a more comprehensive 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 the dynamic response analysis. A nonlinear dynamic response analysis requires a suite of time histories (Article 3.4.4) of earthquake ground motion that is 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 SDC D or in a case in which seismic isolation is included in the design strategy.

Where three time history data sets are used in the analysis of a structure, the maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used to determine design acceptability. Where seven or more time history data sets are employed, the average value of each response parameter may be used to determine design acceptability.

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 properties.

Seismically isolated structures with 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 should represent the hysteretic relationships for the isolator units.

5.5—MATHEMATICAL MODELING USING EDA (PROCEDURE 2)

5.5.1—General

The bridge should be modeled as a three-dimensional space frame with joints and nodes selected to realistically model the stiffness and inertia effects of the structure.

The mass 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.

C5.5.1

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 a cracked stiffness for reinforced concrete elements. However, the stiffness of seismic isolator units, abutments, and foundation soils are represented by a secant stiffness consistent with the maximum deformation. The Designer should consider the displacements from an elastic analysis to verify that they are consistent with the inelastic behavior of the earthquake-resisting elements.

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 expected strengths of the components gives larger plastic deformations than an analysis including overstrength. Hence, it is appropriate to use the expected strength of the components when estimating the displacement capacity.

5.5.2—Superstructure

The superstructure shall, as a minimum, be modeled as a series of space frame members with nodes at such points as the span quarter points in addition to joints at the ends of each span. Discontinuities should be included in the superstructure at the expansion joints and abutments. Care should be taken to distribute properly the lumped mass inertia effects at these locations. The effect of earthquake restrainers at expansion joints may be approximated by superimposing one or more linearly elastic members having the stiffness properties of the engaged restrainer units.

C5.5.2

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 should represent reasonable assumptions about the three-dimensional flow of forces in the superstructure, including composite behavior.

5.5.3—Substructure

The intermediate columns or piers should also be modeled as space frame members. Long, flexible columns should be modeled with intermediate nodes at the third points in addition to the joints at the ends of the columns. The model should consider the eccentricity of the columns with respect to the superstructure. Foundation conditions at the base of the columns and at the abutments may be modeled using equivalent linear spring coefficients.

5.6—EFFECTIVE SECTION PROPERTIES

5.6.1—Effective Reinforced Concrete Section Properties for Seismic Analysis

Because elastic analysis assumes a linear relationship between stiffness and strength, analysis of concrete members shall consider that they display nonlinear response before reaching their idealized yield limit state.

Section properties, flexural stiffness, E_cI_{eff}, shear stiffness parameter $(GA)_{eff}$, and torsional stiffness G_cJ_{eff}, shall reflect the cracking that occurs before the yield limit state is reached. The effective moments of inertia, I_{eff} and J_{eff}, shall be used to obtain realistic values for the structure's period and the seismic demands generated from ESA and EDA analyses.

5.6.2—E_cI_{eff} and $(GA)_{eff}$ for Ductile Reinforced Concrete Members

The effective moment of inertia I_{eff} should be used when modeling ductile elements. I_{eff} may be estimated by Figure 1, or the slope of the M-ϕ curve between the origin and the point designating the first reinforcing bar yield shall be taken as:

$$E_cI_{eff} = \frac{M_y}{\phi_y} \qquad (5.6.2\text{-}1)$$

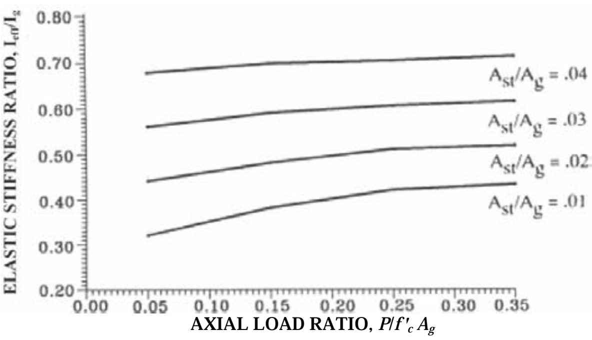
where:

M_y = moment capacity of section at first yield of the reinforcing steel (kip-in.)

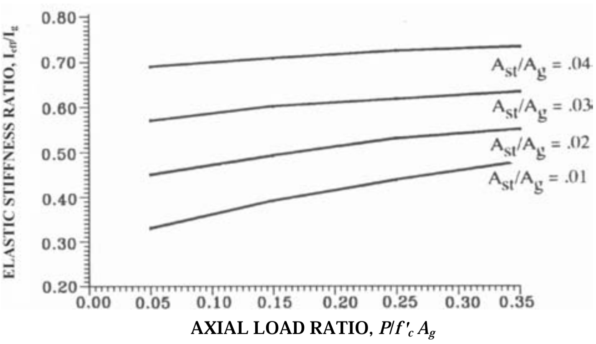
ϕ_y = curvature of section at first yield of the reinforcing steel including the effects of the unfactored axial dead load (1/in.)

E_c = modulus of elasticity of concrete (ksi)

I_{eff} = effective moment of inertia of the section based on cracked concrete and first yield of the reinforcing steel (in.^{4})



(a) Circular Sections



(b) Rectangular Sections

Figure 5.6.2-1—Effective Flexural Stiffness of Cracked Reinforced Concrete Sections

Typically the unfactored axial gravity load shall be used when determining the effective properties.

The M-ϕ analysis parameters shall be taken as specified in Articles 8.4 and 8.5.

For pier walls in the strong direction, the shear stiffness parameter $(GA)_{eff}$ may be determined as follows:

$$(GA)_{eff} = G_c A_{ew} \frac{I_{eff}}{I_g} \quad (5.6.2\text{-}2)$$

where:

$(GA)_{eff}$ = effective shear stiffness parameter of the pier wall (kip)

G_c = shear modulus of concrete (ksi)

A_{ew} = cross-sectional area of pier wall (in.^{2})

I_g = gross moment of inertia taken about the weak axis of the reinforced concrete cross-section (in.^{4})

I_{eff} = effective moment of inertia taken about the weak axis of the reinforced concrete cross-section calculated from Eq. 1 or Figure 1 (in.^{4})

For prestressed concrete piling used in pile bents as the energy dissipation elements of the ERS, the effective stiffness ranges between $0.6I_g$ and $0.75I_g$, and conservative values from this range may be used in SDC B. For SDC C and D, moment-curvature analysis shall be used to determine the effective moments of inertia. For capacity protected elements in pile foundations, the stiffnesses shall be chosen with regard to the loading level expected.

5.6.3—I_{eff} for Box Girder Superstructures

The determination of I_{eff} for box girder superstructures should include consideration of the following:

- I_{eff} in box girder superstructures is dependent on the extent of cracking and the effect of the cracking on the element's stiffness.
- I_{eff} for reinforced concrete box girder sections may be estimated between $0.5I_g$ and $0.75I_g$. The lower bound represents lightly reinforced sections and the upper bound represents heavily reinforced sections.
- The location of the centroid of the prestressing steel and the direction of bending have a significant impact on how cracking affects the stiffness of prestressed members. Multimodal elastic analysis is incapable of capturing the variations in stiffness caused by moment reversal. Therefore, no stiffness reduction is recommended for prestressed concrete box girder sections.

5.6.4—I_{eff} for Other Superstructure Types

Reductions to I_g similar to those specified for box girders may be used for other superstructure types and cap beams. A more refined estimate of I_{eff} based on M-ϕ analysis may be warranted for lightly reinforced girders and precast elements.

5.6.5—Effective Torsional Moment of Inertia

The determination of the torsional stiffness should include consideration of the following:

- A reduction of the torsional moment of inertia is not required for bridge superstructures.

- The torsional stiffness of concrete members can be greatly reduced after the onset of cracking.
- The torsional moment of inertia for columns shall be reduced as follows:

$$J_{eff} = 0.2 J_g \qquad (5.6.5\text{-}1)$$

where:

J_{eff} = effective torsional (polar) moment of inertia of reinforced concrete section ($in.^4$)

J_g = gross torsional (polar) moment of inertia of reinforced concrete section ($in.^4$)

SECTION 6: FOUNDATION AND ABUTMENT DESIGN

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SECTION 6:

FOUNDATION AND ABUTMENT DESIGN

6.1—GENERAL

This Section includes only those foundation and abutment requirements that are specifically related to seismic-resistant construction. It assumes compliance with all the basic requirements necessary to provide support for vertical loads and lateral loads other than those due to earthquake motions. These include, but are not limited to, provisions for the extent of foundation investigation, fills, slope stability, bearing and lateral soil pressures, drainage, settlement control, and pile requirements and capacities.

6.2—FOUNDATION INVESTIGATION

6.2.1—Subsurface Investigation

Subsurface investigations, including borings and laboratory soil tests, shall be conducted to provide pertinent and sufficient information for the determination of the Site Class in Article 3.4.2.1. To provide the input and site characterization needed to complete all geotechnical aspects of the seismic design, subsurface exploration, laboratory tests, in-situ tests, and geophysical tests of the subsurface materials shall be conducted in accordance with Articles 10.4.2, 10.4.3, 10.4.4, and 10.4.5, respectively, of the *AASHTO LRFD Bridge Design Specifications*.

C6.2.1

The goal of the site characterization for seismic design is to develop the subsurface profile and soil property information required for seismic analyses. Soil parameters that may be required for seismic design include:

- Initial dynamic shear modulus at small strains or shear wave velocity,
- Equivalent viscous damping ratio,
- Shear modulus reduction and equivalent viscous damping characteristics as a function of shear strain,
- Cyclic shear strength parameters (peak and residual), and
- Liquefaction resistance parameters.

For Site Class determination, soil should be characterized to a depth of at least 100 ft using standard penetration tests (SPT), cone penetrometer tests (CPT), or shear wave velocity measurements, unless rock is encountered before 100 ft.

The groundwater elevation should be determined. If seasonal groundwater fluctuations occur, the seasonally averaged groundwater elevation should be determined. If seasonal groundwater fluctuations are unknown due to inadequacies in the data (e.g., the period of observation happened to be an unusually dry year), the maximum groundwater elevation as determined from groundwater measurements or observed geologic evidence should be used. See Article 6.8 for use of this data in liquefaction analyses.

6.2.2—Laboratory Testing

Laboratory tests shall be performed to determine the soil type, strength, deformation, and flow characteristics of soil and rock or both, and their suitability for the foundation selected. In areas of higher seismicity (e.g., SDC D), if questionable soils (e.g., nonplastic silts) exist or if the foundation is supporting an essential or critical bridge, it may be appropriate to conduct special dynamic or cyclic tests to establish the liquefaction potential or stiffness and material-damping properties of the soil at some sites.

C6.2.2

In certain soils or conditions, it may be appropriate to conduct cyclic laboratory tests to characterize either the cyclic strength or the stiffness and material damping characteristics of the soil.

- One common example of this requirement involves establishing the potential for post-liquefaction settlement of silts, in view of the limited database for this type of information.
- Cyclic tests may also be required to evaluate both liquefaction potential and post-liquefaction settlement of silts and sands obtained from depths greater than 75 ft. As discussed in Article 6.8, the validity of the simplified empirical method for estimating liquefaction potential is uncertain at depths greater than 75 ft and, therefore, soils from deeper than 75 ft may require special liquefaction testing if the combination of ground motions and soil density appear to result in liquefaction.
- Cyclic tests may also be required to evaluate the effects of earthquake loading on the strength of sensitive clay samples. This evaluation could include determining the reduction in strength by conducting post-cyclic undrained strength tests.

Cyclic testing should be conducted on high-quality undisturbed soil samples. The effects of soil fabric on dynamic soil properties make it very difficult to simulate in-situ soil properties by reconstituting samples to the field density.

For most seismic loading conditions, undrained soil conditions will occur during seismic loading because of the rapid rate of seismic loading and the inability for excess pore-water pressures developed during shear to dissipate quickly enough. These undrained conditions should be simulated during static or cyclic laboratory tests.

When estimating the strength of the soil for seismic loading studies using static triaxial testing methods, total stress strength parameters of cohesion and friction (i.e., c and ϕ) rather than effective stress strength parameters should be determined. The exception to this occurs where very clean granular soils exist or will be used as backfill.

The liquefaction assessment requires characterization of the grain-size distribution of soils in those layers that are potentially susceptible to liquefaction. This requirement is particularly important if the simplified empirical method (Article 6.8) is used to estimate liquefaction potential. When screening soils for liquefaction potential, it is also necessary to establish the plasticity index (*PI*) of the soil, particularly in silty soils.

6.2.3—Foundation Investigation for SDC A

There are no special seismic foundation investigation requirements for SDC A.

6.2.4—Foundation Investigation for SDCs B, C, and D

In addition to the normal site investigation, potential hazards and seismic design requirements related to (1) liquefaction potential, (2) seismic-induced settlement, (3) lateral spreading, (4) slope instability, and (5) increases in lateral earth pressure, all as a result of earthquake motions, should be evaluated. The seismic hazards evaluation shall also consider the potential for and influence of:

- Surface rupture due to faulting if an active fault has been identified within 1 mi of the bridge site (see Article 3.4 for the definition of an active fault),
- Differential ground displacement (lurching), and
- Cyclic loading on the deformation and strength characteristics of foundation soils.

Results of these evaluations shall be documented in a geotechnical report.

C6.2.4

A number of soil-related earthquake loading issues should be considered during design of bridge foundations for SDC B, C, and D sites, including:

- Fill settlement and abutment displacements due to the design earthquake loading that lead to bridge collapse, access problems, or structural damage,
- Seismic-induced excess pore-water pressures and liquefaction of saturated sands, nonplastic silts, and gravels used for fills or as foundation soils that can contribute to slope and abutment instability, and could lead to a loss of foundation bearing capacity and lateral pile support,
- Downdrag forces on pile foundations due to seismic-induced ground settlement,
- Soil-movement-induced lateral forces on the foundation due to lateral spreading or ground lurching, and
- Progressive degradation in the stiffness and strength characteristics of saturated cohesionless soil and soft cohesive soils.

More detailed analyses of these strength, stiffness, and displacement issues should be considered as part of the geotechnical evaluation of site stability during earthquake loading.

6.3—SPREAD FOOTINGS

6.3.1—General

Spread footings in SDC B shall be proportioned to resist overturning, sliding, flexure, and shear due to the lesser of the following:

- The forces obtained from an elastic linear seismic analysis, or
- The forces associated with the overstrength plastic moment capacity of the column or wall.

Spread footings in SDC C and SDC D shall be proportioned to resist overturning, sliding, flexure, and shear due to the forces associated with the overstrength plastic moment capacity of the column or wall.

C6.3.1

In lower seismic hazard areas, seismic demands may not govern the design forces acting on the substructure. These Guide Specifications do not require that the forces associated with the overstrength plastic moment capacity of the column or wall be used to proportion footings in SDC B. However, because the columns in SDC B are designed and detailed to accommodate a displacement ductility demand of 2.0, designing the footings to accommodate the forces associated with the overstrength plastic moment capacity may be warranted.

6.3.2—Modeling of Footings

Spread footing foundations shall be modeled according to Article 5.3.2.

Footings satisfying the requirements of Eq. 1 may be assumed to behave as rigid members. Footings that do not satisfy Eq. 1 require additional analysis and are not addressed in these Guide Specifications.

$$\frac{L - D_c}{2H_f} \leq 2.5 \tag{6.3.2-1}$$

where:

L = length of footing measured in the direction of loading (ft)

D_c = column diameter or depth in direction of loading (ft)

H_f = depth of footing (ft)

C6.3.2

The lateral, vertical, and rotational stiffness of spread footings should be included in the bridge model.

6.3.3—Spread Footings in Liquefiable Soils

Spread footings shall not be located in soils that are susceptible to liquefaction unless the footing is located below the maximum depth of liquefaction or the potential for liquefaction is mitigated by ground improvement. Methods identified in Article 6.8 shall be used to evaluate the liquefaction potential.

C6.3.3

Spread footings founded in liquefiable soils are susceptible to large, unpredictable displacements and have resulted in bridge failures.

Soil densification and other ground improvement methods can be used as a means of mitigating the potential for liquefaction. Once the ground has been improved, and verification studies confirm that the liquefaction potential has been mitigated, spread footings can be located above the improved ground at the site. See Article C6.8 for additional guidance on the use of ground improvement to mitigate liquefaction.

The decision between using ground improvement and spread footings or using deep foundations should be made on the basis of risk or uncertainties involved, economics, schedule, and other factors that must be considered in the type, size, and location of the bridge.

6.3.4—Resistance to Overturning

The overturning demand due to forces associated with the plastic overstrength moment of a column or wall shall be less than the overturning resistance of the footing, and the location of the resultant shall be limited as described below. Overturning shall be examined in each principal direction and satisfy the following requirement:

$$M_{po} + V_{po}H_f \leq \phi P_u \left(\frac{L - a}{2} \right) \tag{6.3.4-1}$$

in which:

$$a = \frac{P_u}{q_n B} \tag{6.3.4-2}$$

C6.3.4

Eq. 1 neglects the lateral soil resistance that may develop along the depth of the footing. The omission of passive soil resistance, as well as interface shear, along the depth of the footing is conservative and, in most cases, is insignificant. When the edge of the footing is cast against rock, Eq. 1 may be modified to incorporate the lateral rock resistance.

The bearing capacity for this evaluation is usually based on the static nominal resistance defined in Section 10 of the *AASHTO LRFD Bridge Design Specifications.* This approach is appropriate for sites where spread footing meet strength and service limit state requirements. As noted in Article 6.3.3, liquefiable soils are the exception. Where liquefiable soils exist, the requirements in Article 6.3.3 shall be satisfied.

where:

M_{po} = overstrength plastic moment capacity of the column calculated in accordance with Article 8.5 (kip-ft)

V_{po} = overstrength plastic shear demand (kip)

H_f = depth of footing (ft)P_u = axial force in column including the axial force associated with overstrength plastic hinging calculated in accordance with Article 4.11 (kip)

L = length of footing measured in the direction of loading (ft)

B = width of footing measured normal to the direction of loading (ft)

q_n = nominal bearing capacity of supporting soil or rock (ksf)

ϕ = resistance factor for overturning of footing taken as 1.0

Additionally, the location of the resultant of the reaction forces shall be located within the middle two-thirds of the base, if no live load is present. Otherwise, Article 6.3.9 is applicable. If full live load is present, then the resultant shall be within the middle eight-tenths of the base. If live load acts to reduce the eccentricity, then it shall not be included in the check of overturning.

6.3.5—Resistance to Sliding

The lateral demand due to the plastic overstrength shear of the column shall be less than the sliding resistance of the footing. Sliding shall be examined in each principal direction and satisfy the following requirement:

$$V_{po} \leq \phi R_n \qquad (6.3.5\text{-}1)$$

where:

V_{po} = overstrength plastic shear demand of the column or wall (kip)

ϕ = resistance factor for sliding of footing taken as 1.0

R_n = nominal sliding resistance against failure by sliding determined in accordance with Article 10.6.3.4 of the *AASHTO LRFD Bridge Design Specifications* (kip)

C6.3.5

Failure against sliding is addressed in Section 10 of the *AASHTO LRFD Bridge Design Specifications.*

6.3.6—Flexure

Flexural demands shall be investigated at the face of a column or wall for both positive and negative flexure and satisfy the following:

$$\phi M_n \geq M_u \quad (6.3.6\text{-}1)$$

where:

M_u = factored ultimate moment demand in footing at the face of the column or wall (kip-ft)

ϕ = resistance factor for concrete in bending

M_n = nominal moment capacity of the footing at the critical section, including the effects of reinforcing bars that are not fully developed at the critical section (kip-ft)

The effective width of the footing, b_{eff}, used to calculate the nominal moment capacity of the footing, M_n, shall be taken as:

$$b_{eff} = B_c + 2H_f \leq B \quad (6.3.6\text{-}2)$$

where:

B_c = diameter or width of column or wall measured normal to the direction of loading (ft)

H_f = depth of footing (ft)

B = width of footing measured normal to the direction of loading (ft)

C6.3.6

The factored ultimate moment demand, M_u, should be based on the actual soil pressure distribution resulting from the plastic overstrength moment of the column and the associated forces. The resulting soil pressure distribution may be linear or nonlinear depending on the magnitude of the demand as well as the nominal compressive resistance of the soil. In lieu of the actual soil pressure distribution under the footing, the moment associated with a fully plastic soil pressure distribution (i.e., nominal bearing capacity fully developed) may be conservatively assumed, in which case M_u would be determined as:

$$M_u = P_u\left(\frac{L-a-D_c}{2}\right) \quad (C6.3.6\text{-}1)$$

in which:

$$a = \frac{P_u}{q_n B} \quad (C6.3.6\text{-}2)$$

where:

P_u = factored ultimate axial force in column including the axial force associated with overstrength plastic hinging calculated in accordance with Article 4.11 (kip)

L = length of footing measured in the direction of loading (ft)

D_c = column diameter or depth in direction of loading (ft)

Shear lag effects in the footing render the reinforcement at the edges of the footing less effective in resisting flexural demands.

Caltrans Seismic Design Criteria permits the use of the full footing width, $b_{eff} = B$, when calculating the nominal moment and shear capacity of the footing, provided that the requirements of Article 6.3.2 and Article 6.3.8 are satisfied.

6.3.7—Shear

Shear demands shall be investigated at the face of the column or wall for both positive and negative bending and satisfy the following:

$$\phi_s V_n \geq V_u \quad (6.3.7\text{-}1)$$

where:

V_u = factored ultimate shear demand in footing at the face of the column or wall (kip)

ϕ_s = resistance factor for concrete in shear

C6.3.7

It is recommended that the minimum amount of shear reinforcement be provided in all footings that are subjected to the overstrength plastic moment capacity of the column. Shear reinforcement in footings is typically provided by “J” bars or headed bars.

V_n = nominal shear capacity of the footing at the face of the column calculated in accordance with Article 5.8 of the *AASHTO LRFD Bridge Design Specifications* (kip)

The effective width of the footing, b_{eff}, used to calculate the nominal shear capacity of the footing, V_n, shall be taken as that specified in Eq. 6.3.6-2.

6.3.8—Joint Shear

Joint shear shall satisfy the requirements of Article 6.4.5.

C6.3.8

Column–footing joints are required to be designed to transfer the overstrength column forces to the footing.

6.3.9—Foundation Rocking

If permitted by the Owner, foundation rocking, as specified in Appendix A, may be explicitly modeled to accommodate seismic demands.

Where rocking is allowed by the Owner as an earthquake-resistant element (ERE), the impacts on system behavior shall be evaluated. Global (i.e., full bridge or frame system) dynamic effects of rocking, whether by individual piers or more, shall be considered. Geotechnical capacities of the foundations, including assessment of potential settlement, shall be assessed to ensure that undesirable system deformations do not jeopardize the resistance or stability of the bridge system (earthquake-resistant system (ERS)).

C6.3.9

Foundation rocking may be used as an effective means of accommodating seismic demands in a manner similar to isolation bearings. However, rocking may result in large permanent soil movement, which may affect bridge performance during and following the seismic event. The amount of permanent movement may be sufficient to affect the ability to repair the bridge after the earthquake. For this reason, the decision to allow rocking beyond the limits in Eq. 6.3.4-1 must be made after careful consideration of the consequences of this approach.

In general, research is ongoing in the area of foundation rocking. At this time, the state of the practice does not warrant the use of foundation rocking for typical highway bridge structures.

6.4—PILE CAP FOUNDATION

6.4.1—General

The design of pile foundation for SDC B should be based on forces determined by capacity design principles or elastic seismic forces, whichever is smaller. The use of forces without the overstrength factor, as outlined in Article 4.11.1, is permitted.

The design of pile foundation for SDC C or D shall be based on forces determined by capacity design principles.

The lateral, vertical, and rotational capacity of the foundation shall exceed the respective demands. The size and number of piles and the pile group layout shall be designed to resist service level moments, shears, and axial loads and the force demands induced by the column plastic hinging mechanism. All forces acting on the pile cap shall be considered, including effects of pile head fixity and passive resistance.

C6.4.1

Pile cap foundations are often relatively complex elements to design for seismic effects. In general, all forces and compatible deformations need to be considered. However, under certain conditions pile cap design may be conservatively simplified, as outlined in Article 6.4.2.

To meet uplift loading requirements during a seismic event, the depth of penetration into the soil may have to be greater than minimum requirements for compressive loading to mobilize sufficient uplift resistance. This uplift requirement can impose difficult installation conditions at locations where very hard bearing layers occur close to the ground surface. Ground anchors, insert piles, and H-pile stingers may be considered in these locations to provide extra uplift resistance.

If batter piles are used, consideration should be given to (1) downdrag forces caused by settlement of the ground around the pile during dissipation of pore-water pressures developed during liquefaction, (2) potential for lateral displacement of the soil from liquefaction-induced flow or lateral spreading, (3) ductility at the connection of the pile to the pile cap, and (4) buckling of the pile under combined horizontal and vertical loading. As such, use of batter piles should be handled on a case-by-case basis. Close interaction between the geotechnical engineer and the structural engineer will be essential when modeling the response of the batter pile for seismic loading.

6.4.2—Moment Capacity of Pile Foundations

The provisions described below shall be taken to apply to columns with monolithic fixed connections to the footings designed for elastic forces as in SDC B or for column plastic hinge formation at the base as in SDC B, C, or D.

The design of pile foundations in competent soil may be simplified using elastic analysis. For piles with a diameter or width larger than 18 in., the distribution of forces to the piles and the pile cap may be influenced by the fixity of the pile connection to the pile cap in addition to the overall piles/pile cap flexibility. A more refined model that takes into account the pertinent parameters should be considered for establishing a more reliable force distribution.

A linear distribution of forces, shown in Figure 1, at different rows of piles, referred to as a simplified foundation model, may be considered adequate provided a rigid footing response may be assumed. The rigid response of a footing may be assumed provided if Eq. 1 is satisfied.

C6.4.2

Capacity Protection for the foundation design is not required for SDC B.

Foundations surrounded by competent soil are capable of resisting the design seismic forces while experiencing small deformations. Indicators of competent soils are SPT blow counts, $N > 20$ in the upper 10 ft, $N > 30$ between 10 ft and 30 ft for cohesionless soils, undrained shear strengths >1500 psf for cohesive soils, or shear wave velocities >600 ft/sec. Additionally, there must be low potential for liquefaction, flow, or spreading.

With competent soil and where certain criteria are met, a simplified model of the pile cap foundation is appropriate (Caltrans, 2006). The criteria are:

- Lateral displacements are small and passive resistance of the cap and upper 4 to 8 diameters of the piles can resist the applied shears,
- The cap is essentially rigid,
- The pile group's nominal resistance is limited to the capacity available when any individual pile reaches its nominal axial resistance,
- Pile spacing is greater than about 3 pile diameters, thus minimizing group effects, and
- Pile-to-cap connection is designed essentially as a pinned connection.

If these criteria are met, then pile head moments may be neglected and the pile forces may be calculated assuming a linear distribution of pile force (Caltrans, 2006).

For piles more than about 18 in. in diameter or width, the pile flexural capacity at the pile-to-cap connection can provide significant force transfer. Neglecting the moments from pile head fixity is not conservative; thus, for larger piles such moment transfer should be considered in design. Additionally, as piles become larger in cross-section, the pile cap may not behave rigidly, and such additional flexibility should be considered in the pile cap stiffness calculation and in the distribution of forces in the pile cap.

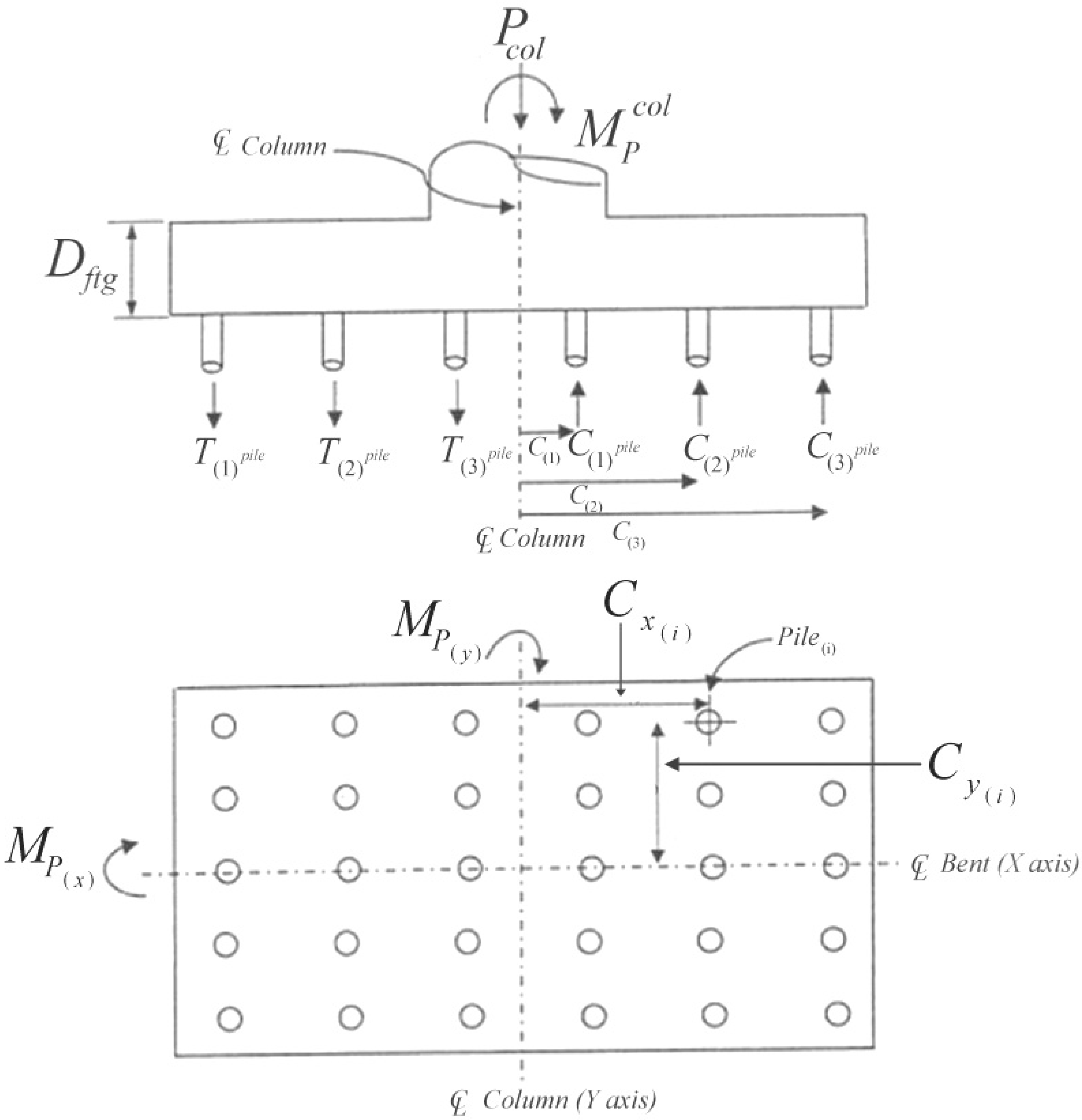

Figure 6.4.2-1—Simplified Model for Pile Foundations in Competent Soil

$$\frac{L_{ftg}}{D_{ftg}} \leq 2.5 \qquad (6.4.2\text{-}1)$$

where:

L_{ftg} = cantilever overhang length measured from the face of wall or column to the outside edge of the pile cap or footing (ft)

D_{ftg} = depth of the pile cap or footing (ft)

In accordance with capacity design principles, the distribution of forces on these piles shall be examined about the x and y axes, in addition to the diagonal direction of the foundation cap, considering that the principal axes of the column correspond to x and y axes. For cases in which the column principal axes do not correspond to pile and cap axes, the Designer shall investigate alternative orientations to ensure hinging in the column.

For SDCs C and D, the axial demand on an individual pile shall be taken as:

$$\left.\begin{matrix} C_{(i)}^{pile} \\ T_{(i)}^{pile} \end{matrix}\right\} = \frac{\sum P}{N_p} \pm \frac{M_{(y)}^{col} c_{x(i)}}{I_{pg(y)}} \pm \frac{M_{(x)}^{col} c_{y(i)}}{I_{pg(x)}} \qquad (6.4.2\text{-}2)$$

in which:

$$I_{pg(y)} = \sum_{i=1}^{N_y} n_x c_{x(i)}^2 \qquad (6.4.2\text{-}3)$$

$$I_{pg(x)} = \sum_{i=1}^{N_x} n_y c_{y(i)}^2 \qquad (6.4.2\text{-}4)$$

where:

$I_{pg(y)}$ = effective moment of inertia of pile group about the y axis (pile-ft^2)

$I_{pg(x)}$ = effective moment of inertia of pile group about the x axis (pile-ft^2)

$M_{(x)}^{col}$ = the component of the column plastic hinging moment capacity about the x axis (kip-ft)

$M_{(y)}^{col}$ = the component of the column plastic hinging moment capacity about the y axis (kip-ft)

N_p = total number of piles in the pile group (pile)

n_x = number of piles in a single row parallel to the y axis

n_y = number of piles in a single row parallel to the x axis

$\sum P$ = total unfactored axial load due to dead load, earthquake load, footing weight, soil overburden, and all other vertical demands acting on the pile group (kip)

$c_{x(i)}$ = distance from neutral axis of pile group to ith row of piles measured parallel to the y axis (ft)

$c_{y(i)}$ = distance from neutral axis of pile group to ith row of piles measured parallel to the x axis (ft)

The Designer should evaluate whether the design moment for the pile group is the plastic moment from the column or the plastic moment with overstrength. The conservative approach is to use the overstrength plastic moment. However, some inherent conservatism exists in the simplified model used in this Article due to the linear elastic approach. Caltrans (2006) uses the plastic moment without overstrength because they have determined that sufficient reserve capacity is present in the design to offset the use of the smaller moment for their typical designs. This practice should be evaluated on a case-by-case basis. Eq. 2 is non-specific with regard to applied column moments for this reason.

$C_{(i)}^{pile}$ = compression force in ith pile (kip)

$T_{(i)}^{pile}$ = tension force in ith pile (kip)

For SDC B, in cases in which elastic forces control, the axial demand on an individual pile shall be determined according to Eq. 2, with the elastic forces and moments according to Article 4.4 substituted for the plastic hinging forces and moments.

In soft soils, consideration shall be given to the possibility that the pile cap may not dominate the lateral stiffness of the foundation, as is expected in competent soil, possibly leading to significant lateral displacements as pile capacities are mobilized in lateral loading. In soft soils, piles shall be designed and detailed to accommodate displacements-induced head moments and axial forces based on analytical findings.

6.4.3—Lateral Capacity of Pile Foundations

The lateral capacity of pile foundations in soils shall be evaluated. The capacity evaluation shall include the resistance developed by the pile cap and the lateral shear resistance of the piles. The amount of displacement to mobilize the resistance from the cap and the piles shall be considered in the capacity estimate. The Designer shall verify that the geotechnical and structural capacity of the pile cap and the piles exceed the lateral demand transmitted by the columns.

C6.4.3

Lateral capacity of the pile cap should include the passive pressure mobilized at the face of the cap and the interface shear resistance developed along each side of the cap. Procedures used to estimate the passive pressure at the face of the cap can normally involve static passive pressure equations and charts given in Section 3 of the *AASHTO LRFD Bridge Design Specifications*. Wall friction of two-thirds of the friction angle should be used in this determination. The amount of displacement to mobilize the passive pressure should follow guidance given in Section 10 of the *AASHTO LRFD Bridge Design Specifications*.

The shear along the side of the cap can be estimated using the effective pressure at the mid-height of the cap thickness (σ_v'), a lateral stress factor (K_o) of 0.5, and the friction angle (ϕ) of the backfill material (i.e., $F_s = (\sigma_v' K_o \tan \phi) A_{surf}$ where A_{surf} is the surface area for each side of the cap. If a cohesive soil is used for backfill, the undrained strength of the cohesive soil is used in place of $\sigma_v' K_o \tan \phi$. The amount of displacement to mobilize the shear capacity along the side of the cap is usually less than 0.5 in. For many cases, the contributions of side shear are small and can be neglected in the capacity estimate.

Methods used to estimate the load-deformation response of piles are established in Section 10 of the *AASHTO LRFD Bridge Design Specifications* and can be used to develop a stiffness value for the pile group. If liquefaction is possible, appropriate adjustments should be made to evaluate stiffness for the liquefied case. This evaluation involves use of the residual strength of the liquefied soils. Because of uncertainties in the development of liquefaction, checks should also be performed for the nonliquefied case to determine the more critical of the two.

6.4.4—Other Pile Requirements

Piles may be used to resist both axial and lateral loads. The minimum depth of embedment, together with the axial and lateral pile capacities, required to resist seismic loads shall be determined by means of the design criteria established in the site investigation report. Group reduction factors established in the geotechnical report should be included in the analysis and design of piles required to resist lateral loads. The nominal geotechnical capacity of the piles should be used in designing for seismic loads.

Where reliable uplift pile capacity and the pile-to-footing connection and structural capacity of the pile are adequate, pile side resistance from uplift of a pile footing may be used in the capacity evaluation with the Owner's approval, provided that the magnitude of footing rotation will not result in unacceptable performance according to P-Δ requirements stated in Article 4.11.5. Additionally, the connection between the footing or cap and the piles should be capacity protected to resist the maximum force the pile could deliver.

All concrete piles shall be reinforced to resist the design moments, shears, and axial loads. Minimum reinforcement shall be in accordance with Article 8.16.

Footings shall be proportioned to provide the minimum embedment, clearance, and spacing requirements according to the provisions of the *AASHTO LRFD Bridge Design Specifications*. The spacing shall be increased when required by subsurface conditions. For SDC D, embedment of pile reinforcement in the footing cap shall be in accordance with Article 8.8.4.

C6.4.4

Friction piles may be considered to have uplift resistance due to skin friction, or, alternately, 50 percent of the ultimate compressive axial load capacity may be assumed for uplift capacity. Uplift capacity need not be taken as less than the weight of the pile (buoyancy considered).

6.4.5—Footing Joint Shear for SDCs C and D

All footing to column moment resistive joints in SDCs C and D shall be proportioned such that the principal stresses meet the following criteria:

Principal compression:

$$p_c \leq 0.25\, f'_c \tag{6.4.5-1}$$

Principal tension:

$$|p_t| \leq 0.38\sqrt{f'_c} \tag{6.4.5-2}$$

in which:

$$p_t = \frac{f_v}{2} - \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \tag{6.4.5-3}$$

$$p_c = \frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \tag{6.4.5-4}$$

$$v_{jv} = \frac{T_{jv}}{B_{eff}^{ftg} D_{ftg}} \tag{6.4.5-5}$$

$$T_{jv} = T_c - \sum T_{(i)}^{pile} \tag{6.4.5-6}$$

where: B_{eff}^{ftg} = effective width of footing (in.)

For circular columns:

$$B_{eff}^{ftg} = \sqrt{2} D_{cj} \tag{6.4.5-7}$$

For rectangular columns:

$$B_{eff}^{ftg} = B_c + D_{cj} \tag{6.4.5-8}$$

and:

$$f_v = \frac{P_{col}}{A_{jh}^{ftg}} \tag{6.4.5-9}$$

in which:

A_{jh}^{ftg} = effective horizontal area at mid-depth of the footing assuming a 45° spread away from the boundary of the column in all directions as shown in Figure 1 (in.^{2})

For circular columns:

$$A_{jh}^{ftg} = (D_{cj} + D_{ftg})^2 \tag{6.4.5-10}$$

For rectangular columns:

$$A_{jh}^{ftg} = \left(B_c + D_{ftg}\right)\left(D_{cj} + D_{ftg}\right) \tag{6.4.5-11}$$

where:

D_{cj} = column width or diameter parallel to the direction of bending (in.)

B_c = diameter or width of column or wall measured normal to the direction of loading (ft)

D_{ftg} = depth of footing (in.)

P_{col} = column axial force including the effects of overturning (kip)

f'_c = uniaxial compressive concrete strength (ksi)

T_c = column tensile force associated with the column overstrength plastic hinging moment, M_{po} (kip)

$\sum T_{(i)}^{pile}$ = summation of the hold down force in the tension piles (kip)

Transverse joint reinforcement shall be provided in accordance with Article 8.8.8.

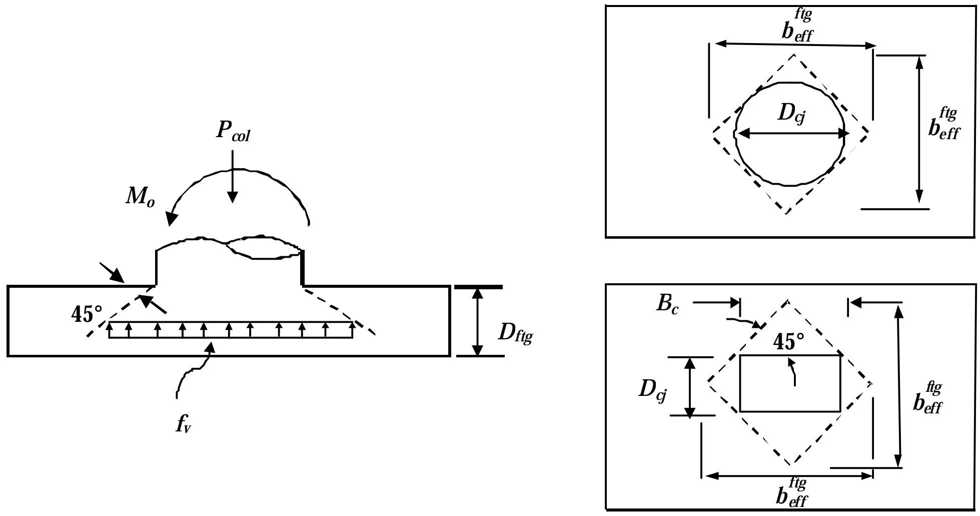

Figure 6.4.5-1—Effective Joint Width for Footing Joint Shear Stress

6.4.6—Effective Footing Width

For footings in SDCs C and D exhibiting rigid response and satisfying joint shear criteria, the entire width of the footing may be considered effective in resisting the column overstrength flexure and the associated shear when calculating the nominal section capacity. Otherwise, the effective footing width specified in Eq. 6.3.6-2 should be used.

6.5—DRILLED SHAFTS

Design requirements of drilled shafts shall conform to requirements of columns in SDC B, C, or D as applicable.

The effects of degradation and aggredation in a streambed on fixity and plastic hinge locations shall be considered for SDCs B, C, and D.

The effects of liquefaction on loss of *P-y* strength shall be considered for locations where a potential for liquefaction occurs following the requirements in Article 6.8.

A stable length shall be ensured for a single column/shaft. The stable length shall be determined in accordance with Article 10.7.3.12 of the *AASHTO LRFD Bridge Design Specifications*, except that a load factor of 1.0 should be applied to the calculated lateral loads for the foundation. Overstrength properties may be used for the foundation and column elements.

The ultimate geotechnical capacity of single column/shaft foundation in compression and uplift shall not be exceeded under maximum seismic loads.

C6.5

Various studies (Lam et al., 1998) have found that conventional *P-y* stiffnesses derived for driven piles are too soft for drilled shafts. This stiffer response is attributed to a combination of (1) higher unit side friction, (2) base shear at the bottom of the shaft, and (3) the rotation of the shaft. The rotation effect is often implicitly included in the interpretation of lateral load tests, as most lateral load tests are conducted in a free-head condition. A scaling factor equal to the ratio of shaft diameter to 0.61 m (2 ft) is generally applicable, according to Lam et al. (1998). The scaling factor is applied to either the linear subgrade modulus or the resistance value in the *P-y* curves. This adjustment is dependent on the construction method.

Base shear can also provide significant resistance to lateral loading for large diameter shafts. The amount of resistance developed in shear will be determined by conditions at the base of the shaft during construction. For dry conditions where the native soil is relatively undisturbed, the contributions for base shear can be significant. However, in many cases, the base conditions result in low interface strengths. For this reason, the amount of base shear to incorporate in lateral analyses will vary from case-to-case.

Lam et al. (1998) provides a detailed discussion of the seismic response and design of drilled shaft foundations. Their discussion includes a summary of procedures to determine the stiffness matrix required to represent the shaft foundation in most dynamic analyses.

Drilled shaft foundations will often involve a single shaft, rather than a group of shafts. In the single-shaft configuration, the relative importance of axial and lateral response changes relative to, for example, a group of driven piles. Without the equivalent of a pile cap, lateral-load displacement of the shaft becomes more critical than the load-displacement relationships discussed above for driven piles.

The depth for stable conditions will depend on the stiffness of the rock or soil. Lower stable lengths are acceptable if the embedment length and the strength of drilled shaft provide sufficient lateral stiffness with adequate allowances for uncertainties in soil stiffness. In Caltrans' practice, a stability factor of 1.2 is applied to single-column bents supported on a pile shaft.

6.6—PILE EXTENSIONS

Design requirements of pile extensions shall conform to requirements of columns in SDC B, C, or D as applicable.

The effects of degradation and aggredation in a streambed on fixity and plastic hinges locations shall be considered in SDCs B, C, and D.

The effects of liquefaction on loss of soil stiffness strength shall be considered in SDC B, C, and D. Group reduction factors shall be included in the analysis and design of pile extensions subjected to lateral loading in the transverse direction.

6.7—ABUTMENT DESIGN REQUIREMENTS

The participation of abutment walls in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads shall be considered in the seismic design of bridges in accordance with Article 5.2.

Abutment design shall be consistent with the demand model for the ERS used to assess intermediate substructure elements.

For conventional semi-gravity cantilever retaining wall construction, horizontal wall translation under dynamic active pressure loading is considered acceptable. However, rotational instability may lead to collapse and thus shall be prevented.

6.7.1—Longitudinal Direction Requirements

The seismic design of free-standing abutments should take into account forces arising from seismically induced lateral earth pressures, additional forces arising from wall inertia effects, and the transfer of seismic forces from the bridge deck through bearing supports that do not slide freely (e.g., elastomeric bearings).

For free-standing abutments that may displace horizontally without significant restraint (e.g., superstructure supported by sliding bearings), the design approach shall be similar to that of a free-standing retaining wall, except that longitudinal force from the bridge superstructure needs to be included in equilibrium evaluations, as the superstructure moves outward from the wall.

Earthquake-induced active earth pressures should be computed using a horizontal acceleration of not less than 50 percent of the acceleration coefficient, A_s, unless supported by displacement analyses and approved by the Owner. The pseudostatic Mononobe–Okabe method of analysis should be used for computing lateral active soil pressures during seismic loading. The effects of vertical acceleration may be omitted.

Abutment displacements having a maximum drift of four percent of the wall height may be tolerated. A limiting equilibrium condition should be checked in the horizontal direction. If necessary, wall design (initially based on a static service loading condition) should be modified to meet the above condition.

For monolithic abutments in which the abutment forms an integral part of the bridge superstructure, the abutment shall be designed using one of the two alternatives depending on the contribution level accounted for in the analytical model as discussed in Article 5.2:

- *Case 1: Earthquake Resisting System (ERS) without Abutment Contribution*—At a minimum, the abutment shall be designed to resist the active pressure applied by the abutment backfill.
- *Case 2: Earthquake Resisting System (ERS) with Abutment Contribution*—If the abutment is part of the ERS and required to mobilize soil resistance, the full passive pressure may be used in developing the bridge model and should be used to design the end diaphragm.

For freestanding abutments that are restrained from horizontal displacement by anchors or concrete batter piles, earthquake-induced active earth pressures should be computed using a horizontal acceleration equal to the acceleration coefficient, A_s, as a first approximation. The Mononobe–Okabe analysis method may be used. Up to a 50 percent reduction in the horizontal acceleration may be used, provided that the various components of the restrained wall can accommodate the increased level of displacement demand.

C6.7.1

These Guide Specifications have been prepared to acknowledge the abutment to be used as an ERE and be a part of the ERS. If designed properly, the reactive capacity of the approach fill can provide significant benefit to the bridge-foundation system.

Use of the 50 percent reduction in A_s in the determination of seismic active earth pressure assumes that several inches of permanent movement of the wall will be permissible. The form of this movement will likely be a combination of sliding with some rotation. The potential consequences of this movement need to be considered when using the 50 percent reduction in A_s as a basis for the Mononobe–Okabe calculation.

6.7.2—Transverse Direction Requirements

The provisions outlined in Article 5.2.4 shall be followed depending on the mechanism of transfer of superstructure transverse inertial forces to the bridge abutments and following the abutment contribution to the ERS applicable for SDCs C and D. These provisions should be considered for SDC B.

6.7.3—Other Requirements for Abutments

To minimize potential loss of bridge access arising from abutment damage, monolithic or end diaphragm construction should be considered for bridges less than 500 ft.

Settlement or approach slabs providing structural support between approach fills and abutments are recommended for all bridges in SDC D. Slabs shall be adequately linked to abutments using flexible ties.

For SDC D, the abutment skew should be minimized. The tendency for increased displacements at the acute corner of bridges with skewed abutments above 20° should be considered. In the case in which a large skew cannot be avoided, sufficient support length in conjunction with an adequate shear key shall be designed to ensure against any possible unseating of the bridge superstructure.

C6.7.3

During strong ground shaking such as will occur for SDC D, loose to medium dense soils making up the approach fill or located below the approach fill can densify. This densification will result in settlement of the roadway surface. The amount of settlement can range from negligible to a foot or more, particularly if layers of saturated sands or silts liquefy. This potential for settlement should be established during the geotechnical investigation for the site.

The differential settlement between a pile-supported abutment and the approach fill can result in a serious safety issue. Therefore, the recommended practice is to construct an approach slab that will provide a smooth transition between the abutment and the approach fill.

6.8—LIQUEFACTION DESIGN REQUIREMENTS

A liquefaction assessment shall be conducted for SDC C and D if both of the following conditions are present:

- *Groundwater Level*: The groundwater level anticipated at the site is within 50 ft of the existing ground surface or the final ground surface, whichever is lower.
- *Soil Characteristics*: Low plasticity silts and sands within the upper 75 ft are characterized by one of the following conditions: (1) the corrected standard penetration test (*SPT*) blow count, $(N_1)_{60}$, is less than or equal to 25 blows/ft in sand and nonplastic silt layers, (2) the corrected cone penetration test (*CPT*) tip resistance, q_{ciN}, is less than or equal to 150 in sand and in non-plastic silt layers, (3) the normalized shear wave velocity, V_{s1}, is less than 660 fps, or (4) a geologic unit is present at the site that has been observed to liquefy in past earthquakes.

Where loose to very loose saturated sands are within the subsurface soil profile such that liquefaction of these soils could impact the stability of the structure, the potential for liquefaction in SDC B should also be considered as discussed in the commentary.

For sites that require an assessment of liquefaction, the potential effects of liquefaction on soils and foundations shall be evaluated. The assessment shall consider the following effects of liquefaction:

- Loss in strength in the liquefied layer or layers,

C6.8

All of the following general conditions are necessary for liquefaction to occur:

- A sustained ground acceleration that is large enough and acting over a long enough period of time to develop excess pore-water pressure, thereby reducing effective stress and soil strength.
- Predominantly cohesionless soil that has the right gradation and composition. Liquefaction has occurred in soils ranging from low plasticity silts to gravels. Clean or silty sands and nonplastic silts are most susceptible to liquefaction.
- The state of the soil is characterized by a density that is low enough for the soil to exhibit contractive behavior when sheared undrained under the initial effective overburden stress.
- The presence of groundwater, resulting in a saturated or nearly saturated soil.

Methods used to assess the potential for liquefaction range from empirically-based design methods to complex numerical, effective stress methods that can model the time-dependent generation of pore-water pressure and its effect on soil strength and deformation. Furthermore, dynamic performance soil tests such as cyclic simple shear or cyclic triaxial tests can be used to assess liquefaction susceptibility and behavior to be used as input for liquefaction analysis and design.

- Liquefaction-induced ground settlement, and
- Flow failures, lateral spreading, and slope instability.

For sites where liquefaction occurs around bridge foundations, bridges should be analyzed and designed in two configurations as follows:

- *Nonliquefied Configuration*: The structure should be analyzed and designed, assuming no liquefaction occurs, using the ground response spectrum appropriate for the site soil conditions in a nonliquefied state.
- *Liquefied Configuration*: The structure as designed in nonliquefied configuration above should be reanalyzed assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance for lateral and axial deep foundation response analyses consistent with liquefied soil conditions (i.e., modified *P-y* curves, modulus of subgrade reaction, or *t-z* curves). The design spectrumshould be the same as that used in a nonliquefied configuration.

With the Owner's approval, or as required by the Owner, a site-specific response spectrum that accounts for the modifications in spectral content from the liquefying soil may be developed. Unless approved otherwise by the Owner, the reduced response spectrum resulting from the site-specific analyses shall not be less than two-thirds of the spectrum at the ground surface developed using the general procedure described in Article 3.4.1 modified by the site coefficients in Article 3.4.2.3.

The Designer should provide explicit detailing of plastic hinging zones for both cases mentioned above since it is likely that locations of plastic hinges for the liquefied configuration are different than locations of plastic hinges for the nonliquefied configuration. Design requirements including shear reinforcement should be met for the liquefied and nonliquefied configuration. Where liquefaction is identified, plastic hinging in the foundation may be permitted with the Owner's approval provided that the provisions of Article 3.3 are satisfied.

For those sites where liquefaction-related permanent lateral ground displacements (e.g., flow, lateral spreading, or slope instability) are determined to occur, the effects of lateral displacements on the bridge and retaining structures should be evaluated. These effects can include increased lateral pressure on bridge foundations and retaining walls.

The effects of liquefaction-related, permanent lateral ground displacements on bridge and retaining wall performance should be considered separate from the inertial evaluation of the bridge structures. However, if large magnitude earthquakes dominate the seismic hazards, the bridge response evaluation should consider the potential simultaneous occurrence of:

- Inertial response of the bridge, and loss in ground response from liquefaction around the bridge foundations, and

The most common method of assessing liquefaction involves the use of empirical methods (e.g., Youd et al., 2001). These methods provide an estimate of liquefaction potential based on *SPT* blowcounts, *CPT* cone tip resistance, or shear wave velocity. This type of analysis should be conducted as a baseline evaluation, even when more rigorous methods are used.

Youd et al. (2001) summarizes the consensus of the profession up to year 2000 regarding the use of the simplified methods. Since the publication of this consensus paper, various other modifications to the consensus approach have been introduced, including those by Cetin et al. (2004), Moss et al. (2006), and Boulanger and Idriss (2006). These more recent methods account for additions to the database on liquefaction, as well as refinements in the interpretation of case history data. The newer methods potentially offer improved estimates of liquefaction potential and can be considered for use.

The simplified empirical methods are suited for use to a maximum depth of approximately 75 ft. This depth limit relates to the database upon which the original empirical method was developed. Most of the database was from observations of liquefaction at depths less than 50 to 60 ft. Extrapolation of the simplified method beyond 75 ft is therefore of uncertain validity. This limitation should not be interpreted as meaning liquefaction does not occur beyond 75 ft. Rather, different methods should be used for greater depths, including the use of site-specific ground motion response modeling in combination with liquefaction testing in the laboratory.

The magnitude for the design earthquake must be determined when conducting liquefaction assessments using the simplified empirical procedures. The earthquake magnitude used to assess liquefaction can be determined from earthquake deaggregation data for the site, available through the USGS national seismic hazard website http://earthquake.usgs.gov/research/hazmaps/ based on the 975-yr return period, i.e., five percent in 50 yr within the USGS website. If a single or a few larger magnitude earthquakes dominate the deaggregation, the magnitude of the single dominant earthquake or the mean of the few dominant earthquakes in the deaggregation should be used.

Liquefaction is generally limited to granular soils, such as sands and nonplastic silts. Loose gravels also can liquefy if drainage is prevented such as might occur if a layer of clay or frozen soil is located over the gravel. Methods for eliminating sites based on soil type have been developed, as discussed by Youd et al, (2001), Bray and Sancio (2006), and Boulanger and Idriss (2006). These methods can be used to screen the potential for liquefaction in certain soil types. In the past soil screening with regard to silts was done using the Chinese criteria (Kramer, 1996). Recent studies (Bray and Sancio, 2006; Boulanger and Idriss, 2006) indicate that the Chinese criteria are unconservative, and therefore their use should be discontinued.

Predicted amounts of permanent lateral displacement of the soil.

Two criteria for assessing liquefaction susceptibility of soils have been recently proposed as replacements to the Chinese criteria:

Boulanger and Idriss (2006) recommend considering a soil to have clay-like behavior (i.e., not susceptible to liquefaction) if the plasticity index (PI) $\geq$ 7.

Bray and Sancio (2006) suggest that a soil with a $PI < 12$ and a ratio of water content to liquid limit (wc/LL) > 0.85 will be susceptible to liquefaction.

There is no current consensus on the preferred of the two criteria, and, therefore, either method may be used, unless the Owner has a specific preference.

To determine the location of soils that are adequately saturated for liquefaction to occur, the seasonally averaged groundwater elevation should be used. Groundwater fluctuations caused by tidal action or seasonal variations will cause the soil to be saturated only during a limited period of time, significantly reducing the risk that liquefaction could occur within the zone of fluctuation.

Liquefaction evaluation is required only for sites meeting requirements for SDC C and D, provided that the soil is saturated and of a type that is susceptible to liquefaction. For loose to very loose sand sites (e.g., $(N_1)_{60} < 10$ bpf or $q_{c1N} < 75$), a potential exists for liquefaction in SDC B, if the acceleration coefficient, A_s, is 0.15 or higher. The potential for and consequences of liquefaction for these sites will depend on the dominant magnitude for the seismic hazard. As the magnitude decreases, the liquefaction resistance of the soil increases due to the limited number of earthquake loading cycles. Generally, if the magnitude is 6 or less, even in these very loose soils, either the potential for liquefaction is very low or the extent of liquefaction is very limited. Nevertheless, a liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact bridge stability and A_s is greater than or equal to 0.15. These loose to very loose sands are likely to be present in hydraulically placed fills and alluvial or estuarine deposits near rivers and waterfronts.

During liquefaction, pore-water pressure build-up occurs, resulting in loss of strength and then settlement as the excess pore-water pressures dissipate after the earthquake. The potential effects of strength loss and settlement include:

- *Slope Failure, Flow Failure, or Lateral Spreading*: The strength loss associated with pore-water pressure build-up can lead to slope instability. Generally, if the factor of safety against liquefaction is less than approximately 1.2 to 1.3, a potential for pore-water pressure build-up will occur, and the effects of this build-up should be assessed. If the soil liquefies, the stability is determined by the residual strength of the soil. The residual strength of liquefied soils can be determined using empirical methods developed by Seed and Harder (1990), Olson and Stark (2002), and others. Loss of lateral resistance can allow abutment soils to move laterally, resulting in bridge substructure distortion and unacceptable deformations and moments in the superstructure.

- *Reduced Foundation Bearing Resistance*: Liquefied strength is often a fraction of nonliquefied strength. This loss in strength can result in large displacements or bearing failure. For this reason, spread footing foundations are not recommended where liquefiable soils occur unless the spread footing is located below the maximum depth of liquefaction or soil improvement techniques are used to mitigate the effects of liquefaction.

- *Reduced Soil Stiffness and Loss of Lateral Support for Deep Foundations*: This loss in strength can change the lateral response characteristics of piles and shafts under lateral load.

- *Vertical Ground Settlement as Excess Pore-Water Pressures Induced by Liquefaction Dissipate, Resulting in Downdrag Loads on Deep Foundations*: If liquefaction-induced downdrag loads can occur, the downdrag loads should be assessed as specified in Article 3.11.8 in the *AASHTO LRFD Bridge Design Specifications*.

Most liquefaction-related damage to bridges during past earthquakes has been the result of lateral movement of the soil, causing severe column distortion and potential structure collapse. Therefore, a thorough analysis of the effects of lateral soil movement due to liquefaction on the structure is necessary. If there is potential for significant soil movement, the structure design should meet the requirements of SDC D, as specified in Article 3.5.

The effects of liquefaction will depend in large part on the amount of soil that liquefies and the location of the liquefied soil with respect to the foundation. On sloping ground, lateral flow, spreading, and slope instability can occur on relatively thin layers of liquefiable soils, whereas the effects of thin liquefied layer on the lateral response of piles or shafts (without lateral ground movement) may be negligible. Likewise, a thin liquefied layer at the ground surface results in essentially no downdrag loads, whereas the same liquefied layer deeper in the soil profile could result in large downdrag loads. Given these potential variations, the site investigation plays a fundamental part of the liquefaction assessment. Article 6.2 in these *Guide Specifications* and Section 10 of the *AASHTO LRFD Bridge Design Specifications* identify requirements for site investigations.

When assessing the effects of liquefaction on bridge response, the recommendation in these Guide Specifications require that structure be designed for two cases, one in which the full seismic acceleration is applied to the structure assuming the soil does not liquefy, and one in which the full seismic acceleration is applied to the structure assuming the soil does liquefy but the spectrum is unchanged by liquefaction. This approach should produce conservative results for bridges with periods less than 1 sec. However, Youd and Carter (2005) suggest that at periods greater than 1 sec, it is possible for liquefaction to result in higher spectral accelerations than occur for equivalent nonliquefied cases, all other conditions being equal. For Site Class C or D and bridges with periods greater than 1 sec, the Designer may consider using a response spectrum constructed using Site Class E for the liquefied condition. Alternately, site-specific ground motion response evaluations may be used to evaluate this potential.

There is currently no consensus on how to address this issue of timing of seismic acceleration and the development of full liquefaction and its combined impact on the structure without resorting to more rigorous analyses, such as by using nonlinear, effective stress methods. In general, the larger the earthquake magnitude (e.g., $M > 8$), the longer the period of time over which strong shaking acts, and the more likely the strong shaking and liquefaction effects will be acting concurrently. The smaller the earthquake magnitude, the more likely that these two effects will not be concurrent, in which case the peak inertial response of the bridge may occur before much, if any, reduction in soil support from liquefaction occurs.

Site-specific dynamic ground motion response analyses offer one method of evaluating the effects of pore-water pressure increases and timing on the development of the response spectrum. These analyses can be conducted using a nonlinear, effective stress method that accounts for the build-up in pore-water pressure and stiffness degradation in liquefiable layers. Use of this approach requires considerable skill in terms of selecting model parameters, particularly the pore pressure model. The complexity of this approach is such that Owner's approval is mandatory, and it is highly advisable that a independent peer review panel with expertise in nonlinear, effective stress modeling be used to review the methods and the resulting spectrum.

The limit of two-thirds for reduction of the liquefied response spectrum below the nonliquefied spectrum is meant to apply to any ordinate of the response spectrum. Generally, liquefied conditions may produce significant reductions in the shorter period range, but the reductions will be smaller or could be increased over nonliquefied conditions in the longer period range over about 1 to 2 sec. The developer of the site response analysis should capture accurate estimates of response for all periods that could be of importance in both nonliquefied and liquefied conditions. This consideration is particularly important if the conventional spectral shapes of Article 3.4.1 are being used.

The timing of liquefaction relative to the development of strong shaking also can be an important consideration for sites where lateral ground movement occurs. Both the development of liquefaction and the ground movement are dependent on the size and magnitude of the earthquake, but they do not necessarily occur at the same time. This issue is especially important when determining how to combine the inertial response of the structure and the response to lateral movement of the soil against the foundations and other substructure elements due to lateral spreading, slope instability, and flow failure. Current practice is to consider these two mechanisms to be independent, and therefore, the analyses are decoupled; i.e., the analysis is first performed to evaluate inertial effects during liquefaction following the same guidance as for level-ground sites, and then the foundation is evaluated for the moving ground, but without the inertial effects of the bridge superimposed. For critical bridges or in areas where very large magnitude earthquakes could occur, detailed studies addressing the two mechanisms acting concurrently may be warranted. This timing issue also affects liquefaction-induced downdrag, in that settlement and downdrag generally does not occur until the pore pressures induced by ground shaking begin to dissipate after shaking ceases.

For assessment of existing structures, the Designer should consider using SDC D regardless of the magnitude of A_s, even when significant lateral soil movement is not expected, if the structure is particularly weak with regard to its ability to resist the forces and displacements that could be caused by liquefaction. Examples of weaknesses that could exacerbate the impact of liquefaction to the structure include presence of shallow foundations, deep foundations tipped in liquefiable soil, very limited bridge support lengths that have little tolerance of lateral movement of the substructure, deterioration of superstructure or substructure components due to advanced age of the structure or severe environmental conditions, and the absence of substructure redundancy.

These Guide Specifications allow for development of full plastic deformation of in-ground hinges in piles (similar to above ground limits) during liquefaction with Owner's approval. If inelastic deformations are expected in the foundation, then the Owner may consider installation of devices that permit post-earthquake assessment; for example installation of inclinometer tubes in drilled shafts permits limited evaluation of the deformations of the foundation, which would otherwise be impossible to inspect at any significant depth. Permitting inelastic behavior below the ground implies that the shaft or piles will be damaged, possibly along with other parts of the bridge, and may need to be replaced.

Design options range from (a) an acceptance of the movements with significant damage to the piles and columns if the movements are large (possibly requiring demolition but still preserving the no-collapse philosophy) to (b) designing the piles to resist the forces generated by lateral spreading. Between these options are a range of mitigation measures to limit the amount of movement to tolerable levels for the desired performance objective. However, tolerable structural movements should be evaluated quantitatively.

Quantitative assessment of liquefaction-induced deformations on foundations may be accomplished using the nonlinear static "pushover" methodology. However, such analysis is complicated by the need to model the nonlinear *P-y* behavior of the liquefied soil along with the nonlinear behavior of the structure. Analyses where the liquefied soil is represented by appropriate residual resistance (*P-y* curves or modulus of sub-grade reaction values) will generally provide conservative results for the actual inelastic behavior of the foundation structural elements. The approach for such analyses should be developed on a case-by-case basis due to the varied conditions found in liquefiable sites. Careful coordination between the geotechnical and structural engineers is essential to estimating the expected response and to evaluating whether the structure can tolerate the response. Often mitigation strategies may be required to reduce structural movements.

Mitigation of the effects of liquefaction-induced settlement or lateral soil movement may include ground stabilization to either prevent liquefaction or add strength to keep soil deformation from occurring, foundation or superstructure modifications to resist the forces and accommodate the deformations that may occur, or both.

It is often cost prohibitive to design the bridge foundation system to resist the loads imposed by liquefaction-induced lateral loads, especially if the depth of liquefaction extends more than about 20 ft below the ground surface and if a nonliquefied crust is part of the failure surface. Ground improvement to mitigate the liquefaction hazard is the likely alternative if it is not practical to design the foundation system to accommodate the lateral loads.

The primary ground improvement techniques to mitigate liquefaction fall into five general categories, namely removal and replacement, densification, reinforcement, altering the soil composition, and enhanced drainage. Any one or a combination of methods can be used. However, drainage improvement is not currently considered adequately reliable to prevent liquefaction-induced, excess pore-water pressure build-up due to (1) the time required for excess pore-water pressures to dissipate through the drainage paths, and (2) the potential for drainage materials to become clogged during installation and in service. In addition, with drainage enhancements some settlement is still likely. Therefore, drainage enhancements should not be used as a means to fully mitigate liquefaction. For further discussion of ground improvement methods, see FHWA-SA-98-086, *Ground Improvement Technical Summaries* (Elias, et al., 2000); FHWA-SA-95-037; Geotechnical Engineering Circular No. 1, *Dynamic Compaction* (Lukas, 1995); and FHWA/RD-83/O2C, *Design and Construction of Stone Columns* (Barkdale and Bachus, 1983).

The use of large diameter shafts in lieu of the conventional pile cap foundation type may be considered in order to achieve the lateral strength and stiffness required to sustain the column demand while minimizing the foundation exposed surface area normal to the lateral flow direction.

SECTION 7: STRUCTURAL STEEL COMPONENTS

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SECTION 7:
STRUCTURAL STEEL COMPONENTS

7.1—GENERAL

The Engineer shall demonstrate that a clear, straightforward load path exists (see Figure 1) within the superstructure, through the bearings or connections to the substructure, within the substructure, and ultimately to the foundation. All components and connections shall be capable of resisting the imposed seismic load effects consistent with the chosen load path.

The flow of forces in the prescribed load path shall be accommodated through all affected components and their connections including, but not limited to, flanges and webs of main beams or girders, cross-frames, steel-to-steel connections, slab-to-steel interfaces, and all components of the bearing assembly from bottom flange interface through the anchorage of anchor bolts or similar devices in the substructure. The substructure shall also be designed to transmit the imposed force effects into the soils beneath the foundations.

The analysis and design of end diaphragms and cross-frames shall include the horizontal supports at an appropriate number of bearings, consistent with Article 7.8 and Article 7.9.

The following requirements shall apply to bridges with either:

- A concrete deck that can provide horizontal diaphragm action, or
- A horizontal bracing system in the plane of the top flange, which in effect provides diaphragm action.

A load path (see Figure 1) shall be established to transmit the inertial loads to the foundation on the basis of the stiffness characteristics of the deck, diaphragms, cross-frames, and lateral bracing. Unless a more refined analysis is made, an approximate load path shall be assumed as follows:

- The seismic inertia loads in the deck shall be assumed to be transmitted directly to the bearings through end diaphragms or cross-frames, and
- The development and analysis of the load path through the deck or through the top lateral bracing, if present, shall use assumed structural actions analogous to those used for the analysis of wind loadings.

Reference to *AASHTO LRFD Bridge Design Specifications* shall apply to the 2007 fourth edition, with subsequent updates pertinent to the Articles mentioned in this document.

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Most steel components of bridges are not expected to behave in a cyclic inelastic manner during an earthquake. The provisions of this Article are only applicable to the limited number of components (such as specially detailed ductile substructures or ductile diaphragms) whose stable hysteretic behavior is relied on to ensure satisfactory bridge seismic performance. The seismic provisions of this Article are not applicable to the other steel members expected to remain elastic during seismic response. In most steel bridges, the steel superstructure is expected (or can be designed) to remain elastic.

One span of the San Francisco–Oakland Bay Bridge collapsed because of loss of support at its bearings during the 1989 Loma Prieta earthquake, and another bridge suffered severe bearing damage (EERI, 1990). The end diaphragms of some steel bridges suffered damage in a subsequent earthquake in northern California (Roberts, 1992). During the 1994 Northridge earthquake, some steel bridges located close to the epicenter sustained damage to their reinforced concrete abutments, connections between concrete substructures and steel superstructures, steel diaphragms, or structural components near the diaphragms (Astaneh-Asl et al., 1994). Furthermore, a large number of steel bridges were damaged by the 1995 Hyogoken–Nanbu (Kobe) earthquake. The concentration of steel bridges in the area of severe ground motion was considerably larger than for any previous earthquake, and some steel bridges collapsed. Many steel piers, bearings, seismic restrainers, and superstructure components suffered significant damage (Bruneau, Wilson, and Tremblay, 1996). This experience emphasizes the importance of ductile detailing in the critical elements of steel bridges.

Research on the seismic behavior of steel bridges (e.g., Astaneh-Asl, Shen, and Cho, 1993; Dicleli and Bruneau, 1995a, 1995b; Dietrich and Itani, 1999; Itani et al., 1998a; McCallen and Astaneh-Asl, 1996; Seim, Ingham, and Rodriguez, 1993; Uang et al., 2000, 2001; Zahrai and Bruneau, 1998) and findings from recent seismic evaluation and rehabilitation projects (e.g., Astaneh and Roberts, 1996; Ballard et al., 1996; Billings et al., 1996; Dameron et al., 1995; Donikian et al., 1996; Gates et al., 1995; Imbsen et al., 1997; Ingham et al., 1996; Jones et al., 1997; Kompfner et al., 1996; Maroney, 1996; Prucz et al., 1997; Rodriguez and Ingham, 1996; Schamber et al., 1997; Shirolé and Malik, 1993; Vincent et al., 1997) further confirm that seismically induced damage is likely in steel bridges subjected to large earthquakes and that appropriate measures shall be taken to ensure satisfactory seismic performance.

The intent of this section is to ensure the ductile response of steel bridges during earthquakes. First, effective load paths should be provided for the entire structure as outlined herein. Following the concept of capacity design, the load effect arising from the inelastic deformations of part of the structure should be properly considered in the design of other elements that are within its load path.

Second, steel substructures should be detailed to ensure stable ductile behavior. Note that the term "substructure" here refers to structural systems exclusive of bearings and articulations. Steel substructures require ductile detailing to provide satisfactory seismic performance.

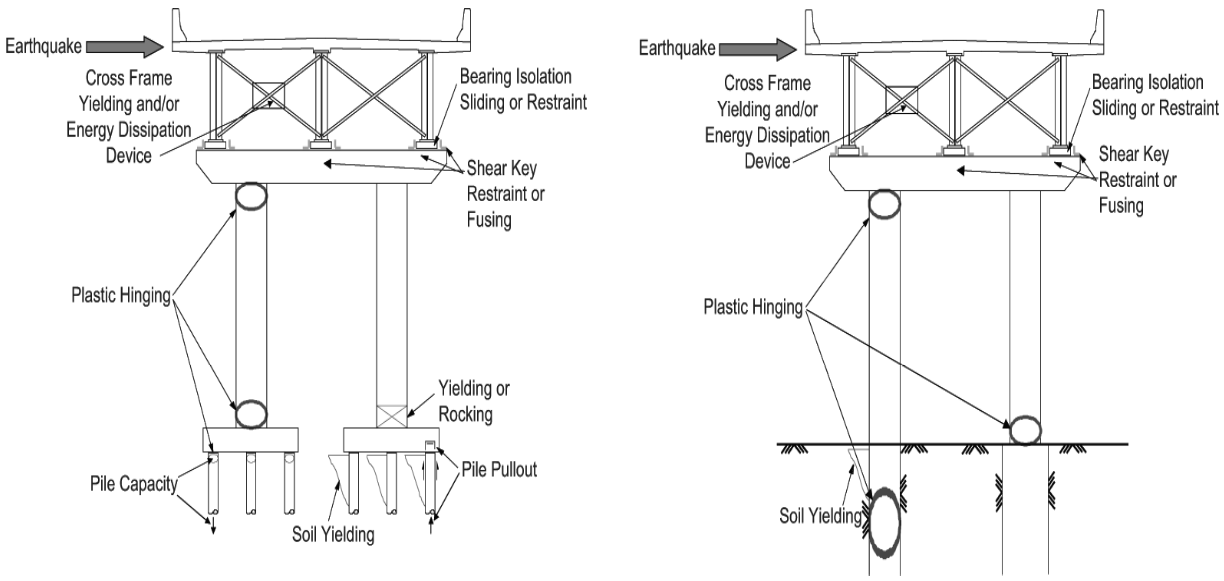
If the forces from the substructure corresponding to the overstrength condition are used to design the superstructure, the distribution of these forces may not be the same as that of the elastic demand analysis forces. The Engineer may calculate a more refined distribution of the inertial forces present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

Diaphragms, cross-frames, lateral bracing, bearings, and substructure elements are part of an 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 followed herein, it is assumed that ductile plastic hinging in substructure or seismic isolator units is the primary source of energy dissipation.

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

A continuous load 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.

All bearings in a bridge 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 should be taken into account in the design of the end diaphragms and pier diaphragms. Also, a significant change in the load distribution between end diaphragm members and the pier may occur.



(a) Pile Footing **(b) Drilled Shaft**

Note: Affected components shown are inclusive to Types 1, 2, and 3 and do reflect specific components that are permitted to fuse under Type 1, 2, or 3 specified in Article 7.2.

Figure 7.1-1—Seismic Load Path and Affected Components

7.2—PERFORMANCE CRITERIA

This Article shall apply to the design of steel components. Those components shall be classified into two categories: ductile and essentially elastic. On the basis of the characteristics of the bridge structure, the Designer may use one of three options for a seismic design strategy:

- *Type 1*—Design a ductile substructure with an essentially elastic superstructure.
- *Type 2*—Design an essentially elastic substructure with a ductile superstructure.
- *Type 3*—Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.

The provisions in this Section shall be used in conjunction with the forced-based seismic design procedure utilized in the *AASHTO LRFD Bridge Design Specifications.*

In this Section, reference to an essentially elastic component is used where the force demand to the nominal capacity ratio of any member in the superstructure is less than 1.5.

C7.2

The design of ductile steel structural response is based primarily on a force-based, not displacement-based, seismic design approach.

Seismic design forces for individual members and connections of bridges identified as Type 2 shall be determined by dividing the unreduced elastic forces by the appropriate response modification factor (R), as specified in Article 7.2.2. These factors shall be used only when all of the design requirements of this Section are satisfied. A combination of orthogonal seismic forces equivalent to the orthogonal seismic displacement combination specified in Article 4.4 shall be used to obtain the unreduced elastic forces.

The nominal capacity of a member, connection, or structure shall be based upon the expected yield strength, F_{ye}, and the nominal dimensions and details of the final section(s), calculated with all material resistance factors, ϕ, taken as 1.0.

Refer to Article 3.10.7 of the *AASHTO LRFD Bridge Design Specifications* for additional guidance on Response Modification Factors.

7.2.1—Type 1

For Type 1 structures, the Designer shall refer to Section 8 or Article 7.5 and Article 7.6, as applicable, of this document on designing for a ductile substructure as applicable to Seismic Design Categories (SDCs) C and D.

7.2.2—Type 2

For Type 2 structures, the design of the superstructure shall be accomplished using a force-based approach with an appropriate reduction for ductility. Those factors shall be used for the design of all ductile load-carrying members. For SDC B, C, or D, a reduction factor, R, equal to 3 is used for ordinary bracing that is a part of the earthquake-resistant system (ERS) not having ductile end diaphragms as defined in Article 7.4.6. The force reduction factor, R, may be increased up to 4 for SDC D as indicated in Article 7.4.6.

For simply supported spans with ductile end-diaphragms in compliance with Article 7.4.6, the location of the diaphragms shall, as a minimum, be placed at the ends of each span.

For continuous spans where ductile diaphragms are used, the location of diaphragms shall, as a minimum, be placed over each bent and one cross-frame spacing adjacent to the opposite faces of the bent. The use of special diaphragms at opposite faces of an in-span hinge should be carefully assessed to ensure adequate vertical load capacity of the in-span hinge when subjected to deformations in the inelastic range.

7.2.3—Type 3

For Type 3 structures, the Designer shall assess the overstrength capacity for the fusing interface including shear keys and bearings, then design for an essentially elastic superstructure and substructure. The minimum lateral design force shall be calculated using an acceleration of $0.4g$ or the elastic seismic force, whichever is smaller. If isolation devices are used, the superstructure shall be designed as essentially elastic (see Article 7.8).

7.3—MATERIALS

The provisions of Section 6 of the *AASHTO LRFD Bridge Design Specifications* for structural steel that is designed to remain essentially elastic during the design seismic event shall apply as applicable.

For SDCs C and D, ductile substructure elements and ductile end diaphragms, as defined in Article 7.4.6 inclusive through Article 7.5, shall be made of steels satisfying the requirements of:

- ASTM A 709 Grade 50,
- ASTM A 709 Grade 50W,
- ASTM A 992,
- ASTM A 500 Grade B, and
- ASTM A 501.

For ASTM A 709 Grade 50 and Grade 50W and ASTM A 992 steels, the expected yield stress, F_{ye}, shall be taken as 1.1 times the nominal yield stress, F_y.

For ASTM A 500 Grade B and ASTM A 501 steels, the expected yield stress, F_{ye}, shall be taken as 1.4 times the nominal yield stress.

For SDC B, ASTM A 709 Grade 36 can be used. For ASTM A 709 Grade 36 steel, the expected yield stress, F_{ye}, shall be taken as 1.5 times the nominal yield stress.

For SDC C and D, ductile concrete filled steel pipe as defined in Article 7.6 shall be made of steels satisfying the requirements of:

- ASTM A 53 Grade B
- API 5L X52

For ASTM A 53 Grade B steel, the expected yield stress, F_{ye}, shall be taken as 1.5 times the nominal yield stress.

For API 5L X52 steel, the expected yield stress, F_{ye}, shall be taken as 1.2 times the nominal yield stress, F_y.

The overstrength capacity shall be taken as the resistance of a member, connection, or structure based on the nominal dimensions and details of the final section(s) chosen. The overstrength capacity shall be determined using the expected yield stress, F_{ye}, and overstrength factor, λ_{mo}, as specified in Article 4.11.2.

C7.3

To ensure that the objective of capacity design is achieved, Grade 36 steel is not permitted for the components expected to respond in a significantly ductile manner. Grade 36 is difficult to obtain, and contractors often substitute it with Grade 50 steel. Furthermore, it has a wide range in its expected yield and ultimate strength and large overstrength factors to cover the anticipated range of property variations. The common practice of dual certification for rolled shapes, recognized as a problem from the perspective of capacity design following the Northridge earthquake, is now becoming progressively more common also for steel plates. As a result, only Grade 50 steels are allowed for structures in SDCs C and D.

In those instances when Grade 36 steel is permitted for use (SDC B), capacity design should be accomplished assuming an effective yield strength factor of 1.5.

The use of A 992 steel is explicitly permitted. Even though this ASTM grade is currently designated for "shapes for buildings or bridges," work currently is being done to expand applicability to other shapes. ASTM A 992 steel, developed to ensure good ductile seismic performance, is specified to have both a minimum and maximum guaranteed yield strength and may be worthy of consideration for ductile energy-dissipating systems in steel bridges.

Because other steels may be used, provided that they are comparable to the approved Grade 50 steels, high-performance steel (HPS) Grade 50 would be admissible, but not HPS Grade 70W (or higher). Based on limited experimental data available, it appears that HPS Grade 70W has a lower rotational ductility capacity and may not be suitable for "ductile fuses" in seismic applications.

When other steels are used for energy-dissipation purposes, it is the responsibility of the Designer to assess the adequacy of material properties available and design accordingly. Other steel members expected to remain elastic during earthquake should be made of steels conforming to Article 6.4 of the *AASHTO LRFD Bridge Design Specifications*.

The American Petroleum Institute (API) provides more stringent requirements for steel pipes. Other steel pipe materials are permitted with the Owner's approval.

The capacity design philosophy and the concept of capacity-protected elements are defined in Article 4.11.

Welding requirements shall be compatible with the *AASHTO/AWS D1.5M/D1.5:2007 Bridge Welding Code*. Undermatched welds are not permitted for special seismic hysteretic energy-dissipating systems (such as ductile substructures and ductile diaphragms).

Steel members expected to undergo significant plastic deformations during a seismic event shall meet the toughness requirements of ASTM Standard A 709/A 709M, Paragraph 10, "Fracture Critical (F) Tension Members, Zone 3." Welds metal connecting these members shall meet the toughness requirements specified in the *AASHTO/AWS D1.5M/D1.5:2007 Bridge Welding Code* for Zone III (ANSI/AASHTO/AWS, 2007).

Steel members and weld materials should have adequate notch toughness to perform in a ductile manner over the range of expected service temperatures. The A 709/A 709M S84 "Fracture-Critical Material Toughness Testing and Marking" requirement, typically specified when the material is to be used in a fracture-critical application as defined by AASHTO, is deemed to be appropriate to provide the level of toughness sought for seismic resistance. For weld metals, the *AASHTO/AWS D1.5M/D1.5:2007 Bridge Welding Code* requirement for Zone III, familiar to the bridge engineering community, is similar to the 20 ft-lbs at -20°F requirement proposed by the SAC Joint Venture for weld metal in welded moment frame connections in building frames.

7.4—MEMBER REQUIREMENTS FOR SDCS C AND D

7.4.1—Limiting Slenderness Ratios

Bracing members shall have a slenderness ratio, KL/r, less than 120. The length of a member shall be taken between the points of intersection of members. An effective length factor, K, of 0.85 of compression members in braced structures shall be used unless a lower value can be justified by an appropriate analysis. The slenderness parameter λ_c for axial compressive load dominant members and λ_b for flexural dominant members shall not exceed the limiting values, λ_{cp} and λ_{bp}, respectively, as specified in Table 1.

C7.4.1

In the ductile design of concentrically braced frames in buildings, the slenderness ratio limits for braces, up until the late 1990s, were approximately 75 percent of the value specified here. The philosophy was to design braces to contribute significantly to the total energy dissipation when in compression. Member slenderness ratio was restricted because the energy absorbed by plastic bending of braces in compression diminishes with increased slenderness. To achieve these more stringent KL/r limits, particularly for long braces, designers have almost exclusively used tubes or pipes for the braces. This is unfortunate, as these tubular members are most sensitive to rapid local buckling and fracture when subjected to inelastic cyclic loading (in spite of the low width-to-thickness limits prescribed). Reviews of this requirement revealed that it may be unnecessary, provided that connections are capable of developing at least the member capacity in tension. This is partly because larger tension brace capacity is obtained when design is governed by the compression brace capacity and partly because low-cycle fatigue life increases for members having greater KL/r. As a result, seismic provisions for buildings (AISC, 2005; CSA, 2001) have been revised to permit members having greater KL/r values. The proposed relaxed limits used here are consistent with the adopted philosophy for buildings.

Table 7.4.1-1—Limiting Slenderness Parameters

Member Classification		Limiting Slenderness Parameter λ_{cp} or λ_{bp}	
Ductile Members	Axial Compression Load Dominant $\frac{P_u}{P_n} \geq \frac{M_u}{M_{ns}}$	λ_{cp}	0.75
	Flexural Moment Dominant $\frac{P_u}{P_n} < \frac{M_u}{M_{ns}}$	λ_{bp}	$\frac{0.086E}{F_y}$
Essentially Elastic/ Capacity Protected	Axial Compression Load Dominant $\frac{P_u}{P_n} \geq \frac{M_u}{M_{ns}}$	λ_{cp}	1.50
	Flexural Moment Dominant $\frac{P_u}{P_n} < \frac{M_u}{M_{ns}}$	λ_{bp}	$4.40\sqrt{\frac{E}{F_y}}$

in which:

Slenderness parameter of axial compressive load dominant members:

$$\lambda_c = \left(\frac{KL}{r\pi}\right)\sqrt{\frac{F_y}{E}} \qquad (7.4.1\text{-}1)$$

Slenderness parameter of flexural moment dominant members:

$$\lambda_b = \frac{L}{r_y} \qquad (7.4.1\text{-}2)$$

where:

M_u = factored moment demand acting on the member (kip-in.)

M_{ns} = nominal flexural moment strength of a member (kip-in.)

P_u = factored axial compressive load acting on the member (kips)

P_n = nominal axial compressive strength of a member (kips)

λ_{cp} = limiting slenderness parameter for axial compressive load dominant members

λ_{bp} = limiting slenderness parameter for flexural moment dominant members

K = effective length factor of the member

L = unsupported length of the member (in.)

r = radius of gyration (in.)

r_y = radius of gyration about minor axis (in.)

F_y = specified minimum yield strength of steel (ksi)

E = modulus of elasticity of steel (ksi)

7.4.2—Limiting Width–Thickness Ratios

For essentially elastic components, the width–thickness ratios shall not exceed the limiting value λ_r as specified in Table 1.

For ductile components, width–thickness ratios shall not exceed the value λ_p as specified in Table 1.

C7.4.2

Premature local buckling of braces prohibits the braced frames from sustaining many cycles of load reversal. Both laboratory tests and real earthquake observations have confirmed that premature local buckling significantly shortens the fracture life of hollow structural section (HSS) braces. The more stringent requirement on the *b/t* ratio for rectangular tubular sections subjected to cyclic loading is based on tests (Tang and Goel, 1987; Uang and Bertero, 1986). The *D/t* limit for circular sections is identical to that in the AISC plastic design specifications (AISC, 2005; Sherman, 1976).

Table 7.4.2-1—Limiting Width–Thickness Ratios

Description of Elements	Width–Thickness Ratios	Essentially Elastic Components, λ_r	Ductile Members, λ_p
Unstiffened Elements			
Flexure and uniform compression in flanges of rolled or built-up I-shaped sections	$\frac{b}{t}$	$0.56\sqrt{\frac{E}{F_y}}$	$0.30\sqrt{\frac{E}{F_y}}$
Uniform compression in flanges of H-pile sections	$\frac{b}{t}$	$0.56\sqrt{\frac{E}{F_y}}$	$0.45\sqrt{\frac{E}{F_y}}$
Uniform compression in legs of single angles, legs of double-angle members with separators, or flanges of tees	$\frac{b}{t}$	$0.45\sqrt{\frac{E}{F_y}}$	$0.30\sqrt{\frac{E}{F_y}}$
Uniform compression in stems of rolled tees	$\frac{d}{t}$	$0.75\sqrt{\frac{E}{F_y}}$	$0.30\sqrt{\frac{E}{F_y}}$
Stiffened Elements			
Rectangular HSS in axial compression and/or flexural compression	$\frac{b}{t}$ $\frac{h}{t_w}$	$1.40\sqrt{\frac{E}{F_y}}$	$0.64\sqrt{\frac{E}{F_y}}$ (tubes)
Unsupported width of perforated cover plates	$\frac{b}{t}$	$1.86\sqrt{\frac{E}{F_y}}$	$0.88\sqrt{\frac{E}{F_y}}$
All other uniformly compressed stiffened elements that are supported along two edges	$\frac{b}{t}$ $\frac{h}{t_w}$	$1.49\sqrt{\frac{E}{F_y}}$	$0.64\sqrt{\frac{E}{F_y}}$ (laced) $0.88\sqrt{\frac{E}{F_y}}$ (others)
Webs in flexural compression or combined flexural and axial compression	$\frac{h}{t_w}$	$5.70\sqrt{\frac{E}{F_y}\left(1-\frac{0.74P_u}{\phi_b P_y}\right)}$	If $P_u \le 0.125\phi_b P_y$, then: $3.14\sqrt{\frac{E}{F_y}\left(1-\frac{1.54P_u}{\phi_b P_y}\right)}$ If $P_u > 0.125\phi_b P_y$, then: $1.12\sqrt{\frac{E}{F_y}\left(2.33-\frac{P_u}{\phi_b P_y}\right)} \ge 1.49\sqrt{\frac{E}{F_y}}$
Longitudinally stiffened plates in compression	$\frac{b}{t}$	$0.66\sqrt{\frac{kE}{F_y}}$	$0.44\sqrt{\frac{kE}{F_y}}$
Round HSS in axial compression or flexure	$\frac{D}{t}$	$\frac{0.09E}{F_y}$	$\frac{0.044E}{F_y}$

in which:

If $n = 1$, then:

$$k = \left(\frac{8I_s}{bt^3}\right)^{1/3} \leq 4 \tag{7.4.2-1}$$

If $n = 2, 3, 4$, or 5, then:

$$k = \left(\frac{14.3I_s}{bt^3n^4}\right)^{1/3} \leq 4 \tag{7.4.2-2}$$

where:

k = plate buckling coefficient for uniform normal stress

n = number of equally spaced longitudinal compression flange stiffeners

I_s = moment of inertia of a single longitudinal stiffener about an axis parallel to the flange and taken at the base of the stiffener (in.^{4})

ϕ_b = 0.9 resistance factor for flexure

F_y = specified minimum yield strength of steel (ksi)

E = modulus of elasticity of steel (ksi)

b = width of unstiffened element (in.)

d = overall depth of section (in.)

D = diameter of HSS tube (in.)

t = thickness of unstiffened element, plate thickness, or HSS wall thickness (in.)

h = web depth (in.)

P_u = factored axial load acting on the member (kips)

P_y = nominal axial yield strength of a member (kips)

t_w = thickness of web plate (in.)

7.4.3—Flexural Ductility for Members with Combined Flexural and Axial Load

Except in columns of ductile moment resisting frames as defined in Article 7.5.1, ductility in bending may be used only if axial loads are less than 60 percent of the nominal yield strength of the member. Demand-to-capacity ratios or displacement ductilities shall be kept less than unity if the axial load coinciding with the moment is greater than 60 percent of the nominal yield strength of the member.

7.4.4—Combined Axial and Bending

Members under combined axial and bending interaction shall be checked using interaction equations following the *AASHTO LRFD Bridge Design Specifications*.

7.4.5—Weld Locations

Welds that are located in the expected inelastic region (see Article 4.11.8) of ductile components shall be made complete joint penetration welds. Partial joint penetration groove welds shall not be permitted in the expected inelastic regions. Splices shall not be permitted in the inelastic region of ductile components.

For SDC D, double angles with stitch welds may be used as members of the ductile diaphragm ERS. Members with stitch welds shall follow the design process included in the *AISC LRFD Specifications,* Chapter E (AISC, 2005) on compact and non-compact prismatic members subject to axial compression through the centroidal axis.

7.4.6—Ductile End Diaphragm in Slab-on-Girder Bridges

Ductile end diaphragms in slab-on-girder bridges may be designed to be the ductile energy-dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

- Specially detailed diaphragms, which are capable of dissipating energy in a stable manner without strength degradation, can be used. The diaphragm behavior shall be verified by cyclic testing.
- Only ductile energy-dissipating systems with adequate seismic performance that has been proven through cyclic inelastic testing are used.
- Design considers the combined relative stiffness and strength of end diaphragms and girders (including bearing stiffeners) in establishing the diaphragms' strength and design forces to consider for the capacity-protected elements.
- The response modification factor, R, to be considered in design of the ductile diaphragm is given by:

$$R = \left(\frac{\mu + \frac{K_{DED}}{K_{SUB}}}{1 + \frac{K_{DED}}{K_{SUB}}} \right) \leq 4 \qquad (7.4.6\text{-}1)$$

where:

μ = displacement ductility capacity of the end diaphragm is not to exceed 4

K_{DED} = stiffness of the ductile end diaphragm (kip/in.)

K_{SUB} = stiffness of the substructure (kip/in.)

- All details/connections of the ductile end diaphragms are welded.
- The bridge does not have horizontal wind bracing connecting the bottom flanges of girders, unless the last wind-bracing panel before each support is designed as a ductile panel equivalent and in parallel to its adjacent vertical end diaphragm.
- An effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the slab to the diaphragm.

Special design provisions for a concentrically braced frame (CBF) or an eccentrically braced frame (EBF), following the *ANSI/AISC Seismic Provisions for Structural Steel Buildings 2005*, shall be used in addition to requirements stated in this document.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used and shall not be less than those specified in Article 4.11.2.

For SDC B, C, or D, a single angle bracing may be used for the diagonal member of the end-cross-frame. As this practice is typical and favored for ease of construction, the design process for a single angle bracing shall follow AISC's stand-alone document, *LRFD Design Specification for Single-Angle Members*.

7.4.7—Shear Connectors

Shear connectors should be provided on the flanges of girders, end cross-frames, or diaphragms to transfer seismic loads from the concrete deck to the abutments or pier supports in SDC B and shall be provided in SDCs C and D.

For the transverse seismic load, the effective shear connectors should be taken as those located on the flanges of girders, end cross-frames, or diaphragms that are no farther than $9t_w$ on each side of the outer projecting elements of the bearing stiffener group.

For the longitudinal seismic load, the effective shear connectors should be taken as all those located on the girder flange within the tributary span length of the support.

The seismic load at columns/piers should be taken as the smaller of the following:

- The overstrength shear of the columns/piers, or
- 1.3 times the capacity of the bracing systems if they are considered as ductile seismic resisting systems.

The seismic load at abutments should be taken as the smaller of the following:

- The overstrength shear capacity of the shear keys, or
- 1.3 times the capacity of the bracing systems if they are considered as ductile seismic resisting systems.

Nominal strength of the shear connectors shall be in accordance with Article 6.10.10 of the *AASHTO LRFD Bridge Design Specifications.*

C7.4.7

These provisions are primarily from Caltrans' *Guide Specifications for Seismic Design of Steel Bridges* (Caltrans, 2001). The cross-frames or diaphragms at the end of each span are the main components to transfer the lateral seismic loads from the deck down to the bearing locations. Tests on a 0.4-scale experimental steel girder bridge (60 ft long) conducted by the University of Nevada, Reno (Carden et al., 2001) indicated that too few shear connectors between the girders and deck at the bridge end did not allow the end cross-frame to reach its ultimate capacity. Supporting numerical analysis on a continuous multispan bridge showed that for noncomposite negative moment regions, the absence of shear connectors at the end of a bridge span caused large weak axis bending stresses in the girders likely to cause buckling or yielding of the girders before the capacity of the ductile component was reached. Furthermore, there were large forces in the intermediate cross-frames; therefore, the end cross-frames were no longer the only main components transferring lateral seismic loads from the deck to the bearings. It is, therefore, recommended that adequate shear connectors be provided above supports to transfer seismic lateral loads. These shear connectors can be placed on the girders or the top struts of the end cross-frames or diaphragms.

7.5—DUCTILE MOMENT-RESISTING FRAMES AND SINGLE-COLUMN STRUCTURES FOR SDCS C AND D

This Article applies to ductile moment-resisting frames and bents, constructed with steel I-shape beams and columns connected with their webs in a common plane. For SDC C or D, complying with a Type 1 performance criteria design, the columns shall be designed as ductile structural elements using a force-reduction factor, R, of not greater than 4. The beams, the panel zone at column–beam intersections, and the connections shall be designed as essentially elastic elements.

C7.5

It is believed that properly detailed fully welded column-to-beam or beam-to-column connections in the moment-resisting frames that would typically be used in bridges (see Figure C1) can exhibit highly ductile behavior and perform adequately during earthquakes (contrary to what was observed in buildings following the Northridge earthquake). As a result, strategies to move plastic hinges away from the joints are not required in these Specifications.

However, the Designer may still elect to provide measures (such as haunches at the end of yielding members) to locate plastic hinges some distance away from the welded beam-to-column or column-to-beam joint (SAC, 1995, 1997, 2000).

Although beams, columns, and panel zones can all be designed, detailed, and braced to undergo severe inelastic straining and absorb energy, the detailing requirements of this Section address common bridge structures with deep noncompact beams much stiffer in flexure than their supporting steel columns and favor systems proportioned so that plastic hinges form in the columns. This is consistent with the philosophy adopted for concrete bridges.

Even though some bridges could be configured and designed to develop stable plastic hinging in beams without loss of structural integrity, the large gravity loads that are simultaneously to be resisted by those beams also make plastic hinging at mid-span likely as part of the plastic collapse mechanism. The resulting deformations can damage the superstructure (for example, the diaphragms or deck).

The special case of multitier frames is addressed in Article 7.5.4.

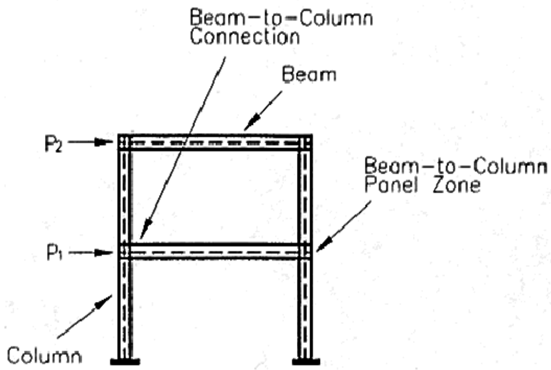


Figure C7.5-1—Example of Moment Frame/Bent

7.5.1—Columns

Width–thickness ratios of compression elements of columns shall be in compliance with Table 7.4.2-1.

Full-penetration flange and web welds shall be specified at column-to-beam (or beam-to-column) connections.

The resistance of columns to combined axial load and flexure shall be determined in accordance with Article 6.9.2.2 of the *AASHTO LRFD Bridge Design Specifications*. The factored axial compression due to seismic load and permanent loads shall not exceed $0.20A_gF_y$.

The shear resistance of the column web shall be determined in accordance with Article 6.10.9 of the *AASHTO LRFD Bridge Design Specifications*.

Except as specified herein, the potential plastic hinge regions (Article 4.11.8), near the top and base of each column, shall be laterally supported and the unsupported distance (i.e., between the plastic hinges) from these locations shall not exceed the value determined from Table 7.4.1-1. The lateral supports shall be provided either directly to the flanges or indirectly through a column web stiffener or a continuity plate, except as specified below. Each column flange lateral support shall resist a force of not less than two percent of the nominal column flange strength ($0.02b_ft_fF_y$) at the support location. The possibility of complete load reversal shall be considered and the potential for plastic hinging about both principal axes of a column shall be considered. The requirements for lateral supports do not apply to potential in-ground plastic hinging zones of pile bents.

C7.5.1

At plastic hinge locations, members absorb energy by undergoing inelastic cyclic bending while maintaining their resistance. Therefore, plastic design rules apply, namely, limitations on width–thickness ratios, web-to-flange weld capacity, web shear resistance, and lateral support.

Axial load in columns is also restricted to avoid early deterioration of beam–column flexural strengths and ductility when subject to high axial loads. Tests by Popov et al. (1975) showed that W-shaped columns subjected to inelastic cyclic loading suffered sudden failure because of excessive local buckling and strength degradation when the maximum axial compressive load exceeded $0.50A_gF_y$. Tests by Schneider et al. (1992) showed that moment-resisting steel frames with hinging columns suffer rapid strength and stiffness deterioration when the columns are subjected to compressive load equal to approximately $0.25A_gF_y$.

Where lateral support cannot be provided, the column maximum slenderness, KL/r, shall not exceed 60 and transverse moments produced by the forces otherwise resisted by the lateral bracing (including the second-order moment due to the resulting column displacement) shall be included in the seismic load combinations.

Splices that incorporate partial joint penetration groove welds shall be located beyond the plastic hinge regions as defined in Article 4.11.8, a minimum distance equal to the greater of:

One-fourth the clear height of column,

Twice the column depth, or

39 in.

The requirements for lateral support are similar to the *AASHTO LRFD Bridge Design Specifications* but are modified to ensure inelastic rotation capacities of at least four times the elastic rotation corresponding to the plastic moment. Consideration of a null moment at one end of the column is assumed and accounts for changes in the location of the contraflexure point of the column moment diagram during earthquake response. Figure 10.27 in Bruneau et al. (1997) could be used to develop other unsupported lengths limits.

Built-up columns made of fastened components (e.g., bolted or riveted) are beyond the scope of these Guide Specifications.

7.5.2—Beams

The factored resistance of the beams shall be determined in accordance with Article 6.12 of the *AASHTO LRFD Bridge Design Specifications*. At a joint between beams and columns, the sum of the factored resistances of the beams shall not be less than the sum of the probable resistances of the column(s) framing into the joint. Unless otherwise demonstrated by rational analysis, the probable flexural resistance of columns, M_{nx}, shall be taken as the product of the overstrength factor times the columns' nominal flexural resistance determined either in accordance with Article 6.9.2.2 of the *AASHTO LRFD Bridge Design Specifications* or by:

$$M_{nx} = 1.18 M_{px}\left(1 - \frac{P_u}{AF_{ye}}\right) \leq M_{px} \quad (7.5.2\text{-}1)$$

where:

M_{px} = plastic moment capacity of the column member based on expected material properties (kip-ft)

A = cross-sectional area of member (in.^{2})

F_{ye} = expected yield stress of structural steel member (ksi)

P_u = factored axial load acting on member (kips)

C7.5.2

Because plastic hinges are not expected to form in beams, beams need not conform to plastic design requirements.

The requirement for beam resistance is consistent with the outlined capacity design philosophy. The beams should be capacity protected. In the extreme load situation, the capacity-protected beams are required to have nominal resistances of not less than the combined effects corresponding to the plastic hinges in the columns attaining their probable capacity and the probable companion permanent load acting directly on the beams. The columns' probable capacity should account for the overstrength due to higher yield than specified yield and strain-hardening effects. The value specified in Article 6.9.2.2 of the *AASHTO LRFD Bridge Design Specifications*, used in conjunction with the resistance factor φ_f for steel beams in flexure of 1.00, is compatible with the AISC (2005) 1.1, R_y, used with a resistance factor φ of 0.9 (here R_y is embedded in F_{ye}).

Eq. 1 was developed for I-sections about strong axis bending and may not be appropriate for other sections.

7.5.3—Panel Zones and Connections

Column–beam intersection panel zones, moment-resisting connections, and column–base connections shall be designed as essentially elastic elements.

Panel zones shall be designed such that the vertical shearing resistance is determined in accordance with Article 6.10.9 of the *AASHTO LRFD Bridge Design Specifications*.

Beam-to-column connections shall have resistance not less than the resistance of the beam stipulated in Article 7.5.2.

C7.5.3

The panel zone should be capacity protected.

Column–base connections should also be capacity protected, unless they are designed and detailed to dissipate energy.

Panel zone yielding is not permitted.

There is a concern that doubler plates in panel zones can be an undesirable fatigue detail. For plate–girder sections, it is preferable to specify a thicker web plate, if necessary, rather than use panel zone doubler plates.

Continuity plates shall be provided on both sides of the panel zone web and shall finish with total width of at least 0.8 times the flange width of the opposing flanges. The continuity plates shall be proportioned to meet the stiffener requirements stipulated in Article 6.10.11.2 of the *AASHTO LRFD Bridge Design Specifications* and shall be connected to both flanges and the web.

Flanges and connection plates in bolted connections shall be designed in accordance with Article 6.8 of the *AASHTO LRFD Bridge Design Specifications,* but in no case shall the factored nominal tensile resistance for fracture on the net section be less than the factored nominal tensile resistance for yielding in the gross section.

The intent of these provisions is to ensure that yielding of the gross section occurs prior to fracture on the net section.

7.5.4—Multitier Frame Bents

For multitier frame bents, capacity design principles, as well as the requirements of Articles 7.5.1, 7.5.2, and 7.5.3, may be modified by the Designer to achieve column plastic hinging only at the top of the column. Column plastic hinging at the base where fixity to the foundation is needed shall be assessed where applicable.

C7.5.4

Multitier frame bents are sometimes used because they are more rigid transversely than single-tier frame bents. In such multitier bents, the intermediate beams are significantly smaller than the top beam as they are not supporting the gravity loads from the superstructure.

As a result, in a multitier frame, plastic hinging in the beams may be unavoidable in all but the top beam. Trying to ensure strong-beam–weak-column design at all joints in multitier bents may have the undesirable effect of concentrating all column plastic hinging in one tier, with greater local ductility demands than otherwise expected in design.

Using capacity design principles, the equations and intent of Article 7.5.1 and Article 7.5.2 may be modified by the Designer to achieve column plastic hinging only at the top and base of the column and plastic hinging at the ends of all intermediate beams, as shown in Figure C1.

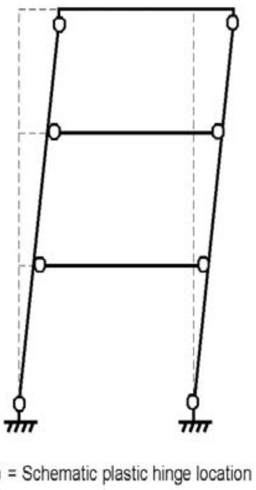


Figure C7.5.4-1—Acceptable Plastic Mechanism for Multitier Bent

7.6—CONCRETE-FILLED STEEL PIPES FOR SDCS C AND D

Concrete-filled steel pipes used as columns, piers, or piles expected to develop full plastic hinging of the composite section as a result of seismic response shall be designed in accordance with Articles 6.9.2.2, 6.9.5, and 6.12.3.2.2 of the *AASHTO LRFD Bridge Design Specifications*, as well as the requirements in this Article.

C7.6

This Article is only applicable to concrete-filled steel pipes without internal reinforcement and connected in a way that allows development of their full composite strength. It is not applicable to design a concrete-filled steel pipe that relies on internal reinforcement to provide continuity with another structural element or for which the steel pipe is not continuous or connected in a way that enables it to develop its full yield strength. When used in pile bent, the full composite strength of the plastic hinge located below ground can be developed only if it can be ensured that the concrete fill is present at that location.

Research (e.g., Alfawahkiri, 1997; Bruneau and Marson, 1999) has demonstrated that the AASHTO equations for the design of concrete-filled steel pipes in combined axial compression and flexure (Articles 6.9.2.2, 6.9.5, and 6.12.2.3.2 of the *AASHTO LRFD Bridge Design Specifications*) provide a conservative assessment of beam–column strength. Consequently, the calculated strength of concrete-filled steel pipes that could be used as columns in ductile moment-resisting frames or pile bents could be significantly underestimated. This is not surprising given that these equations together are deemed applicable to a broad range of composite member types and shapes, including concrete-encased steel shapes.

Although these equations may be perceived as conservative from a nonseismic perspective, an equation that more realistically captures the plastic moment of such columns is essential for capacity design. Capacity-protected elements should be designed with adequate strength to elastically withstand the plastic hinging in the columns. Underestimating the plastic hinging force translates into underdesign of the capacity-protected elements. A column unknowingly stronger than expected will not hinge prior to damaging foundations or other undesirable locations in the structure. This can have severe consequences, as the capacity-protected elements are not detailed to withstand large inelastic deformations. The provisions of Article 7.3 are added to prevent this behavior.

For analysis, the flexural stiffness of the composite concrete-filled pipe section may be taken as given in Eq. C1, which is a modified form of that given in Article 5.7.4.3 of the *AASHTO LRFD Bridge Design Specifications*:

$$(EI)_{eff} = E_s I_s + \frac{E_c I_c}{2.5} \tag{C7.6-1}$$

where:

I_c = moment of inertia of the concrete core (in.^{4})

I_s = moment of inertia of the steel pipe (in.^{4})

E_s = modulus of elasticity of steel (ksi)

E_c = modulus of elasticity of concrete (ksi)

Alternatively, the flexural stiffness of the composite concrete-filled pipe section may be taken as given in Eq. C2, which is a modified form of that given in Article 6.9.5.1 of the *AASHTO LRFD Bridge Design Specifications*:

$$(EI)_{eff} = E_s I_s \left(0.88 + \frac{0.352}{n}\frac{A_c}{A_s}\right) \geq E_s I_s \qquad \text{(C7.6-2)}$$

where:

A_c = area of the concrete core (in.^{2})

A_s = area of the steel pipe (in.^{2})

I_s = moment of inertia of the steel pipe (in.^{4})

n = modular ratio

7.6.1—Combined Axial Compression and Flexure

Concrete-filled steel pipe members required to resist both axial compression and flexure and intended to be ductile substructure elements shall be proportioned such that:

$$\frac{P_u}{P_r} + \frac{BM_u}{M_{rc}} \leq 1.0 \qquad \text{(7.6.1-1)}$$

and:

$$\frac{M_u}{M_{rc}} \leq 1.0 \qquad \text{(7.6.1-2)}$$

in which:

$$B = \frac{P_{ro}}{P_{rc}} - 1 \qquad \text{(7.6.1-3)}$$

$$P_{rc} = \phi_c A_c f'_c \qquad \text{(7.6.1-4)}$$

where:

P_r = factored nominal axial capacity of the member determined in accordance with Article 6.9.5 of the *AASHTO LRFD Bridge Design Specifications* (kips)

M_{rc} = factored nominal moment capacity of the member determined in accordance with Article 7.6.2 (kip-ft)

M_u = factored moment demand acting on the member, including the elastic seismic demand divided by the appropriate force reduction factor, R (kip-ft)

C7.6.1

The interaction equation is known to be reliable up to a maximum slenderness limit $D/t < 0.96E/F_y$, underestimating the flexural moment capacity by 1.25 on average (see Figure C1). It may significantly overestimate column strength having greater D/t ratios.

The interaction equation is only applicable to concrete-filled steel pipes. Revised equations may also be needed to replace those of Article 6.9.2.2 of the *AASHTO LRFD Bridge Design Specifications* for other types of composite columns (such as concrete-encased columns).

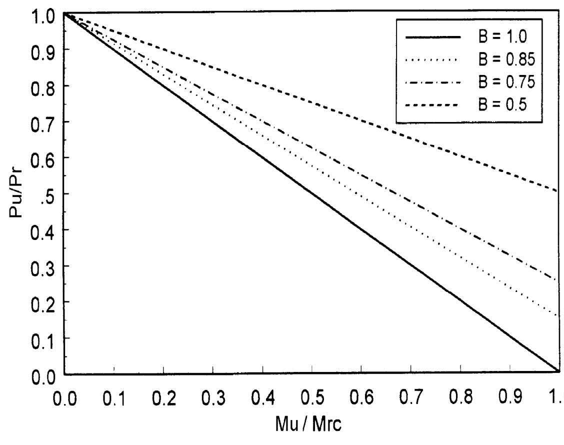


Figure C7.6.1-1—Interaction Curves for Concrete-Filled Pipe

P_{ro} = factored nominal axial capacity of the member calculated determined in accordance with Article 6.9.5 of the *AASHTO LRFD Bridge Design Specifications* using $\lambda = 0$ (kips)

ϕ_c = 0.75 resistance factor for concrete in compression

A_c = area of the concrete core (in.^{2})

f'_c = nominal uniaxial concrete compressive strength (ksi)

7.6.2—Flexural Strength

The factored moment resistance of a concrete-filled steel pipe may be calculated using a strain compatibility approach that uses appropriate constitutive material models. In lieu of a strain compatibility approach, the factored moment resistance of a concrete-filled steel pipe may be calculated using one of the following two methods:

Method 1—Exact Geometry:

$$M_{rc} = \varphi_f \left(C_r e + C'_r e' \right) \tag{7.6.2-1}$$

in which:

$$C_r = F_y \beta \frac{Dt}{2} \tag{7.6.2-2}$$

$$C'_r = f'_c \left[\frac{\beta D^2}{8} - \frac{b_c}{2}\left(\frac{D}{2} - a \right) \right] \tag{7.6.2-3}$$

$$e = b_c \left(\frac{1}{(2\pi - \beta)} + \frac{1}{\beta} \right) \tag{7.6.2-4}$$

$$e' = b_c \left(\frac{1}{(2\pi - \beta)} + \frac{b_c^2}{1.5\beta D^2 - 6 b_c (0.5D - a)} \right) \tag{7.6.2-5}$$

$$a = \frac{b_c}{2} \tan\left(\frac{\beta}{4} \right) \tag{7.6.2-6}$$

$$b_c = D \sin\left(\frac{\beta}{2} \right) \tag{7.6.2-7}$$

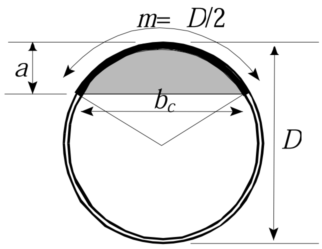
β = central angle formed between neutral axis chord line and the center point of the pipe found by the recursive equation (rad.)

$$\beta = \frac{A_s F_y + 0.25 D^2 f'_c \left[\sin\left(\frac{\beta}{2}\right) - \sin^2\left(\frac{\beta}{2}\right) \tan\left(\frac{\beta}{4}\right) \right]}{0.125 D^2 f'_c + D t F_y} \tag{7.6.2-8}$$

C7.6.2

When using the approximate equations to calculate the forces acting on capacity-protected members as a result of plastic hinging of the concrete-filled pipes, F_{ye} should replace F_y for consistency with the capacity design philosophy.

Figure C1 illustrates the geometric parameters used in this Article.



Note: Shaded area is concrete in compression above the neutral axis

Figure C7.6.2-1—Flexure of Concrete-Filled Pipe

Moment resistance is calculated assuming the concrete in compression at f'_c and the steel in tension and compression at F_y. The resulting free-body diagram is shown in Figure C2, where e is equal to $y_{sc}+y_{st}$, e' is equal to y_c+y_{st}, and y_c is the distance of the concrete compressive force (C'_r) from the center of gravity, and y_{st} and y_{sc} are the respective distances of the steel tensile (T_r) and compressive forces (C_r) from the center of gravity.

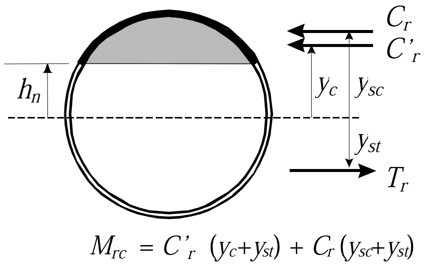


Figure C7.6.2-2—Free-Body Diagram Used to Calculate Moment Resistance of Concrete-Filled Pipe

where:

D = outside diameter of steel pipe (in.)

t = pipe wall thickness (in.)

F_y = nominal yield stress of steel pipe (ksi)

f'_c = nominal uniaxial concrete compressive strength (ksi)

Method 2—Approximate Geometry:

A conservative value of M_{rc} is given by:

$$M_{rc} = \varphi_f \left[(Z - 2th_n^2) F_y + \left(\frac{2}{3}(0.5D - t)^3 - (0.5D - t) h_n^2 \right) f'_c \right] \quad (7.6.2\text{-}9)$$

in which:

$$h_n = \frac{A_c f'_c}{2Df'_c + 4t(2F_y - f'_c)} \quad (7.6.2\text{-}10)$$

where:

ϕ_f = 1.0 resistance factor for structural steel in flexure

A_c = area of the concrete core (in.^{2})

D = outside diameter of steel pipe (in.)

t = pipe wall thickness (in.)

Z = plastic section modulus of steel pipe (in.^{3})

F_y = nominal yield stress of steel pipe (ksi)

f'_c = nominal concrete compressive strength (ksi)

For capacity design purposes, the moment calculated by this approximate method shall be increased according to Article 7.3.

In Method 2, a geometric approximation is made in calculating the area of concrete in compression by subtracting the rectangular shaded area shown in Figure C3 from the total area enclosed by the pipe (and dividing the result by 2). Neutral axis is at height h_n.

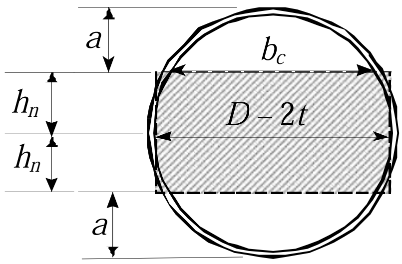


Figure C7.6.2-3 Flexure of Concrete-Filled Pipe—Illustrates Approximation Made in Method 2

Method 2 (using approximate geometry) gives smaller moment capacities than Method 1 (exact geometry). The requirement to increase the calculated moment by ten percent for capacity design when using the approximate method was established from the ratio of the moment calculated by both methods for a D/t of 10. The moment ratio decreases as D/t increases.

7.6.3—Beams and Connections

Capacity-protected members shall be designed to resist the forces resulting from hinging in the concrete-filled pipes calculated according to Article 7.6.2.

C7.6.3

Experimental work by Bruneau and Marson (1999), Shama et al. (2001), and Azizinamini et al. (1999) provides examples of full fixity connection details. In some instances, full fixity may not be needed at both ends of columns. Concrete-filled steel pipes, when used in pile bents, only require full moment connection at the pile cap.

Alternative design details for connecting concrete-filled steel pipes to concrete members have been developed by Priestley et al. (1996).

7.7—CONNECTIONS FOR SDCS C AND D

7.7.1—Minimum Strength for Connections to Ductile Members

Connections and splices between or within members having a ductility demand greater than 1 shall be designed to have a nominal capacity at least ten percent greater than the nominal capacity of the weaker connected member based on expected material properties.

C7.7.1

Special consideration may be required for slip-critical connections that are subjected to cyclic loading. Some researchers have expressed concern that the Poisson effect may cause a reduction in plate thickness when yielding on a component's net section occurs during seismic response, which may translate to a reduced clamping action on the faying surfaces after the earthquake. This has not been experimentally observed, nor noted in post-earthquake inspections, but the impact of such a phenomenon would be to reduce the slip-resistance of the connection, which may have an impact on fatigue resistance. This impact is believed to be negligible for a Category C detail for finite life, and a Category D detail for infinite life. Design to prevent slip for the design earthquake should be also considered.

7.7.2—Yielding of Gross Section for Connections to Ductile Members

Yielding of the gross section shall be checked (see Article 7.7.6). Fracture in the net section and the block shear rupture failure shall be prevented.

7.7.3—Welded Connections

Partial joint penetration welds or fillet welds in regions of members subject to inelastic deformations shall not be used. Outside of the inelastic regions, partial joint penetration welds shall provide at least 150 percent of the strength required by calculation and not less than 75 percent of the strength of the connected parts regardless of the action of the weld.

7.7.4—Gusset Plate Strength

Gusset plates shall be designed to resist shear, flexure, and axial forces generated by overstrength capacities of connected ductile members and force demands of connected essentially elastic members. The design strength shall be based on the effective width in accordance with Whitmore's method.

C7.7.4

The Whitmore (1952) effective width is defined as the distance between two lines radiating outward at 30° angles from the first row of bolts of the gusset plate along a line running through the last row of bolts, as shown in Figure C1.

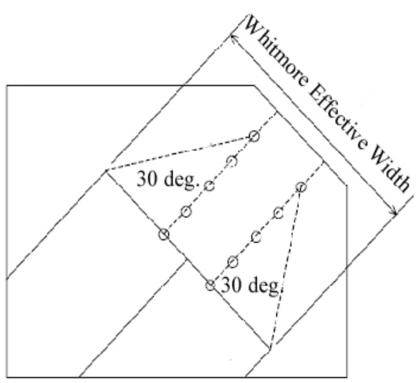


Figure C7.7.4-1—Whitmore Effective Width

7.7.5—Limiting Unsupported Edge Length-to-Thickness Ratio for a Gusset Plate

The unsupported edge length-to-thickness ratio of a gusset plate shall satisfy:

$$\frac{L_g}{t} \leq 2.06\sqrt{\frac{E}{F_y}} \qquad (7.7.5\text{-}1)$$

where:

L_g = unsupported edge length of the gusset plate (in.)

t = thickness of gusset plate (in.)

E = modulus of elasticity of steel (ksi)

F_y = nominal yield stress of steel pipe (ksi)

7.7.6—Gusset Plate Tension Strength

The only acceptable failure mode of gusset plates is yielding on the gross section that will ensure a ductile failure mode. The factored tension strength of a gusset plate, ϕP_n, shall be taken as:

$$\phi P_{ng} = \phi_y A_g F_y \leq \begin{cases} \phi_u A_n F_u \\ \\ \phi_{bs} P_{bs} \end{cases} \qquad (7.7.6\text{-}1)$$

in which:

if $A_{tn} \geq 0.58A_{vn}$, then:

$$P_{bs} = 0.58 F_y A_{vg} + F_u A_{tn} \qquad (7.7.6\text{-}2)$$

if $A_{tn} < 0.58A_{vn}$, then:

$$P_{bs} = 0.58 F_u A_{vn} + F_y A_{tg} \qquad (7.7.6\text{-}3)$$

where:

A_{vg} = gross area of section along the plane resisting shear in block shear failure mode (in.^{2})

A_{vn} = net area of section along the plane resisting shear in block shear failure mode (in.^{2})

A_{tg} = gross area of section along the plane resisting tension in block shear failure mode (in.^{2})

A_{tn} = net area of section along the plane resisting tension in block shear failure mode (in.^{2})

A_g = gross area of section along the plane resisting tension (in.^{2})

A_n = net area of section along the plane resisting tension (in.^{2})

C7.7.6

These provisions are similar to those found in Article 6.13 of the *AASHTO LRFD Bridge Design Specifications* but have been modified for seismic design considerations.

Note that the minimum block shear failure mode may be one of several failure modes. Investigation of all potential block shear failure patterns is required to determine the limiting resistance, P_{bs}.

The intent of these provisions is to ensure that yielding of the gross section occurs prior to fracture on the net section and block shear failure (Caltrans, 2001).

F_y = nominal yield stress of steel (ksi)

F_u = minimum tensile strength of steel (ksi)

ϕ_{bs} = 0.80 resistance factor for block shear failure mechanisms

ϕ_u = 0.80 resistance factor for fracture on net section

ϕ_y = 0.95 resistance factor for yield on gross section

7.7.7—Compression Strength of a Gusset Plate

The nominal compression strength of the gusset plates, P_{ng}, shall be calculated according to the *AASHTO LRFD Bridge Design Specifications*.

7.7.8—In-Plane Moment (Strong Axis)

The nominal yield moment strength of a gusset plate, M_{ng}, shall be taken as:

$$M_{ng} = S_g F_y \tag{7.7.8-1}$$

where:

S_g = elastic section modulus of gusset plate about the strong axis (in.^{3})

F_y = nominal yield stress of steel gusset plate (ksi)

The nominal plastic moment strength of a gusset plate, M_{pg}, shall be taken as:

$$M_{pg} = Z_g F_y \tag{7.7.8-2}$$

where:

Z_g = plastic section modulus of gusset plate about the strong axis (in.^{3})

7.7.9—In-Plane Shear Strength

The nominal shear strength of a gusset plate, V_{ng}, shall be taken as:

$$V_{ng} = 0.58 A_{gg} F_y \tag{7.7.9-1}$$

where:

A_{gg} = gross area of gusset plate (in.^{2})

F_y = nominal yield stress of steel gusset plate (ksi)

7.7.10—Combined Moment, Shear, and Axial Forces

The initial yielding strength of a gusset plate subjected to a combination of in-plane moment, shear, and axial force shall be determined by the following equations:

$$\frac{P_g}{P_{rg}} + \frac{M_g}{M_{rg}} \leq 1.0 \qquad (7.7.10\text{-}1)$$

and:

$$\frac{P_g}{P_{rg}} + \left(\frac{V_g}{V_{rg}}\right)^2 \leq 1.0 \qquad (7.7.10\text{-}2)$$

where:

V_g = shear force acting on the gusset plate (kips)

M_g = moment acting on the gusset plate (kip-in.)

P_g = axial force acting on the gusset plate (kips)

M_{rg} = factored nominal yield moment capacity, ϕM_{ng}, of the gusset plate from Article 7.7.8 (kip-in.)

V_{rg} = factored nominal shear capacity, ϕV_{ng}, of the gusset plate from Article 7.7.9 (kips)

P_{rg} = factored nominal yield axial capacity, ϕP_{ng}, of the gusset plate from Article 7.7.6 (kips)

Full yielding of shear–moment–axial load interaction for a plate shall satisfy:

$$\frac{M_g}{M_{rpg}} + \left(\frac{P_g}{P_{rg}}\right)^2 + \frac{\left(\frac{V_g}{V_{rg}}\right)^4}{\left[1-\left(\frac{P_g}{P_{rg}}\right)^2\right]} \leq 1.0 \qquad (7.7.10\text{-}3)$$

where:

M_{rpg}= factored nominal plastic moment capacity, ϕM_{pg}, of the gusset plate from Article 7.7.8 (kip-in.)

7.7.11—Fastener Capacity

Fastener capacity and other related design requirements shall be determined in accordance with Article 6.13 of the *AASHTO LRFD Bridge Design Specifications.*

7.8—ISOLATION DEVICES

Design and detailing of seismic isolation devices shall be designed in accordance with the provisions of the *AASHTO Guide Specifications for Seismic Isolation Design*.

C7.8

The requirements for analysis of bridges with seismic isolation 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 7.3.

Bridges that have elastomeric or sliding bearings at each pier may be designed as isolated structures.

7.9—FIXED AND EXPANSION BEARINGS

7.9.1—Applicability

The provisions shall be taken to apply to pin bearings, roller bearings, rocker bearings, bronze or copper-alloy sliding bearings, elastomeric bearings, spherical bearings, pot bearings, and disc bearings in common slab-on-steel girder bridges. Bearings for curved bridges, seismic isolation-type bearings, and structural fuse bearings are not addressed in these Guide Specifications.

C7.9.1

Bearings are important elements of the overall ERS of a bridge structure. The 1995 Kobe earthquake, and others that preceded it or have occurred since, clearly showed poor performance of some bearing types and the disastrous consequences that a bearing failure can have on the overall performance of a bridge. A consensus was developed that some testing of bearings would be desirable provided a Designer had the option of providing restraints or permitting the bearing to fail if an adequate surface for subsequent movement is provided. An example occurred in Kobe where a bearing failed. The steel diaphragm and steel girder were subsequently damaged because the girder became jammed on the failed bearing and could not move.

There have been a number of studies performed in which girders slide either on specially designed bearings or concrete surfaces. A good summary of the range of the results that can be anticipated from these types of analyses can be found in Dicleli and Bruneau (1995).

7.9.2—Design Criteria

The selection of seismic design of bearings shall be related to the strength and stiffness characteristics of both the superstructure and the substructure.

Bearing design shall be consistent with the intended seismic design strategy and the response of the whole bridge system.

Rigid-type bearings are assumed not to move in restrained directions; therefore, the seismic forces from the superstructure shall be assumed to be transmitted through diaphragms or cross-frames and their connections to the bearings and then to the substructure without reduction due to local inelastic action along that load path.

Deformable-type bearings having less than full rigidity in the restrained directions but not specifically designed as base isolators or fuses have demonstrated a reduction in force transmission and may be used in seismic applications. The reduced force transmitted through the bearing shall not be less than 0.4 times the bearing dead load reaction.

7.9.3—Design and Detail Requirements

The Designer should consider the impact on the lateral load path due to unequal participation of bearings considering connection tolerances, unintended misalignments, the capacity of individual bearings, and skew effects.

Roller bearings or rocker bearings shall not be used in new bridge construction. Expansion bearings and their supports shall be designed in such a manner that the structure can undergo movements in the unrestrained direction not less than the seismic displacements determined from analysis without collapse. Adequate support length shall also be provided for fixed bearings.

In their restrained directions, bearings shall be designed and detailed to engage at essentially the same movement in each direction.

The frictional resistance of the bearing interface sliding surfaces shall be neglected when it contributes to resisting seismic loads. Conversely, the frictional resistance shall be conservatively calculated (i.e., overestimated) when the friction resistance results in the application of greater force effects to the structural components.

Elastomeric expansion bearings shall be provided with anchorage to adequately resist the seismically induced horizontal forces in excess of those accommodated by shear in the pad. The sole plate and base plate shall be made wider to accommodate the anchor bolts. Inserts through the elastomer shall not be allowed. The anchor bolts shall be designed for the combined effect of bending and shear for seismic loads. Elastomeric fixed bearings shall be provided with horizontal restraint adequate for the full horizontal load.

Spherical bearings shall be evaluated for component and connection strength and bearing stability.

Pot and disc bearings should not be used for seismic applications where significant vertical acceleration is present. Where the use of pot and disc bearings is unavoidable, they shall be provided with an independent seismically resistant anchorage system.

C7.9.3

The types of tests that are required by these Guide Specifications are similar to but significantly less extensive than those required for seismically isolated bridges. Each manufacturer is required to conduct a prototype qualification test to qualify a particular bearing type and size for its design forces or displacements. This series of tests needs to be performed only once to qualify the bearing type and size, whereas for seismically isolated bridges, prototype tests are required on every project. The quality control tests required on 1 of every 10 bearings is the same as that required for every isolator on seismic isolation bridge projects. The cost of the much more extensive prototype and quality control testing of isolation bearings is approximately 10–15 percent of the total bearing cost, which is on the order of two percent of the total bridge cost. The testing proposed herein is much less stringent than that required for isolation bearings and is expected to be less than 0.1 percent of the total bridge cost. However, the benefits of testing are considered to be significant because owners would have a much higher degree of confidence that each new bearing will perform as designed during an earthquake. The testing capability exists to do these tests on full-size bearings. The Owner has the final determination on the extent of the testing requirements as deemed appropriate for the type of bridge considered.

7.9.4—Bearing Anchorage

Sufficient reinforcement shall be provided around the anchor bolts to develop the horizontal forces and anchor them into the mass of the substructure unit. Potential concrete crack surfaces next to the bearing anchorage shall have sufficient shear friction capacity to prevent failure.

SECTION 8: REINFORCED CONCRETE COMPONENTS

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SECTION 8

REINFORCED CONCRETE COMPONENTS

8.1—GENERAL

Design and construction of concrete components that include superstructures, columns, piers, footings, and their connections shall conform to the requirements of this Section.

For the purpose of this Article, a vertical support shall be considered to be a column if the ratio of the clear height to the maximum plan dimensions of the support is greater than 2.5. For a flared column, the maximum plan dimension shall be taken at the minimum section of the flare. For supports with a ratio less than 2.5, the provisions for piers of Articles 8.6.8 to 8.6.10 shall apply.

A pier shall be designed as a pier member in its strong direction and a column in its weak direction.

The pile extensions of pile bents as well as drilled shafts and caissons shall be regarded as columns for design and detailing purposes.

If architectural flares or other treatments are provided to columns adjacent to potential plastic hinge zones, they shall be "structurally isolated" in such a way that they do not add to the flexural strength capacity of the columns. If "structural isolation" is not used, then the column and adjacent structural elements shall be designed to resist the forces generated by increased flexural strength capacity according to Article 8.14.

C8.1

The 1989 Loma Prieta and 1994 Northridge earthquakes confirmed the vulnerability of columns with inadequate transverse reinforcement and inadequate anchorage of longitudinal reinforcement. Also of concern are:

- Lack of adequate reinforcement for positive moments that may occur in the superstructure over monolithic supports when the structure is subjected to longitudinal dynamic loads;
- Lack of adequate shear strength in joints between columns and bent caps under transverse dynamic loads;
- Inadequate reinforcement for torsion, particularly in outrigger-type bent caps; and
- Inadequate transverse reinforcement for shear and for restraint against global buckling of longitudinal bars ("bird caging").

The purpose of the design is to ensure that a column is provided with adequate ductility and is forced to yield in flexure and that the potential for a shear, compression failure due to longitudinal bar buckling or loss of anchorage mode of failure is minimized.

The actual ductility demand on a column or pier is a complex function of a number of variables, including:

- Earthquake characteristics, including duration, frequency content, and near-field (or pulse) effects;
- Design force level;
- Periods of vibration of the bridge;
- Shape of the inelastic hysteresis loop of the columns, and hence effective hysteretic damping;
- Elastic damping coefficient;
- Contributions of foundation and soil conditions to structural flexibility; and
- Spread of plasticity (plastic hinge length) in the column.

The damage potential of a column is also related to the ratio of the duration of strong ground shaking to the natural period of vibration of the bridge.

The definition of a column in this Article is provided as a guideline to differentiate between the additional design requirements for a wall-type pier and the requirements for a column.

Certain oversize columns exist for architectural or aesthetic reasons. These columns, if fully reinforced, place excessive demands of moment, shear, or both on adjoining elements. The Designer should strive to "isolate structurally" those architectural elements that do not form part of the primary energy-dissipation system that are located either within or in close proximity to plastic hinge zones. Nevertheless, the architectural elements should remain serviceable throughout the life of the structure. For this reason, minimum steel for temperature and shrinkage should be provided. When architectural flares are not isolated, Article 8.14.2 requires that the design shear force for a flared column be the worst case calculated using the overstrength moment of the oversized flare or the shear generated by a plastic hinge at the bottom of the flare.

8.2—SEISMIC DESIGN CATEGORY (SDC) A

The provisions of Article 4.6 for force demand shall be satisfied. The provisions of Article 4.12 for support length shall be satisfied.

When S_{D1} is greater than or equal to 0.10 but less than 0.15, minimum shear reinforcement shall be provided according to the requirements of Article 8.6.5 for SDC B. When such transverse reinforcement is provided, the provisions of Article 8.8.9 should apply. The length over which this reinforcement shall extend shall be the plastic hinge region defined in Article 4.11.7. Alternately, this length may be that defined in Article 5.10.11.4.1e of the *AASHTO LRFD Bridge Design Specifications*.

8.3—SEISMIC DESIGN CATEGORIES B, C, AND D

8.3.1—General

Initial sizing of columns should be performed using strength and service load combinations defined in the *AASHTO LRFD Bridge Design Specifications.*

C8.3.1

For post-tensioned box girders, it is recommended that the least dimension of column or pier wall be less than or equal to the superstructure depth. See Article 8.10.

8.3.2—Force Demands for SDC B

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls. Force demands shall be less than capacities established in Articles 8.5 and 8.6.

C8.3.2

SDC B structures are designed and detailed to achieve a displacement ductility, μ_D, of at least 2.

8.3.3—Force Demands for SDCs C and D

The design forces shall be based on forces resulting from the overstrength plastic hinging moment capacity or the maximum connection capacity following the capacity design principles specified in Article 4.11.

8.3.4—Local Ductility Demands for SDC D

The local displacement ductility demands, μ_D, of members shall be determined on the basis of the analysis method adopted in Section 5. The local displacement ductility demand shall not exceed the maximum allowable displacement ductilities established in Article 4.9.

8.4—PROPERTIES AND APPLICATIONS OF REINFORCING STEEL, PRESTRESSING STEEL, AND CONCRETE FOR SDCS B, C, AND D

For SDCs B and C, the expected material properties shall be used to determine the section stiffness and overstrength capacities.

For SDC D, the expected material properties shall be used to determine section stiffness, overstrength capacities, and displacement capacities.

8.4.1—Reinforcing Steel

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric, and welded deformed wire fabric shall conform to the material standards as specified in the *AASHTO LRFD Bridge Design Specifications.*

Use of high-strength, high-alloy bars with an ultimate tensile strength of up to 250 ksi shall be permitted for longitudinal column reinforcement for seismic loading provided that it can be demonstrated through testing that the low-cycle fatigue properties are not inferior to normal reinforcing steels with yield strengths of 75 ksi or less.

Use of wire rope or strand shall be permitted for spirals in columns if it can be shown through testing that the modulus of toughness exceeds 14 ksi.

For SDCs B and C, use of ASTM A 706 or ASTM A 615 Grade 60 reinforcing steel shall be permitted.

For SDC D, ASTM A 706 reinforcing steel in members where plastic hinging is expected shall be used.

C8.4.1

High-strength reinforcement reduces congestion and cost, as demonstrated by Mander and Cheng (1999). However, it is important to ensure that the cyclic fatigue life is not inferior when compared with ordinary mild steel reinforcing bars. Mander, Panthaki, and Kasalanati, (1994) have shown that modern high-alloy prestressing threadbar steels can have sufficient ductility to justify their use in seismic design.

The modulus of toughness is defined as the area beneath the monotonic tensile stress–strain curve from initial loading (zero stress) to fracture.

8.4.2—Reinforcing Steel Modeling

Reinforcing steel shall be modeled with a stress–strain relationship that exhibits an initial elastic portion, a yield plateau, and a strain-hardening range in which the stress increases with strain, as shown in Figure 1. In lieu of specific data, the steel reinforcement properties provided in Table 1 should be used.

Within the elastic region, the modulus of elasticity, E_s, shall be taken as 29 000 ksi.

C8.4.2

The steel reinforcement properties provided in Table 1 are based on data collected by Caltrans.

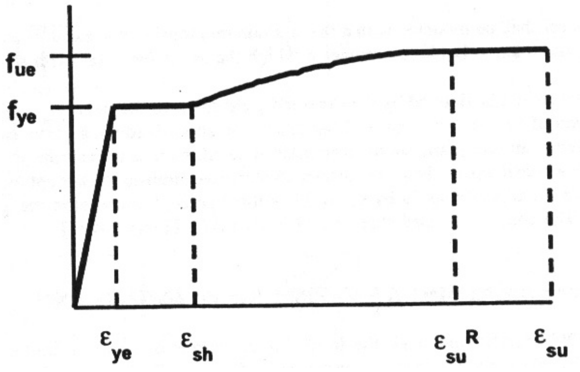

Figure 8.4.2-1—Reinforcing Steel Stress–Strain Model

Table 8.4.2-1—Stress Properties of Reinforcing Steel Bars

Property	Notation	Bar Size	ASTM A 706	ASTM A 615 Grade 60
Specified minimum yield stress (ksi)	f_y	#3– #18	60	60
Expected yield stress (ksi)	f_{ye}	#3– #18	68	68
Expected tensile strength (ksi)	f_{ue}	#3– #18	95	95
Expected yield strain	ε_{ye}	#3– #18	0.0023	0.0023
Onset of strain hardening	ε_{sh}	#3– #8	0.0150	0.0150
		#9	0.0125	0.0125
		#10 & #11	0.0115	0.0115
		#14	0.0075	0.0075
		#18	0.0050	0.0050
Reduced ultimate tensile strain	ε_{su}^{R}	#4– #10	0.090	0.060
		#11– #18	0.060	0.040
Ultimate tensile strain	ε_{su}	#4– #10	0.120	0.090
		#11– #18	0.090	0.060

8.4.3—Prestressing Steel Modeling

Prestressing steel shall be modeled with an idealized nonlinear stress–strain model. Figure 1 shows an idealized stress–strain model for a 7-wire low-relaxation prestressing strand.

Essentially elastic prestress steel strain, $\varepsilon_{ps,EE}$, shall be taken as the following:

For 250-ksi strands:

$$\varepsilon_{ps,\mathrm{EE}} = 0.0076$$

For 270-ksi strands:

$$\varepsilon_{ps,\mathrm{EE}} = 0.0086$$

Reduced ultimate prestress steel strain shall be taken as the following:

$$\varepsilon^{R}_{ps,u} = 0.03$$

The stress, f_{ps}, in the prestressing steel shall be taken as the following:

For 250-ksi strands:

$$f_{ps} = 28{,}500\varepsilon_{ps} \text{ when } \varepsilon_{ps} \leq 0.0076 \quad (8.4.3\text{-}1)$$

$$f_{ps} = 250 - \frac{0.25}{\varepsilon_{ps}} \text{ when } \varepsilon_{ps} > 0.0076 \quad (8.4.3\text{-}2)$$

For 270-ksi strands:

$$f_{ps} = 28{,}500\varepsilon_{ps} \text{ when } \varepsilon_{ps} \leq 0.0086 \quad (8.4.3\text{-}3)$$

$$f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \text{ when } \varepsilon_{ps} > 0.0086 \quad (8.4.3\text{-}4)$$

where:

ε_{ps} = strain in prestressing steel

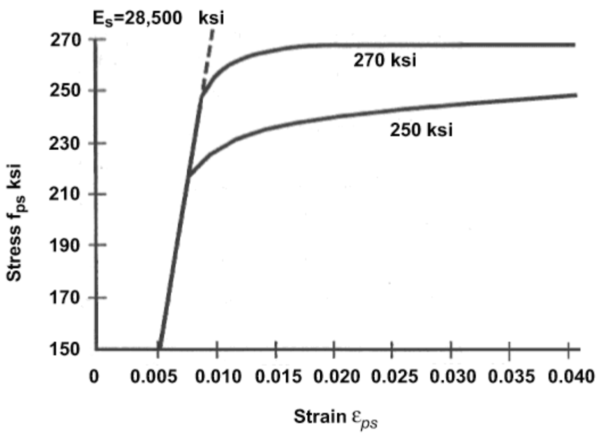

Figure 8.4.3-1—Prestressing Strand Stress–Strain Model

Where prestressed concrete piling is used for pile bents, in-ground hinging may be expected depending on the demand. The reduced ultimate prestress strain limit of 0.03 will permit some damage. If damage is to be limited, reduced strains in the range of 0.015 to 0.02 may be considered (Port of Long Beach, 2007 and Priestley, et al., 2007).

8.4.4—Concrete Modeling

A stress–strain model for confined and unconfined concrete shall be used as depicted in Figure 1. Mander's stress–strain model for confined concrete should be used for determining section response.

The expected concrete compressive strength, f'_{ce}, shall be taken as the most probable long-term concrete strength based on regional experience and shall be taken as:

$$f'_{ce} \geq 1.3 f'_c \qquad (8.4.4\text{-}1)$$

where:

f'_c = compressive strength of concrete (ksi)

The unconfined concrete compressive strain at the maximum compressive stress, ε_{co}, shall be taken as equal to 0.002. The ultimate unconfined compression strain, ε_{sp}, based on spalling shall be taken as equal to 0.005.

The confined compressive strain, ε_{cc}, and the ultimate compressive strain, ε_{cu}, for confined concrete should be computed using Mander's model.

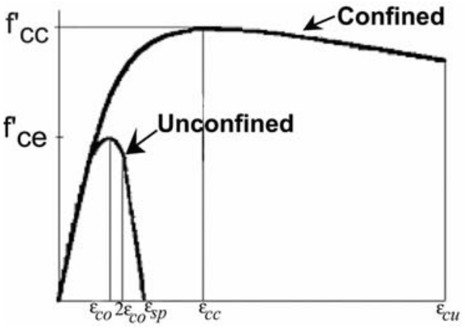

Figure 8.4.4-1—Concrete Stress–Strain Model

C8.4.4

For more information on Mander's confined concrete model, refer to Mander et al. (1988a and 1988b) and Priestley et al. (1996).

Typical values for the ultimate compressive strain, ε_{cu}, range from 0.008 to 0.025 depending on the amount of transverse confinement reinforcement. In normal design practice, ultimate compressive strain values are limited to about 0.02.

Where in-ground plastic hinging is part of the ERS, the confined concrete of the core should be limited to a maximum compressive strain of 0.008 to limit in-ground damage.

8.5—PLASTIC MOMENT CAPACITY FOR DUCTILE CONCRETE MEMBERS FOR SDCS B, C, AND D

The plastic moment capacity of all ductile concrete members shall be calculated by moment-curvature (M-ϕ) analysis on the basis of the expected material properties. The moment-curvature analysis shall include the axial forces due to dead load together with the axial forces due to overturning, as given in Article 4.11.4.

The M-ϕ curve should be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross-section. The elastic portion of the idealized curve shall pass through the point marking the first reinforcing bar yield. The idealized plastic moment capacity shall be obtained by equating the areas between the actual and the idealized M-ϕ curves beyond the first reinforcing bar yield point, as shown in Figure 1.

C8.5

Moment-curvature analysis obtains the curvatures associated with a range of moments for a cross-section on the basis of the principles of strain compatibility and equilibrium of forces. A moment-curvature analysis based on strain compatibility and nonlinear stress–strain relations can be used to determine plastic limit states. The results from this rational analysis are used to establish the rotational capacity of plastic hinges as well as the associated plastic deformations. The process of using the moment-curvature sectional analysis to determine the lateral load-displacement relationship of a frame, column, or pier is known as a "pushover analysis."

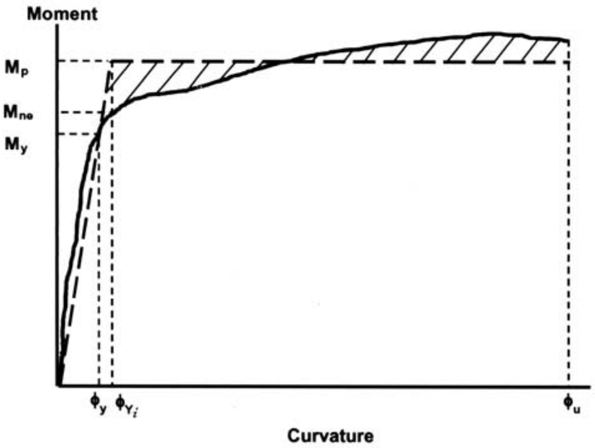

Figure 8.5-1—Moment-Curvature Model

The expected nominal moment capacity, M_{ne}, for essentially elastic response shall be based on the expected concrete and reinforcing steel strengths when the concrete strain reaches a magnitude of 0.003. For SDC B, the expected nominal moment capacity, M_{ne}, may be used as M_p in lieu of development of a moment-curvature analysis.

Use of the expected nominal moment capacity for the plastic moment capacity is acceptable in SDC B, because inelastic demands should be relatively small. This simplification also allows conventional software that can develop the nominal moment capacity as defined in the *AASHTO LRFD Bridge Design Specifications*, albeit with expected material properties, to be used in SDC B.

To determine force demands on capacity-protected members connected to a hinging member, a overstrength magnifier, λ_{mo}, shall be applied to the plastic moment capacity of the hinging member such that:

$$M_{po} = \lambda_{mo} M_p \tag{8.5-1}$$

where:

M_p = idealized plastic moment capacity of reinforced concrete member based on expected material properties (kip-ft)

M_{po} = overstrength plastic moment capacity (kip-ft)

λ_{mo} = overstrength magnifier

= 1.2 for ASTM A 706 reinforcement

= 1.4 for ASTM A 615 Grade 60 reinforcement

The overstrength magnifier, λ_{mo}, accounts for:

- Material strength variations between the column and adjacent members (e.g., superstructure, bent cap, footings, oversized pile shafts), and
- Column moment capacities greater than the idealized plastic moment capacity.

The ultimate curvature, ϕ_u, is determined as the smaller of:

- The ultimate compressive strain, ε_{cu}, of the confined concrete divided by the distance from the plastic neutral axis to the extreme fiber of the confined concrete core, or
- The reduced ultimate tensile strain, ε^R_{su}, of the reinforcing steel divided by the distance from the plastic neutral axis to the extreme tension fiber of the longitudinal column reinforcement.

Typical values for the ultimate curvature, ϕ_u, range from $0.03/B_o$ to $0.08/B_o$ depending on many factors, such as:

- The amount of transverse confinement reinforcement provided,
- The reduced ultimate tensile strain of the longitudinal column reinforcement,
- The magnitude of the axial load, and

- The shape and dimensions of the column cross-section.

The location of the plastic neutral axis is determined based on satisfying the requirements of compatibility and equilibrium of the section using material models such as those outlined in Article 8.4.

8.6—SHEAR DEMAND AND CAPACITY FOR DUCTILE CONCRETE MEMBERS FOR SDCS B, C, AND D

8.6.1—Shear Demand and Capacity

The shear demand for a column, V_u, in SDC B shall be determined on the basis of the lesser of:

- The force obtained from a linear elastic seismic analysis, or
- The force, V_{po}, corresponding to plastic hinging of the column including an overstrength factor.

The shear demand for a column, V_u, in SDC C or D shall be determined on the basis of the force, V_{po}, associated with the overstrength moment, M_{po}, defined in Article 8.5 and outlined in Article 4.11.
The column shear strength capacity within the plastic hinge region as specified in Article 4.11.7 shall be calculated on the basis of the nominal material strength properties and shall satisfy:

$$\phi_s V_n \geq V_u \quad (8.6.1\text{-}1)$$

in which:

$$V_n = V_c + V_s \quad (8.6.1\text{-}2)$$

where:

ϕ_s = 0.90 for shear in reinforced concrete

V_n = nominal shear capacity of member (kips)

V_c = concrete contribution to shear capacity as specified in Article 8.6.2 (kips)

V_s = reinforcing steel contribution to shear capacity as specified in Article 8.6.3 (kips)

The factored nominal shear resistance for members outside the plastic hinge region as defined in Article 4.11.7 shall be determined in accordance with the *AASHTO LRFD Bridge Design Specifications.*

C8.6.1

The requirements of this Article are, in part, intended to avoid column shear failure by using the principles of "capacity protection." For SDCs C and D, the design shear force is specified as a result of the overstrength plastic moment capacity, regardless of the elastic earthquake design forces. This requirement is necessary because of the potential for superstructure collapse if a column fails in shear.

In SDC B, either the elastic shear demand force or the plastic hinging shear force may be used for shear design of a column. It is recommended that the plastic hinging forces be used wherever practical.

A column may be loaded in either the longitudinal or transverse direction. The shear force corresponding to the maximum shear developed in either direction for noncircular columns should be used for the determination of the transverse reinforcement.

8.6.2—Concrete Shear Capacity

The concrete shear capacity, V_c, of members designed for SDCs B, C, and D shall be taken as:

$$V_c = v_c A_e \tag{8.6.2-1}$$

in which:

$$A_e = 0.8 A_g \tag{8.6.2-2}$$

If P_u is compressive:

$$v_c = 0.032\alpha'\left(1+\frac{P_u}{2A_g}\right)\sqrt{f'_c} \le \min\begin{cases} 0.11\sqrt{f'_c} \\ 0.047\alpha'\sqrt{f'_c} \end{cases} \tag{8.6.2-3}$$

otherwise:

$$v_c = 0 \tag{8.6.2-4}$$

For circular columns with spiral or hoop reinforcing:

$$0.3 \le \alpha' = \frac{f_s}{0.15} + 3.67 - \mu_D \le 3 \tag{8.6.2-5}$$

$$f_s = \rho_s f_{yh} \le 0.35 \tag{8.6.2-6}$$

$$\rho_s = \frac{4A_{sp}}{sD'} \tag{8.6.2-7}$$

For rectangular columns with ties:

$$0.3 \le \alpha' = \frac{f_w}{0.15} + 3.67 - \mu_D \le 3 \tag{8.6.2-8}$$

$$f_w = 2\rho_w f_{yh} \le 0.35 \tag{8.6.2-9}$$

$$\rho_w = \frac{A_v}{bs} \tag{8.6.2-10}$$

C8.6.2

The shear provisions in the *AASHTO LRFD Bridge Design Specifications* are not applicable for sections that are expected to accommodate a significant amount of plastic deformation. The concrete shear strength within the plastic hinge region degrades as the ductility demand increases but is improved with increasing transverse confinement.

where:

A_g = gross area of member cross-section (in.^{2})

P_u = ultimate compressive force acting on section (kips)

A_{sp} = area of spiral or hoop reinforcing bar (in.^{2})

s = pitch of spiral or spacing of hoops or ties (in.)

D' = diameter of spiral or hoop for circular column (in.)

A_v = total cross-sectional area of shear reinforcing bars in the direction of loading (in.^{2})

b = width of rectangular column (in.)

f_{yh} = nominal yield stress of transverse reinforcing (ksi)

f'_c = nominal concrete compressive strength (ksi)

μ_D = maximum local displacement ductility ratio of member as defined below

For SDC B, the concrete shear capacity, V_c, of a section within the plastic hinge region shall be determined using $\mu_D = 2$.

For SDC C, the concrete shear capacity of a section within the plastic hinge region shall be determined using $\mu_D = 3$.

For SDC D, the concrete shear capacity of a section within the plastic hinge region shall be determined using the μ_D value determined from Eq. 4.9-5.

8.6.3—Shear Reinforcement Capacity

For members that are reinforced with circular hoops, spirals, or interlocking hoops or spirals as specified in Article 8.6.6, the nominal shear reinforcement strength, V_s, shall be taken as:

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right) \tag{8.6.3-1}$$

where:

n = number of individual interlocking spiral or hoop core sections

A_{sp} = area of spiral or hoop reinforcing bar (in.^{2})

f_{yh} = yield stress of spiral or hoop reinforcement (ksi)

D' = core diameter of column measured from center of spiral or hoop (in.)

s = pitch of spiral or spacing of hoop reinforcement (in.)

C8.6.3

Examples of transverse column reinforcement are shown in Figures C1 to C4. The required total area of hoop reinforcement should be determined for both principal axes of a rectangular or oblong column, and the greater value should be used.

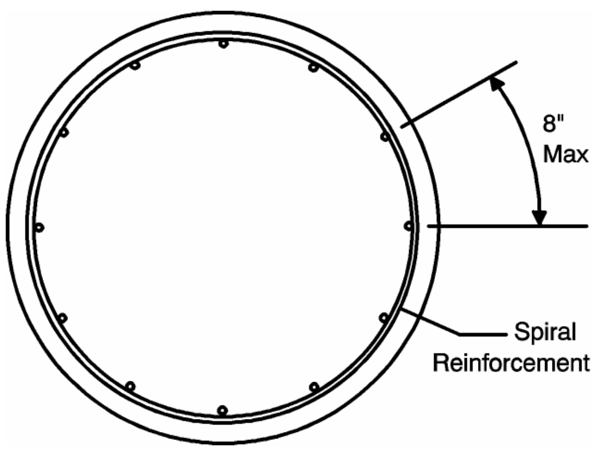

Figure C8.6.3-1 Single Spiral

For members that are reinforced with rectangular ties or stirrups, including pier walls in the weak direction, the nominal shear reinforcement strength, V_s, shall be taken as:

$$V_s = \frac{A_v f_{yh} d}{s} \quad (8.6.3\text{-}2)$$

where:

A_V = cross-sectional area of shear reinforcement in the direction of loading (in.^{2})

d = effective depth of section in direction of loading measured from the compression face of the member to the center of gravity of the tension reinforcement (in.)

f_{yh} = yield stress of tie reinforcement (ksi)

s = spacing of tie reinforcement (in.)

These Guide Specifications allow the use of spirals, hoops, or ties for transverse column reinforcement. The use of spirals is recommended as the most effective and economical solution. Where more than one spiral cage is used to confine an oblong column core, the spirals should be interlocked with longitudinal bars as shown in Figure C3. Center-to-center spacing of the spirals should not exceed 75 percent of the spiral diameter. Spacing of longitudinal bars at a maximum of 8 in. center-to-center is also recommended to help confine the column core.

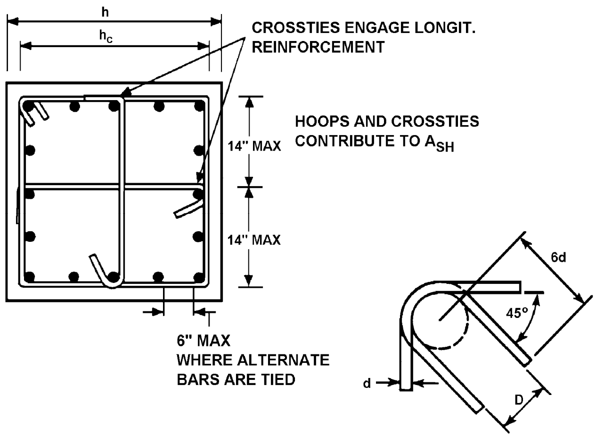

Figure C8.6.3-2 —Column Tie Details

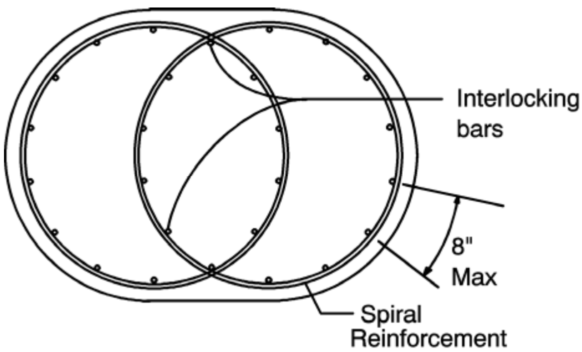

Figure C8.6.3-3—Column Interlocking Spiral Details

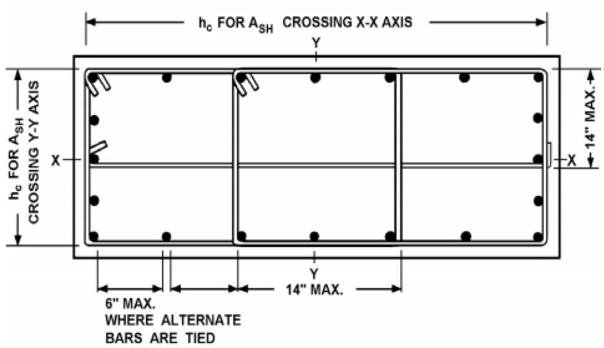

Figure C8.6.3-4—Column Tie Details

8.6.4—Maximum Shear Reinforcement

The shear strength provided by the reinforcing steel, V_s, shall not be taken as greater than:

$$V_s \leq 0.25\sqrt{f'_c}A_e \tag{8.6.4-1}$$

where:

A_e = effective area of the cross-section for shear resistance as defined by Eq. 8.6.2-2 (in.^{2})

f'_c = compressive strength of concrete (ksi)

C8.6.4

This requirement is intended to ensure that the concrete in the section does not crush prior to yield of the transverse reinforcement.

8.6.5—Minimum Shear Reinforcement

The area of column spiral or circular hoop reinforcement, A_{sp}, and column web reinforcement, A_v, shall be used to determine the reinforcement ratios, ρ_s and ρ_w, as given by Eq. 8.6.2-7 and Eq. 8.6.2-10, respectively. The spiral or circular hoop reinforcement ratio, ρ_s, for each individual circular core of a column and the web reinforcement ratio, ρ_w, shall satisfy:

- For SDC B:

$$\rho_s \geq 0.003 \tag{8.6.5-1}$$

$$\rho_w \geq 0.002 \tag{8.6.5-2}$$

- For SDCs C and D:

$$\rho_s \geq 0.005 \tag{8.6.5-3}$$

$$\rho_w \geq 0.004 \tag{8.6.5-4}$$

8.6.6—Shear Reinforcement Capacity of Interlocking Spirals

The shear reinforcement strength provided by interlocking spirals or hoops shall be taken as the sum of all individual spiral or hoop shear strengths determined in accordance with Eq. 8.6.3-1.

8.6.7—Minimum Vertical Reinforcement in Interlocking Portion

The longitudinal reinforcing bars in the interlocking portion of the column shall have a maximum spacing of 8 in. and need not be anchored in the footing or the bent cap unless deemed necessary for the flexural capacity of the column. The longitudinal reinforcing bar size in the interlocking portion of the column shall be chosen correspondingly to the reinforcing bars outside the interlocking portion as specified in Table 1.

Table 8.6.7-1—Reinforcement Size for Interlocking Portion of Columns

Minimum size of bars required inside the interlocking portion	Size of bars used outside the interlocking portion
#6	#10
#8	#11
#9	#14
#11	#18

8.6.8—Pier Wall Shear Capacity in the Weak Direction

The shear capacity for pier walls in the weak direction shall be determined according to Articles 8.6.1, 8.6.2, and 8.6.3.

8.6.9—Pier Wall Shear Capacity in the Strong Direction

The factored nominal shear capacity of pier walls in the strong direction, ϕV_n, shall be greater than the maximum shear demand, V_u, as specified in Eq. 1. The maximum shear demand, V_u, need not be taken as greater than the lesser of:

- The overstrength capacity of the superstructure to substructure connection,
- The overstrength capacity of the foundation,
- The force demands determined in accordance with Article 8.3, or
- The unreduced elastic demand obtained when using analysis Procedure 1 or 2 specified in Article 4.2.

$$\phi_s V_n \geq V_u \quad (8.6.9\text{-}1)$$

in which:

$$V_n = \left(0.13\sqrt{f'_c} + \rho_h f_{yh}\right) bd \leq 0.25\sqrt{f'_c} A_e \quad (8.6.9\text{-}2)$$

$$\rho_h = \frac{A_v}{bs} \quad (8.6.9\text{-}3)$$

where:

ϕ_s = 0.90 for shear in reinforced concrete

A_v = cross-sectional area of shear reinforcement in the direction of loading (in.^{2})

d = depth of section in direction of loading (in.)

b = width of section (in.)

C8.6.9

Studies of squat shear walls have demonstrated that the large shear stresses associated with the moment capacity of the wall may lead to a sliding failure brought about by crushing of the concrete at the base of the wall. The thickness of pier walls should be selected such that the shear stress satisfies the upper limit specified in Eq. 2.

f_{yh} = yield stress of tie reinforcement (ksi)

f'_c = compressive strength of concrete (ksi)

s = spacing of tie reinforcement (in.)

A_e = effective area of the cross-section for shear resistance as defined by Eq. 8.6.2-2 (in.^{2})

8.6.10—Pier Wall Minimum Reinforcement

The horizontal reinforcement ratio, ρ_h, shall not be less than 0.0025. The vertical reinforcement ratio, ρ_v, shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 18 in.

The reinforcement required for shear shall be continuous and shall be distributed uniformly. Horizontal and vertical layers of reinforcement shall be provided on each face of a pier. Splices in horizontal pier reinforcement shall be staggered.

C8.6.10

The requirement that $\rho_v \geq \rho_h$ is intended to avoid the possibility of having inadequate web reinforcement in piers that are short in comparison with their height.

Stagger splices to avoid weakened sections.

8.7—REQUIREMENTS FOR DUCTILE MEMBER DESIGN

8.7.1—Minimum Lateral Strength

The minimum lateral flexural capacity of each column shall be taken as:

$$M_{ne} \geq 0.1 P_{trib} \left(\frac{H_h + 0.5 D_s}{\Lambda} \right) \quad (8.7.1\text{-}1)$$

where:

M_{ne} = nominal moment capacity of the column based on expected material properties as shown in Figure 8.5-1 (kip-ft)

P_{trib} = greater of the dead load per column or force associated with the tributary seismic mass collected at the bent (kips)

H_h = the height from the top of the footing to the top of the column or the equivalent column height for a pile extension column (ft)

D_s = depth of superstructure (ft)

Λ = fixity factor for the column defined in Article 4.8.1

The flexural capacity of pile extension members and pier walls in the weak direction shall also satisfy the requirements of Eq. 1 when the ductility demand is >1.

8.7.2—Maximum Axial Load in a Ductile Member in SDCs C and D

The maximum axial load acting on a column or pier where the ductility demand, μ_D, is >2 and a moment-curvature pushover analysis is not performed shall satisfy:

$$P_u \leq 0.2\, f'_c A_g \quad (8.7.2\text{-}1)$$

where:

P_u = ultimate compressive force acting on the section including seismic-induced vertical demands (kips)

f'_c = compressive strength of concrete (ksi)

A_g = gross area of member cross-section (in.^{2})

A higher axial load value, P_u, may be used provided that a moment-curvature pushover analysis is performed to compute the maximum ductility capacity of the member.

8.8—LONGITUDINAL AND LATERAL REINFORCEMENT REQUIREMENTS

8.8.1—Maximum Longitudinal Reinforcement

The area of longitudinal reinforcement for compression members shall satisfy:

$$A_\ell \leq 0.04 A_g \quad (8.8.1\text{-}1)$$

where:

A_g = gross area of member cross-section (in.^{2})

A_ℓ = area of longitudinal reinforcement in the member (in.^{2})

C8.8.1

This requirement is intended to apply to the full section of the columns. The maximum ratio is to avoid congestion and extensive shrinkage cracking and to permit anchorage of the longitudinal steel, but most important, the smaller the amount of longitudinal reinforcement, the greater the ductility of the column.

8.8.2—Minimum Longitudinal Reinforcement

The minimum area of longitudinal reinforcement for compression members shall not be less than:

- For columns in SDCs B and C:

$$A_\ell \geq 0.007 A_g \quad (8.8.2\text{-}1)$$

- For columns in SDC D:

$$A_\ell \geq 0.010 A_g \quad (8.8.2\text{-}2)$$

- For pier walls in SDCs B and C:

$$A_\ell \geq 0.0025 A_g \quad (8.8.2\text{-}3)$$

C8.8.2

This requirement is intended to apply to the full section of the columns. The lower limit on the column or wall reinforcement reflects the traditional concern for the effect of time-dependent deformations as well as the desire to avoid a sizable difference between the flexural cracking and yield moments.

- For pier walls in SDC D:

$$A_\ell \geq 0.005 A_g \tag{8.8.2-4}$$

where:

A_g = gross area of member cross-section (in.^{2})

A_ℓ = area of longitudinal reinforcement in the member (in.^{2})

8.8.3—Splicing of Longitudinal Reinforcement in Columns Subject to Ductility Demands for SDCs C and D

Splicing of longitudinal column reinforcement in SDC C or D shall be outside the plastic hinging region as defined in Article 4.11.7, except as permitted below.

For a pile or shaft in SDC D where liquefaction is anticipated, the design shall consider that the zone comprising the location of potential plastic hinging in the liquefied and nonliquefied cases can be large. For a pile or shaft in SDC D where splicing in the zone cannot be avoided, mechanical couplers that are capable of developing the expected tensile strength of the bars and as approved by the Owner shall be specified.

C8.8.3

It is often desirable to lap longitudinal reinforcement with dowels at the column base. This is undesirable for seismic performance because:

- The splice occurs in a potential plastic hinge region where requirements for bond are critical, and
- Lapping the main reinforcement will tend to concentrate plastic deformation close to the base and reduce the effective plastic hinge length as a result of stiffening of the column over the lapping region, which may result in a severe local curvature demand.

At the discretion of the Owner, the splicing requirements of this Article may be applied to SDC B.

8.8.4—Minimum Development Length of Reinforcing Steel for SDCs C and D

Column longitudinal reinforcement shall be extended into footings and cap beams as close as practically possible to the opposite face of the footing or cap beam.

The anchorage length for longitudinal column bars developed into the cap beam or footing for seismic loads shall satisfy:

$$\ell_{ac} \geq \frac{0.79 d_{b\ell} f_{ye}}{\sqrt{f'_c}} \tag{8.8.4-1}$$

where:

ℓ_{ac} = anchored length of longitudinal reinforcing bars into the cap beam or footing (in.)

$d_{b\ell}$ = diameter of the longitudinal column bar (in.)

f_{ye} = expected yield stress of the longitudinal reinforcement (ksi)

f'_c = nominal compressive strength of concrete (ksi)

For SDC D, the anchorage length shall not be reduced by means of adding hooks or mechanical anchorage devices. If hooks are provided, the tails should be pointed inward toward the joint core.

8.8.5—Anchorage of Bundled Bars in Ductile Components for SDCs C and D

The anchorage length of individual column bars within a bundle anchored into a cap beam shall be increased by 20 percent for a two-bar bundle and 50 percent for a three-bar bundle. Four-bar bundles shall not be permitted in ductile elements.

8.8.6—Maximum Bar Diameter for SDCs C and D

To ensure adequate bond to concrete, the nominal diameter of longitudinal reinforcement, $d_{b\ell}$, in columns shall satisfy:

$$d_{b\ell} \le \frac{0.79\sqrt{f'_c}\left(L-0.5D_c\right)}{f_{ye}} \qquad (8.8.6\text{-}1)$$

where:

L = length of the column from the point of contraflexure to the point of maximum moment based on capacity design principles (in.)

D_c = diameter or depth of the column in direction of loading (in.)

f'_c = nominal compressive strength of concrete (ksi)

f_{ye} = the expected yield strength (ksi)

Where longitudinal bars in columns are bundled, the requirement of adequate bond (Eq. 1) shall be checked for the effective bar diameter, assumed as $1.2d_{b\ell}$ for two-bar bundles, and $1.5d_{b\ell}$ for three-bar bundles.

C8.8.6

In short columns, where plastic hinges of opposite sign develop simultaneously at the top and bottom of the column, bond conditions caused by the requirement to transfer force from bar to concrete as a result of the rapidly changing moment may be extreme. It is thus important to use smaller diameter bars in such situations (Priestley et al., 1996).

8.8.7—Lateral Reinforcement Inside the Plastic Hinge Region for SDCs C and D

The volume of lateral reinforcement, ρ_s or ρ_w, specified in Article 8.6.2 provided inside the plastic hinge region as specified in Article 4.11.7 shall be sufficient to ensure that the column or pier wall has adequate shear capacity and confinement level to achieve the required ductility capacity.

C8.8.7

These provisions ensure that the concrete is adequately confined so that the transverse hoops will not prematurely fracture as a result of the plastic work done on the critical column section. For typical bridge columns with low levels of axial load, these equations rarely govern but should be checked.

If a section has been detailed in accordance with the transverse reinforcement requirement of these Guide Specifications, then the section is assumed to be "capacity protected" against undesirable modes of failure such as shear, buckling of longitudinal bars, and concrete crushing due to lack of confinement.

Longitudinal reinforcing bars in potential plastic hinge zones may be highly strained in compression to the extent that they may buckle. Buckling of longitudinal reinforcing may be either:

For columns designed to achieve a displacement ductility demand greater than 4, the lateral reinforcement shall be either butt-welded hoops or spirals.

Combination of hoops and spiral shall not be permitted except in the footing or the bent cap. Hoops may be placed around the column cage (i.e., extended longitudinal reinforcing steel) in lieu of continuous spiral reinforcement in the cap and footing.

At spiral or hoop-to-spiral discontinuities, the spiral shall terminate with one extra turn plus a tail equal to the cage diameter.

a. Local between two successive hoop sets or spirals, or

b. Global and extend over several hoop sets or spirals.

Condition (a) is prevented by using the maximum vertical spacing of transverse reinforcement given by Article 8.8.9.

Although research has been conducted to determine the amount of transverse reinforcement required to prevent condition (b), this research has not been fully peer reviewed, and thus has not been included as part of these Guide Specifications. However, Designers should not ignore the possibility of condition (b) and should take steps to prevent it from occurring (see the final report for the NCHRP 12-49 project and other related research).

Preventing the loss of concrete cover in the plastic hinge zone as a result of spalling requires careful detailing of the confining steel. It is inadequate to simply lap the spiral reinforcement. If the concrete cover spalls, the spiral will be able to unwind, resulting in a sudden loss of concrete confinement. Similarly, rectangular hoops should be anchored by bending ends back into the core.

8.8.8—Lateral Column Reinforcement Outside the Plastic Hinge Region for SDCs C and D

The volumetric ratio of lateral reinforcement required outside of the plastic hinge region shall not be less than 50 percent of that determined in accordance with Article 8.8.7 and 8.6.

The lateral reinforcement type outside the plastic hinge region shall be the same type as that used inside the plastic hinge region.

At spiral or hoop-to-spiral discontinuities, splices shall be provided that are capable of developing at least 125 percent of the specified minimum yield stress, f_{yh}, of the reinforcing bar.

Lateral reinforcement shall extend into footings to the beginning of the longitudinal bar bend above the bottom mat.

Lateral reinforcement shall extend into bent caps at a distance that is as far as is practical and adequate to develop the reinforcement for development of plastic hinge mechanisms.

8.8.9—Requirements for Lateral Reinforcement for SDCs B, C, and D

All longitudinal bars in compression members shall be enclosed by lateral reinforcement.

Transverse hoop reinforcement may be provided by single or overlapping hoops. Cross-ties having the same bar size as the hoop may be used. Each end of the cross-tie shall engage a peripheral longitudinal reinforcing bar. All cross-ties shall have seismic hooks.

Seismic hooks shall consist of a 135° bend, plus an extension of not less than the larger of 6.0 bar diameters or 3.0 in. Seismic hooks shall be used for transverse reinforcement in regions of expected plastic hinges. Such hooks and their required locations shall be detailed in the contract documents.

Transverse reinforcement meeting the following requirements shall be considered to be a cross-tie:

- The bar shall be a continuous bar having a hook of not less than 135°, with an extension of not less than six diameters but not less than 3.0 in. at one end and a hook of not less than 90° with an extension of not less than six diameters at the other end.
- The hooks shall engage peripheral longitudinal bars.
- The 90° hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Transverse reinforcement meeting the following requirements shall considered to be a hoop:

- The bar shall be closed tie or continuously wound tie.
- A closed tie may be made up of several reinforcing elements with 135° hooks having a six-diameter but not less than a 3.0-in. extension at each end.
- A continuously wound tie shall have at each end a 135° hook with a six-diameter but not less than a 3.0-in. extension that engages the longitudinal reinforcement.

The minimum size of lateral reinforcing bars shall be:

- #4 bars for #9 or smaller longitudinal bars,
- #5 bars for #10 or larger longitudinal bars, and
- #5 bars for bundled longitudinal bars.

The maximum spacing for lateral reinforcement in the plastic hinge regions as defined in Article 4.11.7 shall not exceed the smallest of:

- One-fifth of the least dimension of the cross-section for columns and one-half of the least cross-section dimension of piers,

C8.8.9

In addition to providing shear strength and concrete confinement, lateral reinforcement is used to provide lateral support to the longitudinal column reinforcement. See Figures C8.6.3-2 and C8.6.3-4 for examples of typical lateral tie reinforcement details. See Figures C8.6.3-1 and C8.6.3-3 for examples of typical hoop and spiral reinforcement details.

For SDC B, the plastic hinging region may alternately be taken as that defined in Article 5.10.11.4.1e of the *AASHTO LRFD Bridge Design Specifications*.

- Six times the nominal diameter of the longitudinal reinforcement,
- 6 in. for single hoop or spiral reinforcement,
- 8 in. for bundled hoop reinforcement.

With the Owner's approval, the use of deformed wire, wire rope, or welded wire fabric of equivalent area should be permitted instead of bars for the ties, hoops, or spirals.

8.8.10—Development Length for Column Bars Extended into Oversized Pile Shafts for SDCs C and D

Column longitudinal reinforcement should be extended into enlarged shafts in a staggered manner with the minimum embedment lengths of $2D_{c,max}$ and $3D_{c,max}$, where $D_{c,max}$ is the larger cross-section dimension of the column. Other methods of developing longitudinal column reinforcement in the shaft may be used if confirmed by experimental test data and approved by Owner.

C8.8.10

Terminating all of the column reinforcement in the oversized shaft at one location will result in a weakened section with a sudden change in stiffness. Such conditions should be avoided.

8.8.11—Lateral Reinforcement Requirements for Columns Supported on Oversized Pile Shafts for SDCs C and D

The volumetric ratio of lateral reinforcement for columns supported on oversized pile shafts shall meet the requirements specified in Articles 8.8.7 and 8.8.8. At least 50 percent of the confinement reinforcement required at the base of the column shall extend over the entire embedded length of the column cage.

8.8.12—Lateral Confinement for Oversized Pile Shafts for SDCs C and D

The volumetric ratio of lateral reinforcement in an oversized shaft shall be 50 percent of the confinement at the base of the column provided the shaft is designed for a flexural expected nominal capacity equal to 1.25 times the moment demand generated by the overstrength moment of the embedded column. The lateral confinement shall extend along the shaft until the embedded column cage is terminated. The spacing of the oversized shaft confinement may be doubled beyond the column cage termination length.

8.8.13—Lateral Confinement for Non-Oversized Strengthened Pile Shafts for SDCs C and D

The volumetric ratio of lateral confinement in the top segment of the shaft, $4D_{c,max}$, where $D_{c,max}$ is the larger cross-section dimension of the column, shall be at least 75 percent of the confinement reinforcement required at the base of the column provided the shaft is designed for a flexural expected nominal capacity equal to 1.25 times the moment demand generated by the overstrength moment of the embedded column. The lateral confinement shall extend along the shaft until the embedded column cage is terminated. The spacing of the shaft confinement can be doubled beyond the column cage termination length.

8.9—REQUIREMENTS FOR CAPACITY-PROTECTED MEMBERS

Capacity-protected members such as footings, bent caps, oversized pile shafts, joints, and integral superstructure elements that are adjacent to the plastic hinge locations shall be designed to remain essentially elastic when the plastic hinge reaches its overstrength moment capacity, M_{po}.

The expected nominal capacity, M_{ne}, is used in establishing the capacity of essentially elastic members and should be determined based on a strain compatibility analysis using a M-ϕ diagram as illustrated in Figure 8.5-1 and outlined in Article 8.5.

C8.9

All loads acting on the capacity-protected member should be considered when determining the factored nominal capacity of the member. For example, the axial demands (including tension in columns or uplift in piles) imparted on a bent cap beam or footing due to the lateral demands should be considered when calculating a cap beam's or footing's nominal capacity.

Typically, the design forces in the capacity-protected member resulting from the overstrength plastic hinge capacity and other demands are taken at the face of the column.

8.10—SUPERSTRUCTURE CAPACITY DESIGN FOR INTEGRAL BENT CAPS FOR LONGITUDINAL DIRECTION FOR SDCS C AND D

The superstructure shall be designed as a capacity-protected member. Any moment demand caused by dead load or secondary prestress effects shall be distributed to the entire width of the superstructure. The column overstrength moment, M_{po}, in addition to the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the superstructure shall be distributed to the spans framing into the bent on the basis of their stiffness distribution factors. This moment demand shall be considered within the effective width of the superstructure.

The effective width of superstructure resisting longitudinal seismic moments, B_{eff}, shall be determined by Eqs. 1 and 2:

- For box girders and solid superstructure:

$$B_{eff} = D_c + 2D_s \tag{8.10-1}$$

- For open soffit, girder-deck superstructures:

$$B_{eff} = D_c + D_s \tag{8.10-2}$$

C8.10

The effective width for open soffit structures (i.e., T-beams and I-girders) is reduced because they offer less resistance to the torsional rotation of the bent cap. The effective superstructure width can be increased at a 45° angle away from the bent cap until the full section becomes effective. On skewed bridges, the effective width shall be projected normal to the girders where the centerline of girder intersects the face of the bent cap. (See Figure C1.)

Additional superstructure width can be considered effective if the Designer verifies that the torsional stiffness of the cap can distribute the rotational demands beyond the effective widths stated in Eqs. 1 and 2.

where:

D_c = diameter of column (in.)

D_s = depth of superstructure (in.)

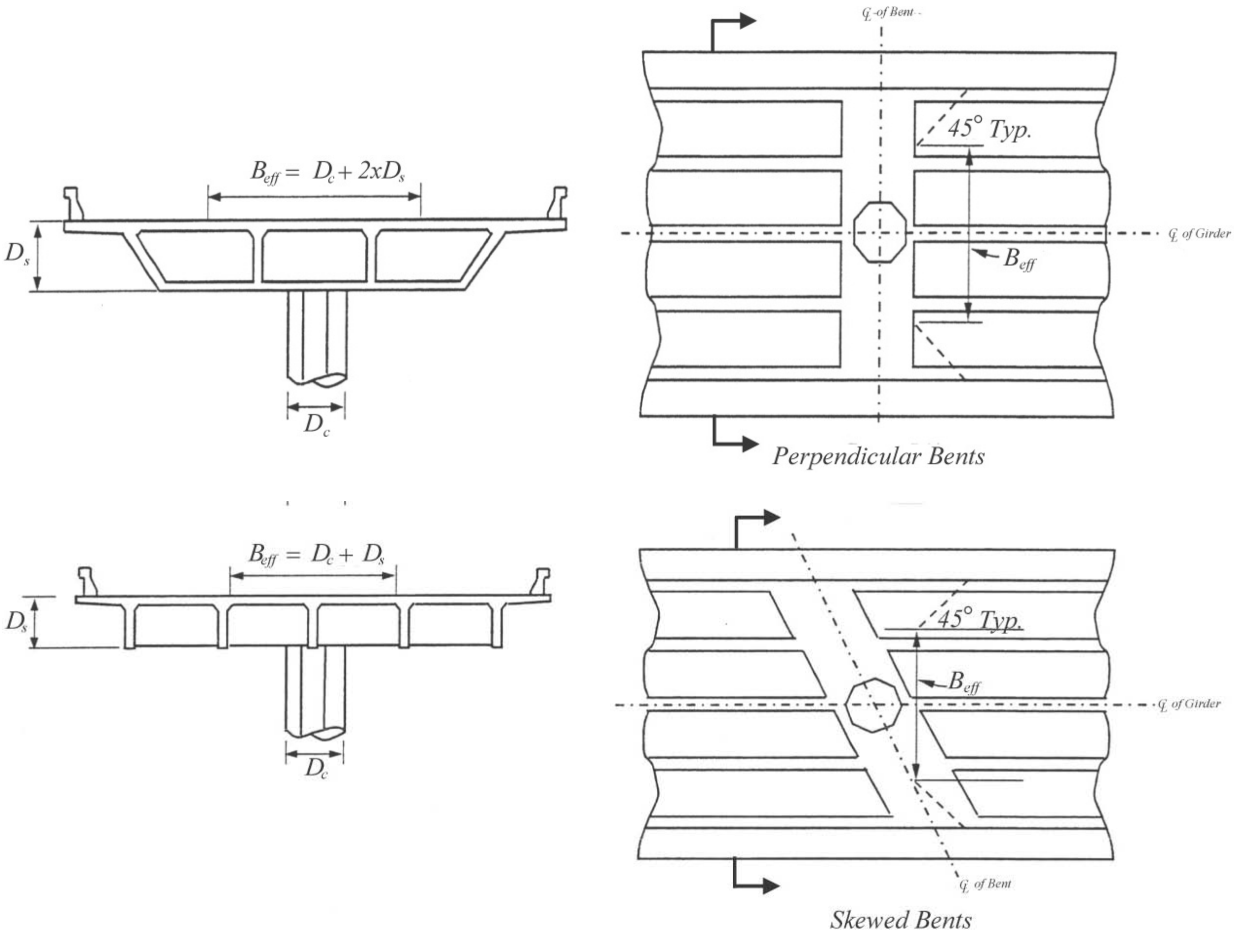

Figure C8.10-1—Effective Superstructure Width

8.11—SUPERSTRUCTURE CAPACITY DESIGN FOR TRANSVERSE DIRECTION (INTEGRAL BENT CAP) FOR SDCS C AND D

Bent caps are considered integral if they terminate at the outside of the exterior girder and respond monolithically with the girder system during dynamic excitation.

The bent cap shall be designed as an essentially elastic member. Any moment demand caused by dead load or secondary prestress effects shall be distributed to the effective width of the bent cap, B_{eff}, as shown in Figure 1.

The column overstrength moment, M_{po}, and the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the bent cap shall be distributed on the basis of the effective stiffness characteristics of the frame. The moment shall be considered within the effective width of the bent cap. The effective width, B_{eff}, shall be taken as:

$$B_{eff} = B_{cap} + 12t \tag{8.11-1}$$

where:

t = thickness of the top or bottom slab (in.)

B_{cap}= thickness of the bent cap (in.)

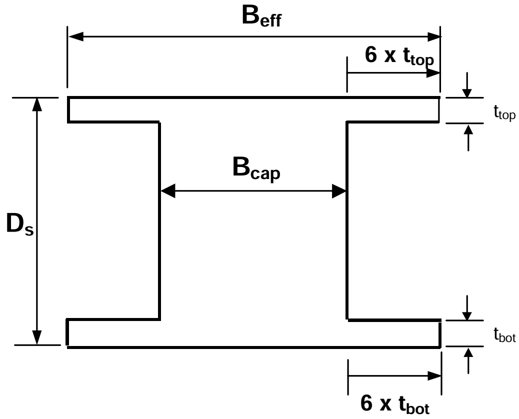

Figure 8.11-1—Effective Bent Cap Width

For SDCs C and D, longitudinal flexural bent cap beam reinforcement shall be continuous. As a minimum, splicing of reinforcement shall be accomplished using mechanical couplers capable of developing 125 percent of the expected yield strength, f_{ye}, of the reinforcing bars.

8.12—SUPERSTRUCTURE DESIGN FOR NONINTEGRAL BENT CAPS FOR SDCS C AND D

Nonintegral bent caps shall satisfy all requirements stated for frames with integral bent cap in the transverse direction.

For superstructure-to-substructure connections that are not intended to fuse, a lateral force transfer mechanism shall be provided at the interface that is capable of transferring the maximum lateral force associated with plastic hinging of the ERS. For superstructure-to-substructure connections that are intended to fuse, the minimum lateral force at the interface shall be taken as 0.40 times the dead load reaction plus the overstrength shear key(s) capacity, V_{ok}.

Superstructure members supported on nonintegral bent caps shall be simply supported at the bent cap or span continuously with a separation detail such as an elastomeric pad or isolation bearing between the bent cap and the superstructure. Refer to the Type 3 choice of Article 7.2.

Nonintegral cap beams supporting superstructures with expansion joints at the cap shall have sufficient support length to prevent unseating. The minimum support lengths for nonintegral bent caps shall be determined on the basis of Article 4.12. Continuity devices such as rigid restrainers or web plates are permissible to help ensure that unseating does not occur but shall not be used in lieu of adequate bent cap width.

8.13—JOINT DESIGN FOR SDCS C AND D

8.13.1—Joint Performance

Moment-resisting connections shall be designed to transmit the maximum forces produced when the column has reached its overstrength capacity, M_{po}.

C8.13.1

A "rational" design is required for joint reinforcement when principal tension stress levels become excessive. The amounts of reinforcement required are based on a strut and tie mechanism similar to that shown in Figure C1.

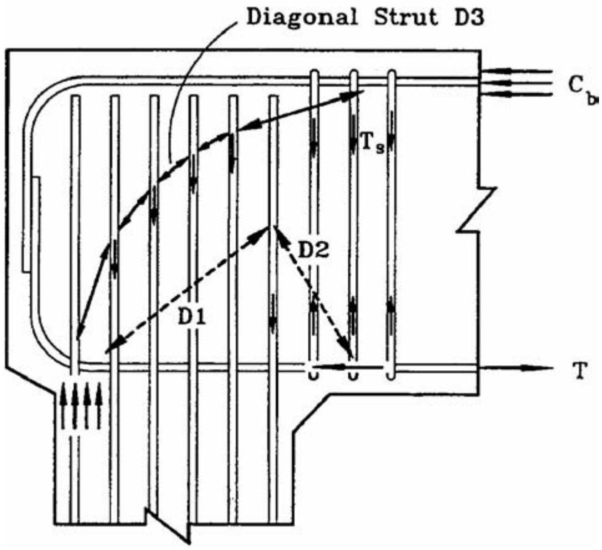

Figure C8.13.1-1—External Vertical Joint Reinforcement for Joint Force Transfer

8.13.2—Joint Proportioning

Moment-resisting joints shall be proportioned so that the principal stresses satisfy the requirements of Eq. 1 and Eq. 2.

- For principal compression, p_c:

$$p_c \le 0.25\, f'_c \tag{8.13.2-1}$$

- For principal tension, p_t:

$$p_t \le 0.38\sqrt{f'_c} \tag{8.13.2-2}$$

in which:

$$p_t = \left| \left(\frac{f_h + f_v}{2} \right) - \sqrt{\left(\frac{f_h - f_v}{2} \right)^2 + v_{jv}^2} \right| \tag{8.13.2-3}$$

$$p_c = \left(\frac{f_h + f_v}{2} \right) + \sqrt{\left(\frac{f_h - f_v}{2} \right)^2 + v_{jv}^2} \tag{8.13.2-4}$$

$$v_{jv} = \frac{T_c}{A_{jv}} \tag{8.13.2-5}$$

$$A_{jv} = \ell_{ac} B_{cap} \tag{8.13.2-6}$$

$$f_v = \frac{P_c}{A_{jh}} \tag{8.13.2-7}$$

$$A_{jh} = \left(D_c + D_s \right) B_{cap} \tag{8.13.2-8}$$

$$f_h = \frac{P_b}{B_{cap} D_s} \tag{8.13.2-9}$$

$$Tc = \frac{M_{po}}{h} \tag{8.13.2-10}$$

where:

B_{cap} = bent cap width (in.)

D_c = cross-sectional dimension of column in the direction of bending (in.)

D_s = depth of superstructure at the bent cap for integral joints or depth of cap beam for nonintegral bent caps (in.)

ℓ_{ac} = length of column reinforcement embedded into the bent cap (in.)

P_c = column axial force including the effects of overturning (kips)

C8.13.2

Figure C1 illustrates the forces acting on the joint as well as the associated principal stresses.

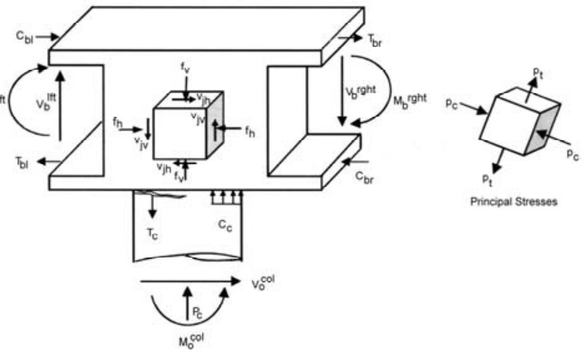

Figure C8.13.2-1—Stress in T-Joints

The substitution of f'_{ce} for f'_c throughout Article 8.13 may be acceptable provided that historic concrete test data and the Owner's approval support this action.

Unless a horizontal prestressing force is specifically designed to provide horizontal joint compression, f_h can typically be ignored without significantly affecting the principal stress calculation.

P_b = beam axial force at the center of the joint including the effects of prestressing and the shear associated with plastic hinging (kips)

h = distance from the center of gravity of the tensile force to the center of gravity of the compressive force of the column section (in.)

T_c = column tensile force associated with the column overstrength plastic hinging moment, M_{po} (kips)

M_{po} = overstrength plastic moment capacity of column determined in accordance with Article 8.5 (kip-in.)

In lieu of Eq. 10, T_c may be obtained directly from the moment-curvature analysis.

8.13.3—Minimum Joint Shear Reinforcing

Transverse reinforcement in the form of tied column reinforcement, spirals, hoops, or intersecting spirals or hoops shall be provided. The joint shear reinforcement may also be provided in the form of column transverse steel or exterior transverse reinforcement continued into the bent cap.

Where the principal tension stress in the joint, p_t, as specified in Article 8.13.2 is less than $0.11\sqrt{f'_c}$, the transverse reinforcement in the joint, ρ_s, shall satisfy Eq. 1 and no additional reinforcement within the joint is required:

$$\rho_s \geq \frac{0.11\sqrt{f'_c}}{f_{yh}} \tag{8.13.3-1}$$

where:

f_{yh} = nominal yield stress of transverse reinforcing (ksi)

f'_c = nominal concrete compressive strength (ksi)

ρ_s = volumetric reinforcement ratio of transverse reinforcing provided within the cap as defined by Eq. 8.6.2-7

Where the principal tension stress in the joint, p_t, is greater than $0.11\sqrt{f'_c}$, then transverse reinforcement in the joint, ρ_s, shall satisfy Eq. 2 and additional joint reinforcement is required as indicated in Article 8.13.4 for integral bent cap beams or Article 8.13.5 for nonintegral bent cap beams:

$$\rho_s \geq 0.40\frac{A_{st}}{\ell_{ac}^2} \tag{8.13.3-2}$$

where:

A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

ℓ_{ac} = length of column reinforcement embedded into the bent cap (in.)

For interlocking cores, ρ_s shall be based on the total area of reinforcement of each core.

8.13.4—Integral Bent Cap Joint Shear Design

8.13.4.1—Joint Description

The following types of joints are considered "T" joints for joint shear analysis:

- Integral interior joints of multicolumn bents in the transverse direction,
- All column/superstructure joints in the longitudinal direction, and
- Exterior column joints for box girder superstructures if the cap beam extends beyond the joint far enough to develop the longitudinal cap reinforcement.

All other exterior joints shall be considered knee joints in the transverse direction and require special analysis and detailing that are not addressed herein.

The bent cap width shall extend 12 in. on each side of the column as shown in Figure 8.13.4.2.1-2.

C8.13.4.1

The design of beam–column joints is based on research and experiments for circular columns framing into rectangular beams. Although no specific requirements have been developed for rectangular columns framing into rectangular beams, the requirements of this Article may be used.

8.13.4.2—Joint Shear Reinforcement

8.13.4.2.1—Vertical Stirrups

Vertical stirrups or ties shall be placed transversely within a distance equal to the column diameter, Dc, extending from either side of the column centerline. The vertical stirrup area, A_s^{jv}, shall be provided on each side of the column or pier wall, as depicted Figures 1, 2, and 3. The stirrups provided in the overlapping areas shown in Figure 1 shall count toward meeting the requirements of both areas creating the overlap. These stirrups may be used to meet other requirements documented elsewhere, including the shear in the bent cap:

$$A_s^{jv} \geq 0.20 A_{st} \qquad (8.13.4.2.1\text{-}1)$$

where:

A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

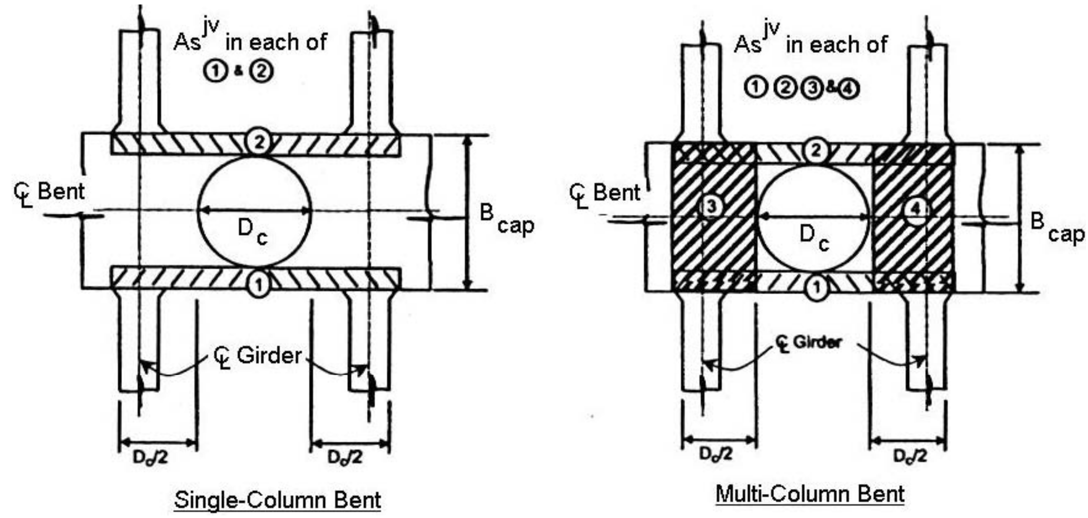

Figure 8.13.4.2.1-1—Location of Vertical Joint Shear Reinforcement

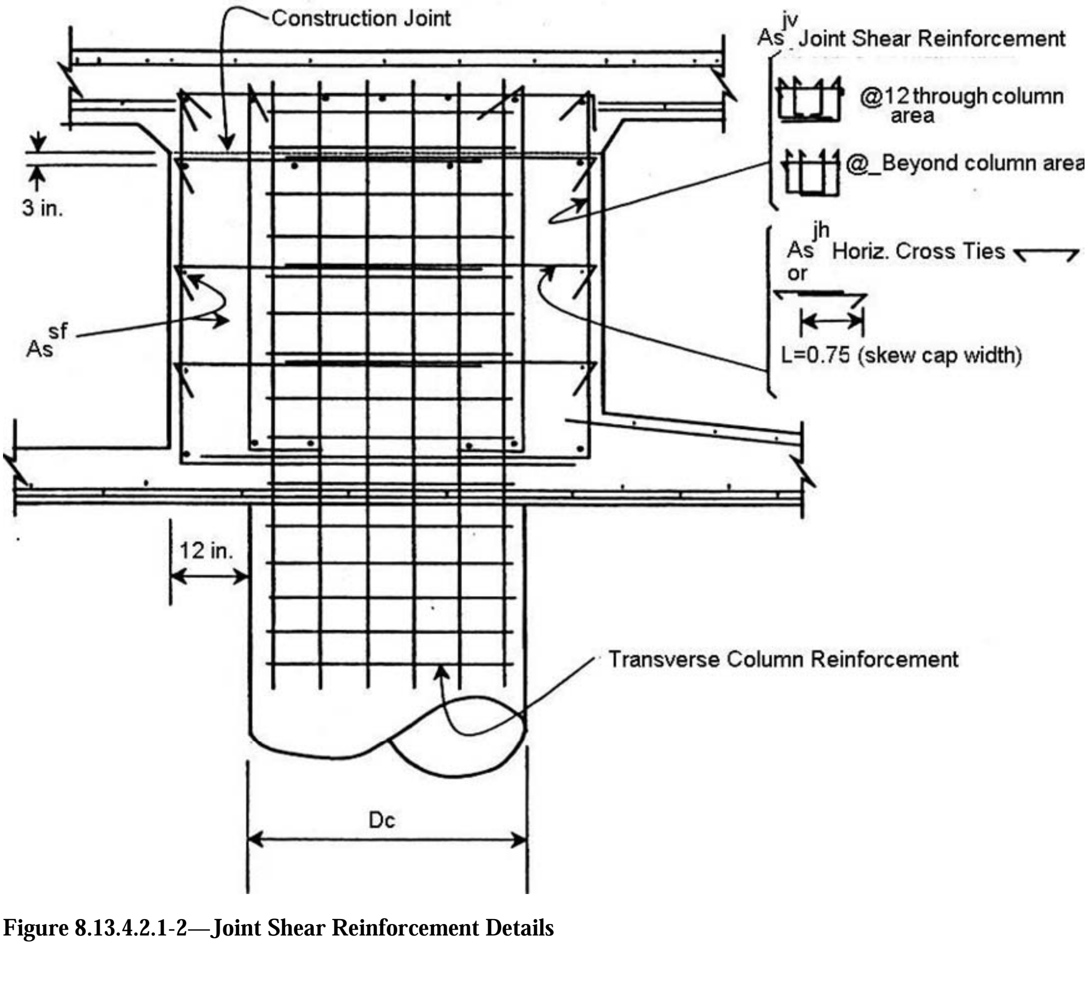

Figure 8.13.4.2.1-2—Joint Shear Reinforcement Details

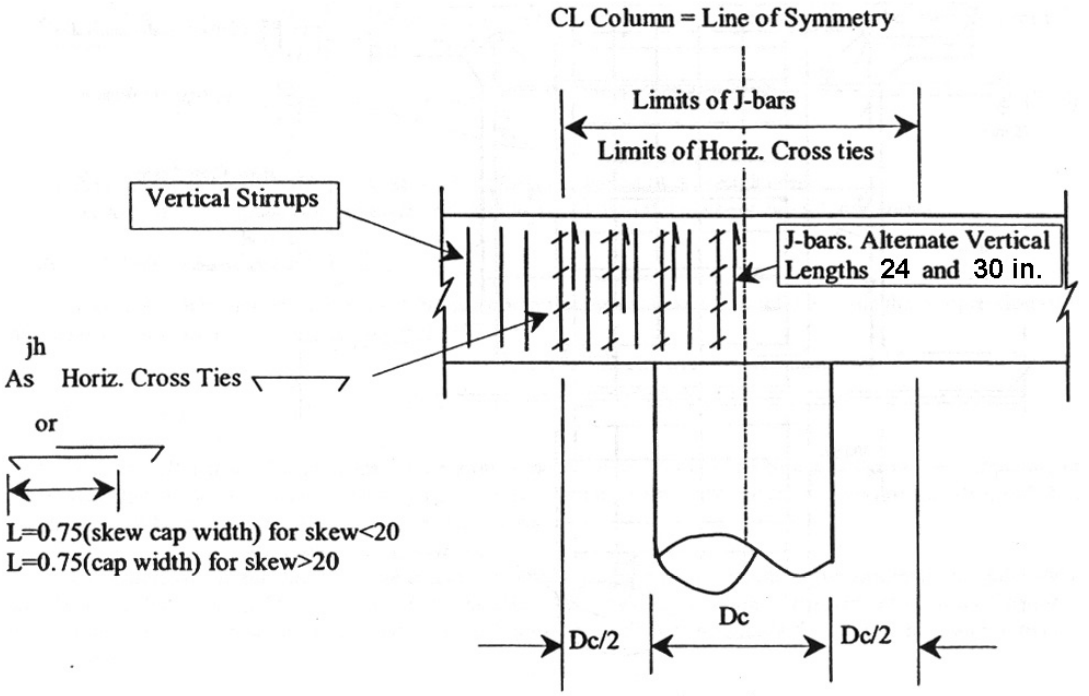

Figure 8.13.4.2.1-3—Location of Horizontal Joint Shear Reinforcement

8.13.4.2.2—Horizontal Stirrups

Horizontal stirrups or ties shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 in. The horizontal reinforcement, A_s^{jh}, shall be placed within a distance D_c from each side of the column centerline as shown in Figure 8.13.4.2.1-3.

$$A_s^{jh} \geq 0.10 A_{st} \quad (8.13.4.2.2\text{-}1)$$

where:

A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

8.13.4.2.3—Horizontal Side Reinforcement

The total longitudinal side face reinforcement in the bent cap shall be at least equal to the greater of the areas specified in Eq. 1 and shall be placed near the side faces of the bent cap with a maximum spacing of 12 in. as shown in Figure 8.13.4.2.1-2. Any side reinforcement placed to meet other requirements shall count toward meeting the requirement of this Article.

$$A_s^{sf} \geq \max \begin{cases} 0.10 A_{cap}^{top} \\ \\ 0.10 A_{cap}^{bot} \end{cases} \quad (8.13.4.2.3\text{-}1)$$

where:

A_s^{sf} = area of longitudinal side reinforcement in the bent cap (in.^{2})

A_{cap}^{top} = area of bent cap top flexural steel (in.^{2})

A_{cap}^{bot} = area of bent cap bottom flexural steel (in.^{2})

8.13.4.2.4— J-Bars

For integral cap of bents skewed greater than 20°, vertical J-bars hooked around the longitudinal top deck steel extending alternatively 24 in. and 30 in. into the bent cap shall be specified. The J-dowel reinforcement shall satisfy:

$$A_s^{j-bar} \geq 0.08 A_{st} \qquad (8.13.4.2.4\text{-}1)$$

The J-bars shall be placed within a rectangular region defined by the width of the bent cap and the distance D_c on either side of the centerline of the column. (See Figure 1 and Figure 8.13.4.2.1-3.)

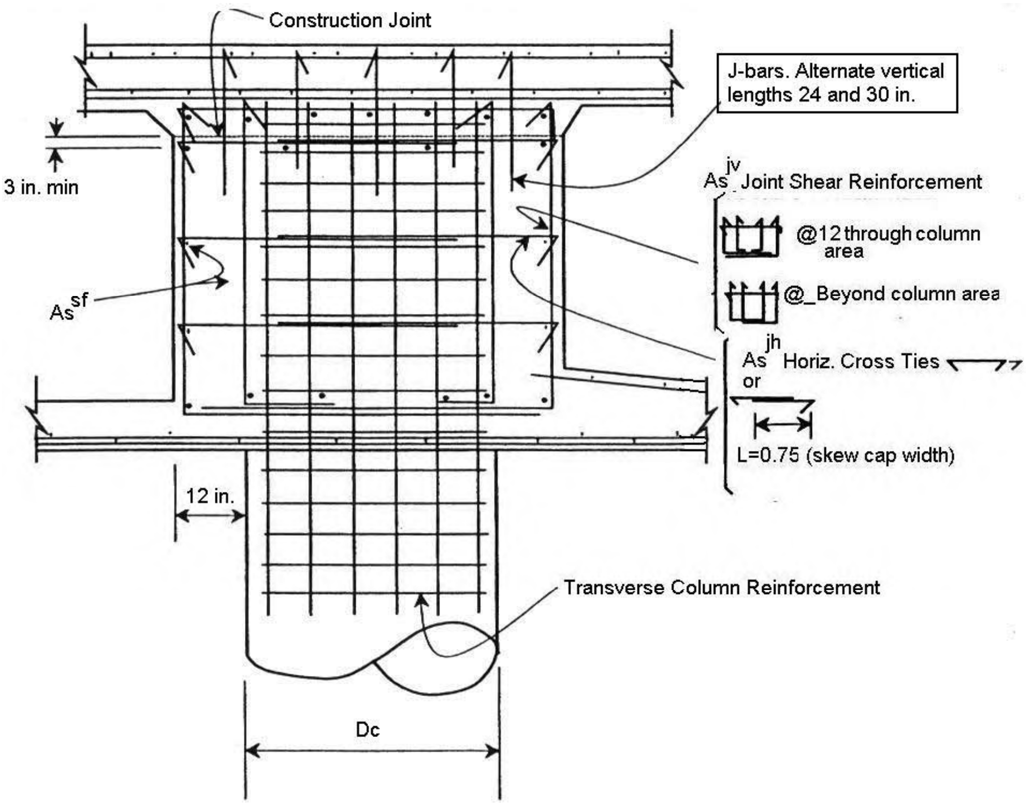

Figure 8.13.4.2.4-1—Additional Joint Shear Steel for Skewed Bridges

8.13.5—Nonintegral Bent Cap Joint Shear Design

Bent cap beams satisfying Eq. 1 shall be reinforced in accordance with the requirements of Article 8.13.5.1. Bent cap beams not satisfying Eq. 1 shall be designed on the basis of the strut and tie provisions of the *AASHTO LRFD Bridge Design Specifications* and as approved by the Owner.

$$D_c \leq d \leq 1.25 D_c \qquad (8.13.5\text{-}1)$$

where:

D_c = column diameter (in.)

d = total depth of the bent cap beam (in.)

8.13.5.1—Joint Shear Reinforcement

8.13.5.1.1 Vertical Stirrups Outside the Joint Region

Vertical stirrups with a total area, A_s^{jvo}, provided to each side of the column shall satisfy:

$$A_s^{jvo} \geq 0.175 A_{st} \qquad (8.13.5.1.1\text{-}1)$$

where:

A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

Vertical stirrups or ties shall be placed transversely within a distance equal to the column diameter, D_c, extending from each face of the column as shown in Figure 1 and Figure 2. The area of these stirrups shall not be used to meet other requirements such as shear in the bent cap.

C8.13.5

Beam–column joints shall be designed to accommodate the forces associated with the column's overstrength plastic hinging moment capacity in an essentially elastic manner.

The design of nonintegral bent cap bridge joints is summarized in Sritharan (2005).

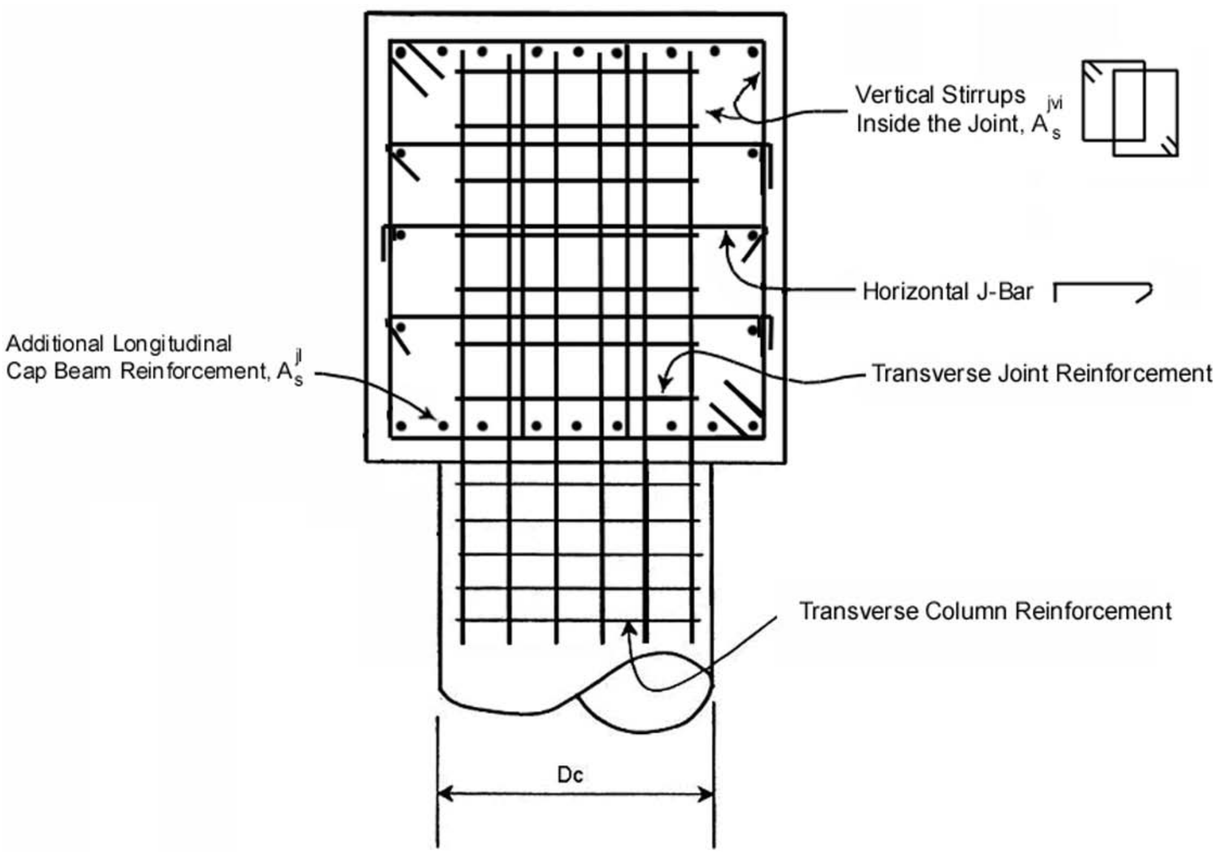

Figure 8.13.5.1.1-1—Joint Shear Reinforcement Details

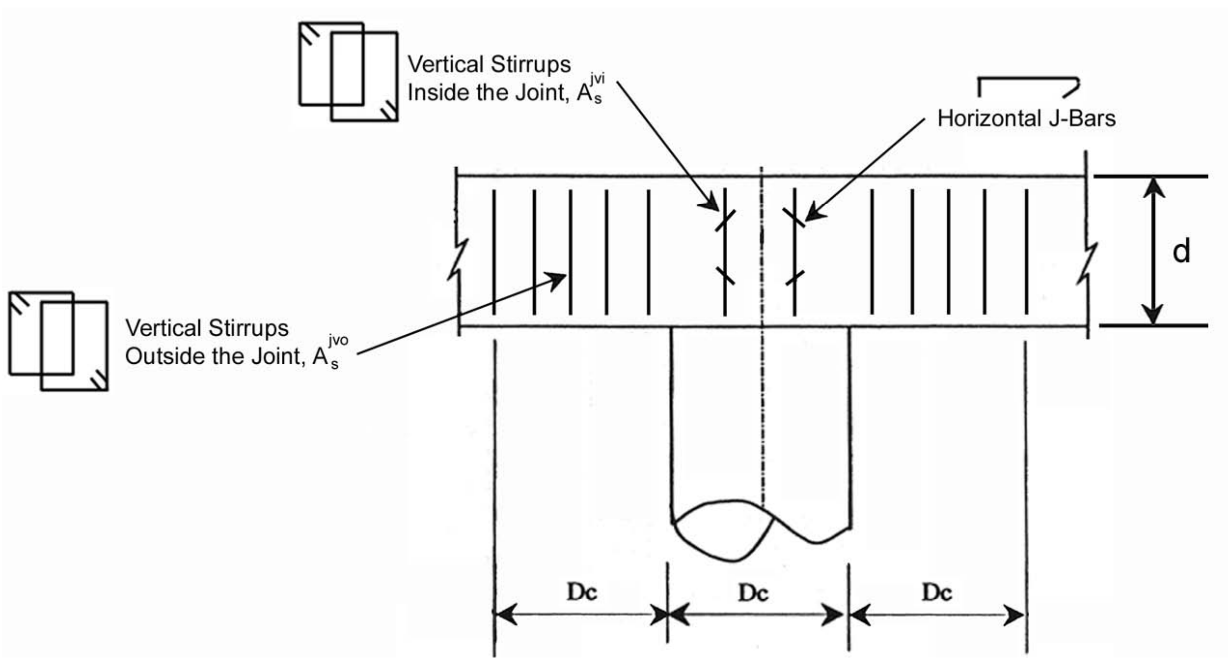

Figure 8.13.5.1.1-2—Location of Vertical Joint Shear Reinforcement

8.13.5.1.2—Vertical Stirrups Inside the Joint Region

Vertical stirrups with a total area, A_s^{jvi}, spaced evenly over the column shall satisfy:

$$A_s^{jvi} \geq 0.135 A_{st} \qquad (8.13.5.1.2\text{-}1)$$

where:

A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

8.13.5.1.3—Additional Longitudinal Cap Beam Reinforcement

Longitudinal reinforcement, $A_s^{j\ell}$, in both the top and bottom faces of the cap beam shall be provided in addition to that required to resist other loads. The additional area of the longitudinal steel shall satisfy:

$$A_s^{j\ell} \geq 0.245 A_{st} \qquad (8.13.5.1.3\text{-}1)$$

where:

A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

8.13.5.1.4—Horizontal J-Bars

Horizontal J-bars hooked around the longitudinal reinforcement on each face of the cap beam shall be provided as shown in Figure 8.13.5.1.1-1. At a minimum, horizontal J-bars shall be located at every other vertical-to-longitudinal bar intersection within the joint. The J-dowel reinforcement bar shall be at least a #4 size bar.

8.14—COLUMN FLARES FOR SDCS C AND D

8.14.1—Horizontally Isolated Flares

The preferred method for detailing flares should be to horizontally isolate the top of flared sections from the soffit of the cap beam, which allows the flexural hinge to form at the top of the column, thus minimizing the seismic shear demand on the column.

A horizontal gap isolating the flare from the cap beam shall extend over the entire cross-section of the flare, excluding a core region equivalent to the prismatic column cross-section. For SDC C, a minimum gap thickness of 4 in. shall be used.

For SDC D, the gap shall be large enough so that it will not close during a seismic event. The gap thickness shall be the largest of:

- 1.5 times the calculated plastic rotation demand from the pushover analysis times the distance from the center of the column to the extreme edge of the flare, or
- 4 in.

The added mass and stiffness of the isolated flare may typically be ignored in the dynamic analysis.

8.14.2—Integral Column Flares

Column flares that are integrally connected to the bent cap should be avoided whenever possible. Lightly reinforced integral flares should be used only when required for service load design or aesthetic considerations and are permitted for SDCs A and B. The flare geometry should be kept as slender as possible.

The higher plastic hinging forces shall be considered in the design of the column, superstructure, and footing.

C8.14.2

Test results have shown that slender, lightly reinforced flares perform adequately after cracking has developed in the flare concrete, essentially separating the flare from the confined column core. However, integral flares require higher shear forces and moments to form the plastic hinge at the top of the column compared with isolated flares.

8.14.3—Flare Reinforcement

Column flares shall be nominally reinforced outside the confined column core to prevent the flare concrete from completely separating from the column at high ductility levels.

The reinforcement ratio for the transverse reinforcement, outside of the column core, that confines the flared region shall be 0.0045 for the upper third of the flare and 0.00075 for the bottom two-thirds of the flare.

The minimum longitudinal reinforcement within the flare shall be equivalent to #5 bars at 12-in. spacing.

8.15—COLUMN SHEAR KEY DESIGN FOR SDCS C AND D

Column shear keys shall be designed for the axial and shear forces associated with the column's overstrength moment capacity, M_{po}, including the effects of overturning. The key reinforcement shall be located as close to the center of the column as possible to minimize developing a force couple within the key reinforcement.

Steel pipe sections may be used in lieu of reinforcing steel to relieve congestion and reduce the moment generated within the key.

Moment generated by the key reinforcing steel should be considered in applying capacity design principles.

8.16—CONCRETE PILES

8.16.1—Transverse Reinforcement Requirements

For SDC C or D where piles are not designed as capacity-protected members (i.e., piles, pile shafts, pile extensions where plastic hinging is allowed in soft soil E or F, liquefaction case), the upper portion of every pile shall be reinforced and confined as a potential plastic hinge region as specified in Article 4.11.

Spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at a pitch not exceeding 9.0 in., except that a 3.0-in. pitch shall be used within a confinement length of not less than 4.0 ft below the pile cap reinforcement. For cast-in-place piles, the 3.0-in. pitch limit may be expanded to 4.0 in.

The shear reinforcement requirements specified in Article 8.6 shall apply. If an analysis of the bridge and pile system indicates that a plastic hinge can form at a lower level, the plastic hinge region shall extend $3D$ above and below the point of maximum moment, and the requirements mentioned above shall apply.

Where piles are part of the main energy dissipation ERS, as in pile bents, the zone of expected plastic hinging shall be identified and transverse steel for shear and confinement provided accordingly.

C8.16.1

Cast-in-place concrete pilings may only have been vibrated directly beneath the pile cap, or in the uppermost sections. Where concrete is not vibrated, nondestructive tests in the State of California have shown that voids and rock pockets form when adhering to maximum confinement steel spacing limitations from some seismic recommendations. Concrete does not readily flow through the resulting clear distances between bar reinforcing, weakening the concrete section, and compromising the bending resistance to lateral seismic loads. Instead of reduced bar spacing, bar diameters should be increased, which results in larger openings between the parallel longitudinal and transverse reinforcing steel.

Use of the shear provisions in Article 8.6 for pile bents with prestressed piling is generally conservative, because the effect of the prestressing compressive force on the shear capacity is neglected. This is also appropriate, because a length of pile at the top does not have this force present due to the development of the strand.

If dowels are used to connect the pile to the cap, then a conventionally reinforced section will exist at the top of the pile and the requirements for such sections apply.

The amount of transverse reinforcement in prestressed concrete piles for SDC D should be at least that required to meet the required displacement capacity and shear demands. In SDC D, moment-curvature relationships will be developed to calculate the inelastic deformation capacity of such piles.

The amount of prescriptive transverse reinforcement calculated by the *AASHTO LRFD Bridge Design Specifications*, Eq. 5.7.4.6-1, when used for piling, is often so large that fabrication becomes difficult. The problem is rooted in the ratio of A_g/A_c in the equation. This quantity is often unfavorable in smaller members because the cover concrete is a large portion of the gross area. The Precast/Prestressed Concrete Institute (PCI) provides a summary of this issue in Chapter 20 of their *Bridge Design Manual* (PCI, 2004). Research is currently underway to improve design recommendations for transverse steel in prestressed concrete piling.

8.16.2—Cast-in-Place and Precast Concrete Piles

For cast-in-place and precast concrete piles, longitudinal steel shall be provided for the full length of the pile. In the upper two-thirds of the pile, the longitudinal steel ratio shall not be less than 0.007. Longitudinal reinforcement shall be provided by no fewer than four bars.

For piles in which a permanent steel casing is used, the extent of longitudinal reinforcement may be reduced to only the upper portion of the pile required to develop ultimate tension and compression capacities of the pile.

C8.16.2

Connection of piles using embedment of prestressing strand into cap beams of pile bents is not permitted in SDCs C and D.

Use of hollow prestressed piles in plastic hinging zones is not permitted in SDCs C and D unless the interior cavity is filled with concrete.

Strand developed into a member adjacent to a plastic hinging zone will permit large cracks to form at the interface due to slip.

Hollow prestressed piles will spall into the cavity under plastic hinging even if confinement steel is used. This behavior is prevented if the cavity is filled with concrete.

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APPENDIX A: FOUNDATION-ROCKING ANALYSIS

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APPENDIX A:

FOUNDATION-ROCKING ANALYSIS

A.1—ANALYSIS

A design strategy based on transient foundation uplift, or foundation rocking, involving separation of the foundation from the subsoil, may be permitted under seismic loading, provided that foundation soils are not susceptible to loss of strength under the imposed cyclic loading. The displacement, or drift Δ, as shown in Figure A-1, shall be calculated on the basis of the flexibility of the column in addition to the effect of the footing rocking mechanism. For multicolumn bents with monolithic connections to the substructure, the effect of rocking shall be examined on the overturning and framing configuration of the subject bent.

For the longitudinal response, multicolumn bents that are not monolithic to the superstructure shall be treated similar to a single-column bent.

Rocking displacement demands shall be calculated with due consideration of the dynamics of the bridge system or frame. The tributary inertial weight and articulation and/or restraint of other elements of the frame shall be incorporated into the analysis. The following equations were derived for an individual single column bent. Engineering judgment is required when employing these equations in situations that may be considered sufficiently analogous to that of a single column bent.

For the case of a single-column bent or a multicolumn bent without a monolithic connection to the superstructure, the footing should be considered to be supported on a rigid perfectly plastic soil with uniform compressive capacity q_n. The overturning and rocking on the foundation may be simplified using a linear force-deflection relationship as outlined in the following procedure:

- Assume a value for the displacement Δ or consider a displacement Δ corresponding to a fixed base analysis.
- Calculate the applied force F at the superstructure level on the basis of rocking equilibrium shown in Figure A-1.

From statics:

$$F = W_T \frac{(L_F - a)}{2H_r} - W_s \frac{\Delta}{H_r} \tag{A-1}$$

in which:

$$a = \frac{W_T}{(B_r q_n)} \tag{A-2}$$

- Calculate the equivalent system stiffness:

$$K_r = \frac{F}{\Delta} \tag{A-3}$$

- Calculate the period "T" of the bent system based on K_r and $W_s + {}^{1}/_{2}\,(W_{COLUMN})$. For a single degree of freedom system, T may be calculated as follows:

$$T = 2\pi\sqrt{\frac{W_s + \frac{1}{2}(W_{COLUMN})}{gK_r}} \tag{A-4}$$

- Recalculate Δ considering ten percent damping; this would typically reduce the spectral acceleration ordinates S_a of a five percent damped spectrum by approximately 20 percent, which is reflected by the 0.8 factor given in the following:

$$\Delta = \left(\frac{T^2}{4\pi^2}\right)(0.8S_a) \tag{A-5}$$

where:

Δ = the total displacement on top of the column (ft)

S_a = the spectral acceleration (ft/s^2)

- Iterate until convergence; otherwise, the bent is shown to be unstable.
- Once a convergent solution is reached, the local ductility term μ can be calculated to ensure the column adequacy where rocking mechanism is not mobilized:

$$\mu = \frac{\Delta}{\Delta_{ycol}} \tag{A-6}$$

where:

Δ_{ycol} = column idealized yield displacement

For soil cover greater than 3 ft, the effect of soil passive resistance should be included in the rocking equilibrium of forces.

The design of a column on spread footing systems shall follow the steps identified on the flowchart shown on Figure A-2.

The restoring moment M_r shall be calculated as follows:

$$M_r = W_T\left(\frac{L_F - a}{2}\right) \tag{A-7}$$

For the case in which $M_o \geq 1.5M_r$, the column shear capacity shall be determined on the basis of Article 8.6, following SDC B requirements. The column shear demand shall be determined on the basis of $1.5M_r$ moment demand.

For the case in which $M_r \geq M_o$, forces based on column plastic hinging shall be considered, the column shear capacity shall be determined on the basis of Article 8.6, following SDC D requirements. For all other cases ($M_r < M_o < 1.5M_r$), the column shall be designed for P-Δ requirements on the basis of rocking analysis as well as column plastic hinging shear capacity requirements considering a fixed-based analysis and following Article 8.6 SDC C requirements.

The shear component of loading should not be included during the overturning check, that is, a decoupled approach should be used in treating the two loads. Experience has shown that combining the horizontal load and moment in simplified bearing capacity equations can result in unreasonably sized footings for seismic loading.

Unfactored resistance shall be used for the moment capacity check for two reasons: (1) The potential for the design seismic load is very small, and (2) the peak load will occur for only a short duration. The distribution and magnitude of bearing stress, as well as liftoff of the footing, are limited to control settlement of the footing from the cycles of load.

Nontriangular stress distributions of greater than 50 percent liftoff may be used if analysis can show that soil settlement from cyclic shakedown does not exceed amounts that result in damage to the bridge or unacceptable movement of the roadway surface. By limiting stress distribution and the liftoff to the specified criteria, the amount of shakedown will normally be small under normal seismic loading conditions.

This work was derived on the basis of that presented by Priestley et al. (1996).

A.2—FIGURES

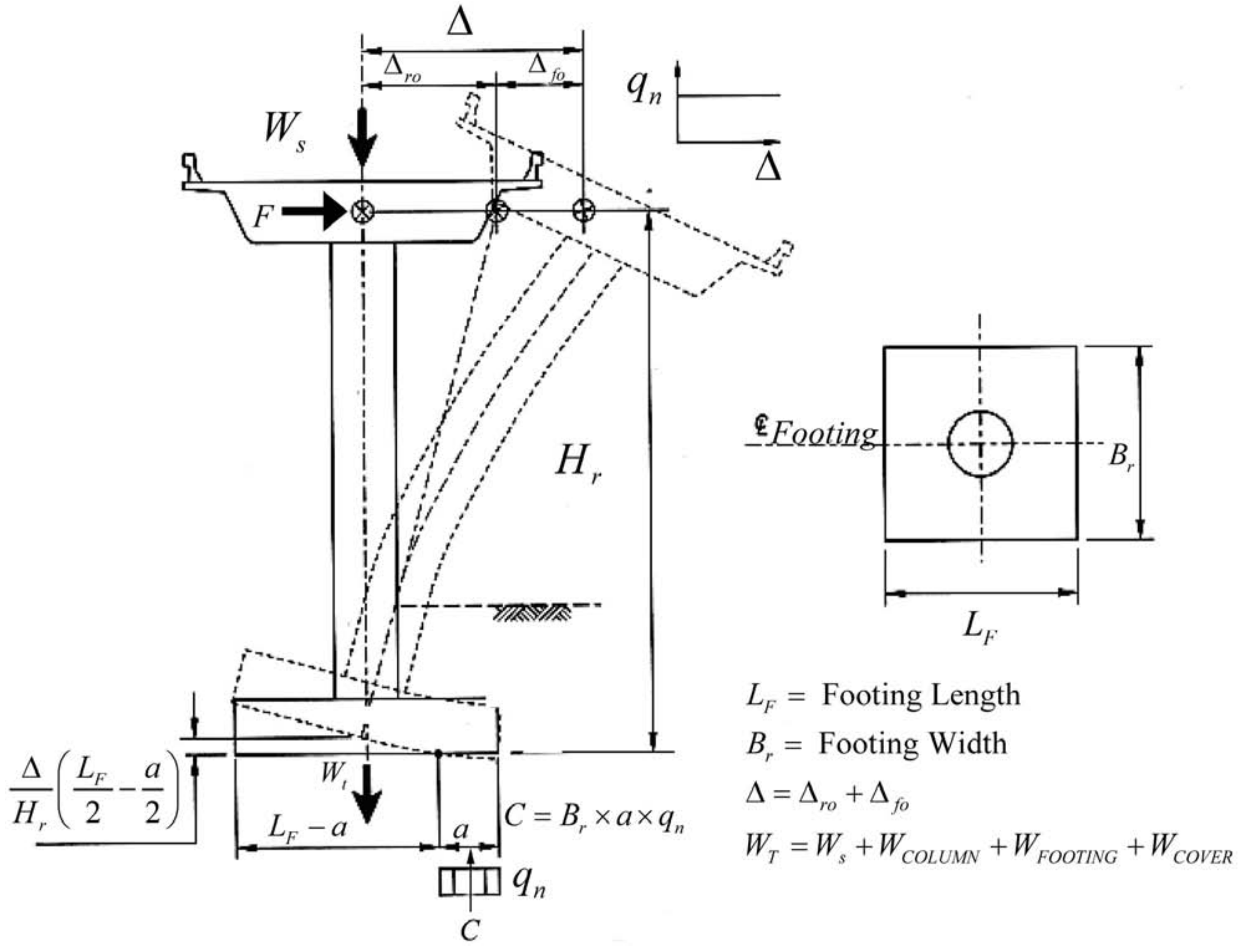

Figure A-1—Rocking Equilibrium of a Single-Column Bent

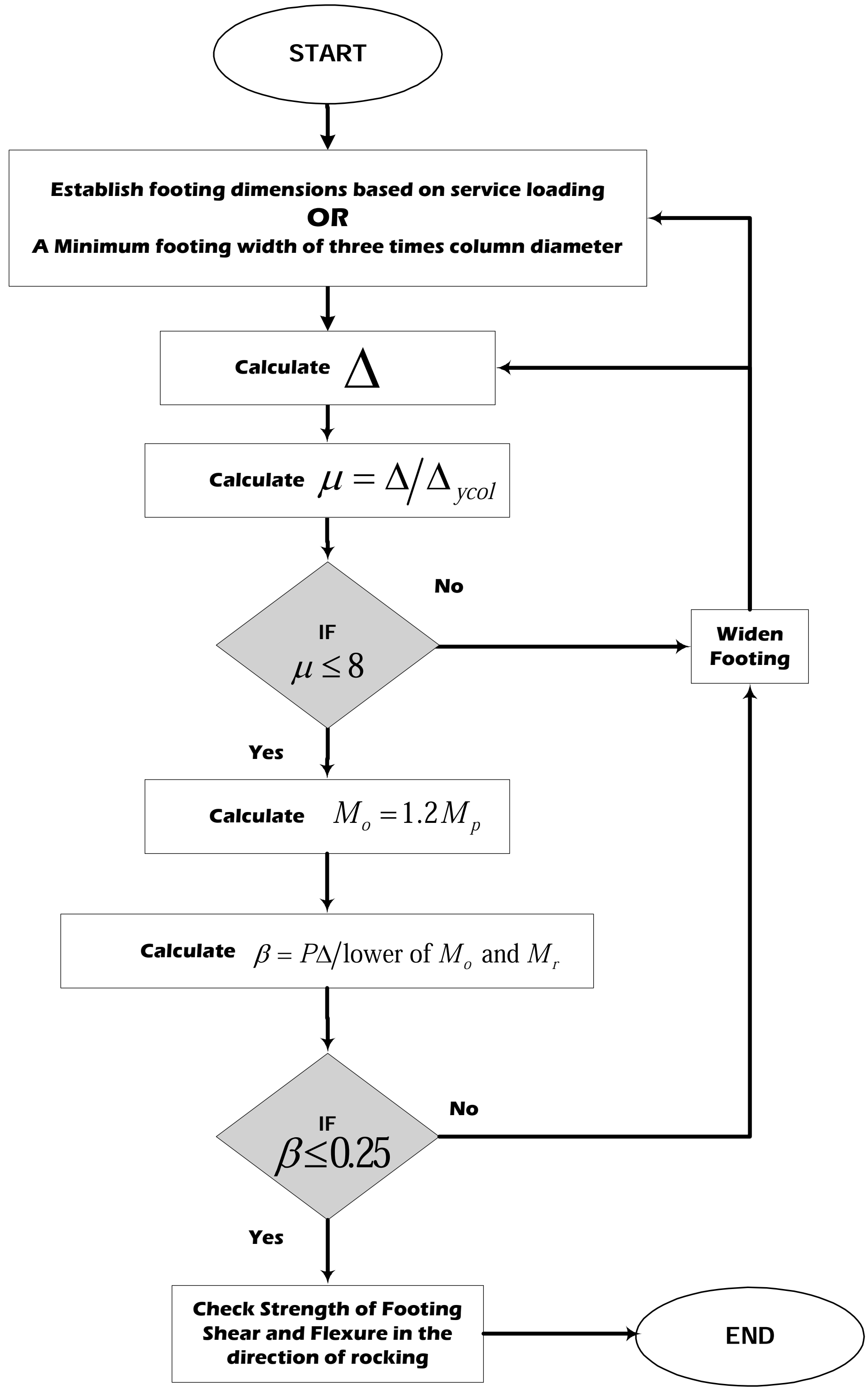

Figure A-2—Flowchart for Design of a Column and Spread Footing Using Rocking Analysis