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AASHTO Guide Specifications for

LRFD Seismic Bridge Design

2nd Edition



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

PREFACE

This second edition of the *Guide Specifications for LRFD Seismic Bridge Design* includes technical content approved by the Highways Subcommittee on Bridges and Structures through 2011. In addition to revising the first edition content, the authors have added Appendix B, “Design Flowcharts.”

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

AASHTO Publications Staff

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SECTION 1: INTRODUCTION

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

SDC A

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

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.

SDC B

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

SDC C

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

SDC D

- o Pushover analysis required
 - o Capacity design required
 - o SDC D level of detailing
 - o Liquefaction assessment required
- Allow for three types of a bridge structural system:
 - o *Type 1*—Design a ductile substructure with an essentially elastic superstructure.
 - o *Type 2*—Design an essentially elastic substructure with a ductile superstructure.
 - o *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 1.3-1 to 1.3-5.

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 1.3-1 guides the designer on the applicability of the Guide Specifications and the seismic design procedure for bridges in SDC A and single-span bridges. Figures 1.3-2 through 1.3-4 show seismic design procedure flowcharts for bridges in SDC B through D respectively. Figure 1.3-5 shows foundation design and detailing flowcharts.

Alternate flowcharts are provided in Appendix B.

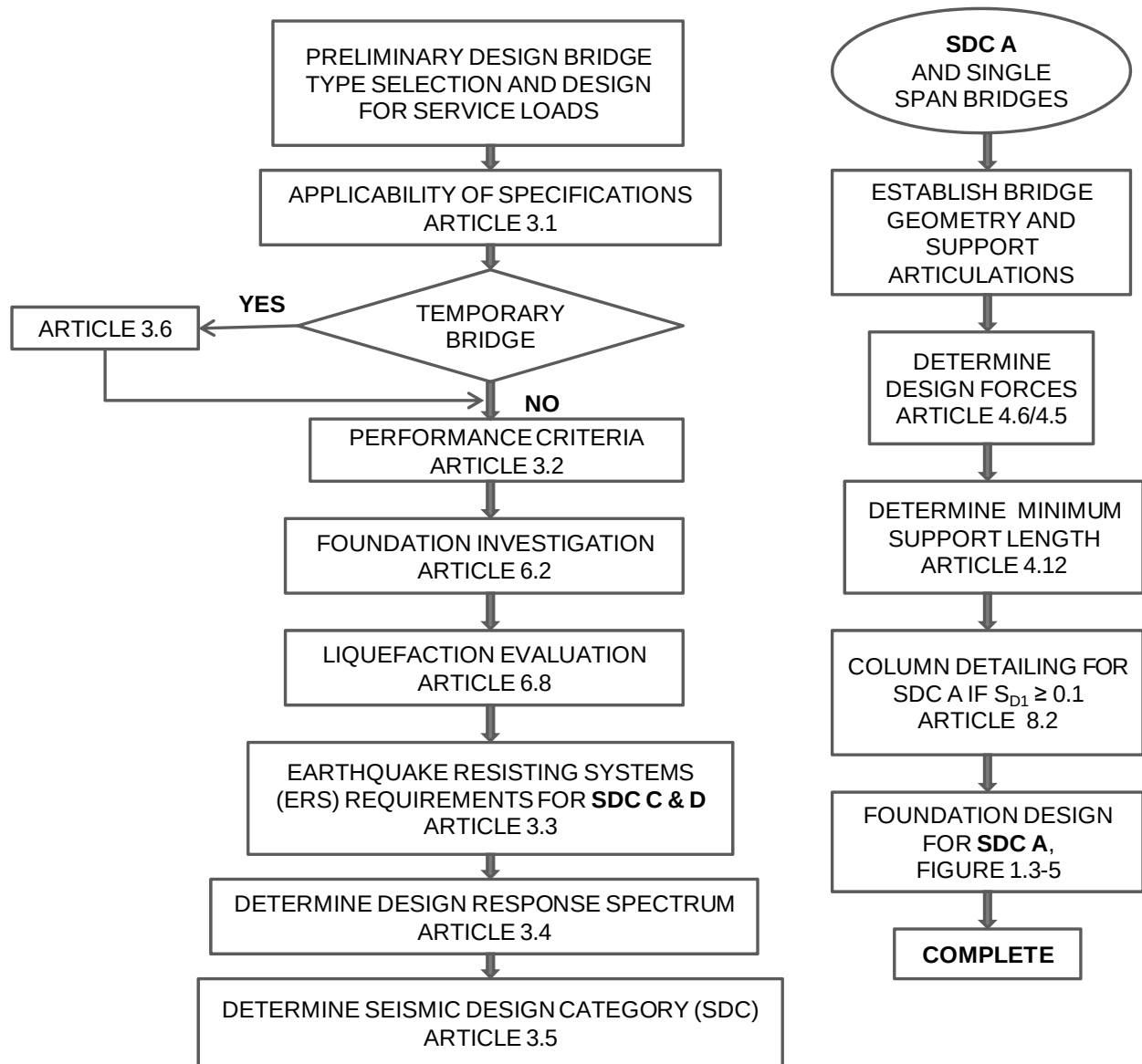


Figure 1.3-1—Applicability of the Guide Specifications and the Seismic Design Procedure for Bridges in SDC A and Single-Span Bridges

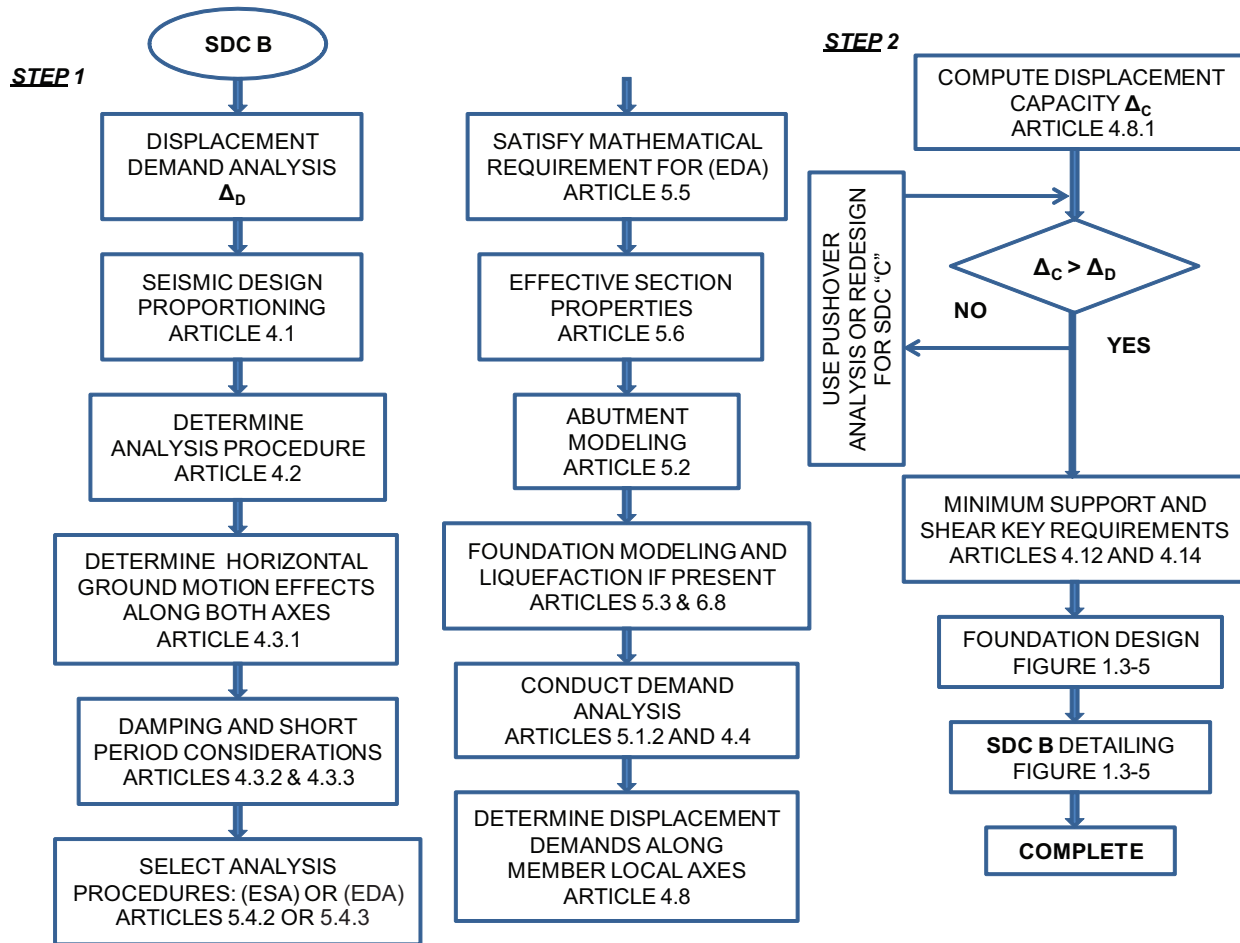


Figure 1.3-2—Seismic Design Procedure Flowchart for Bridges in SDC B

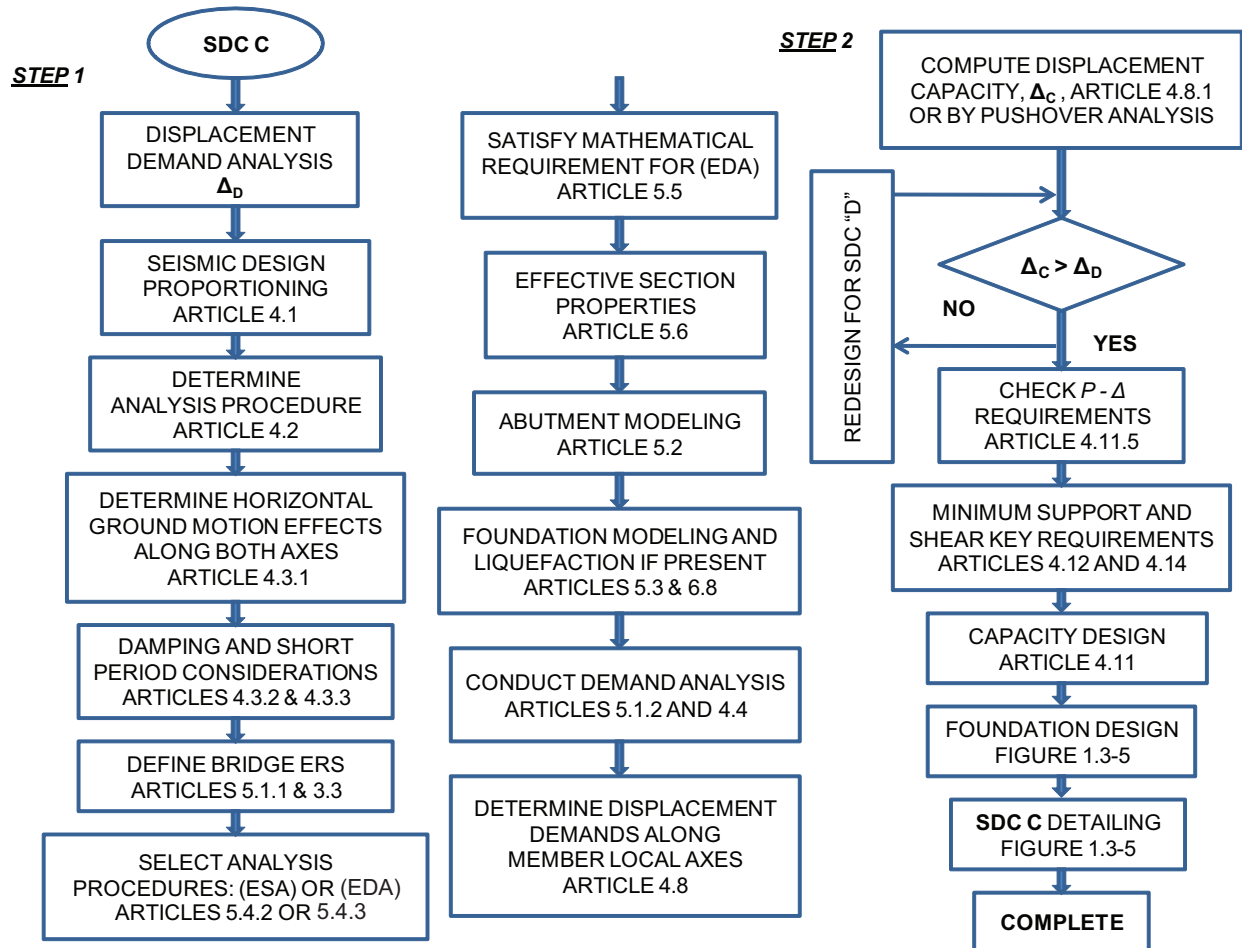


Figure 1.3-3—Seismic Design Procedure Flowchart for Bridges in SDC C

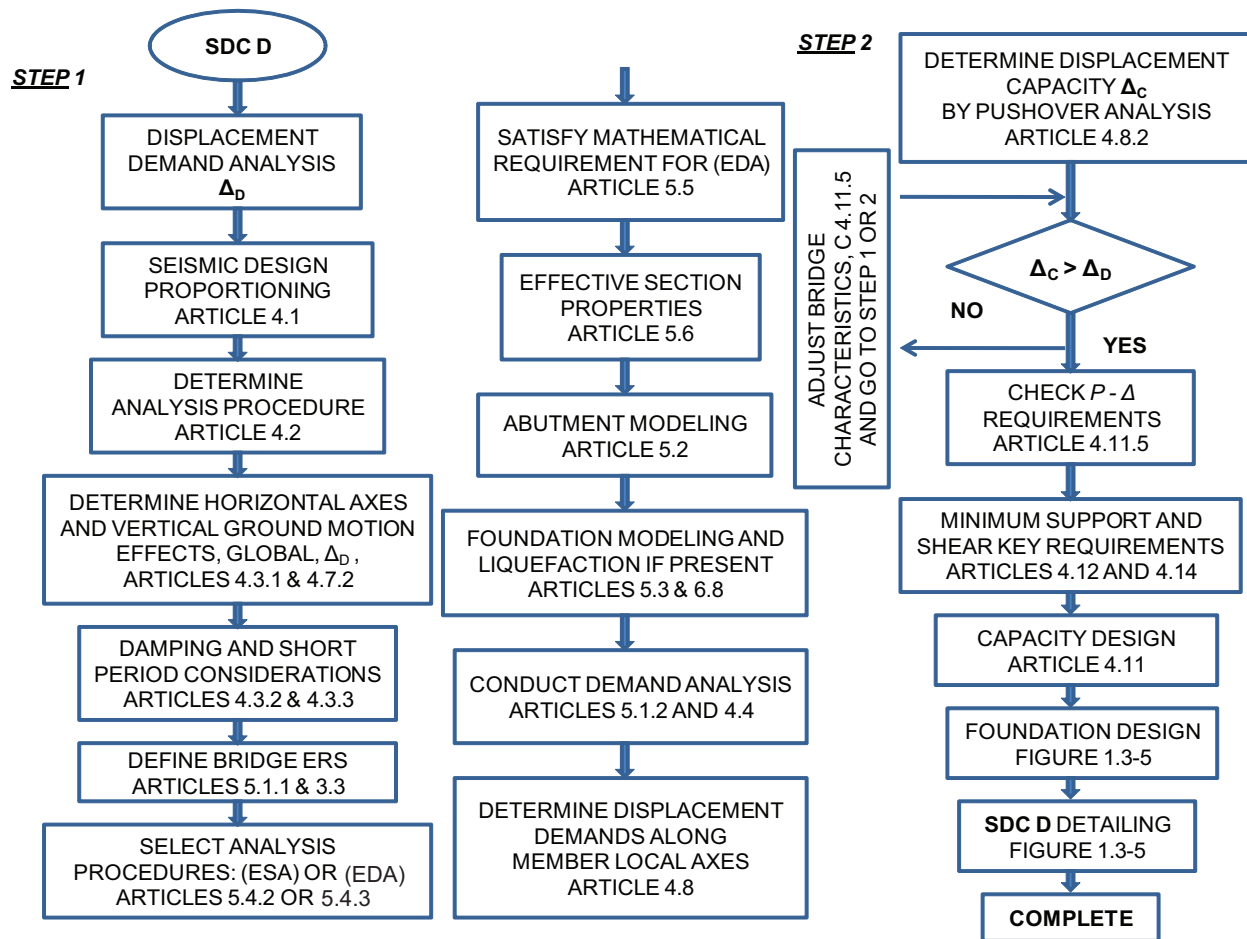


Figure 1.3-4—Seismic Design Procedure Flowchart for Bridges in SDC D

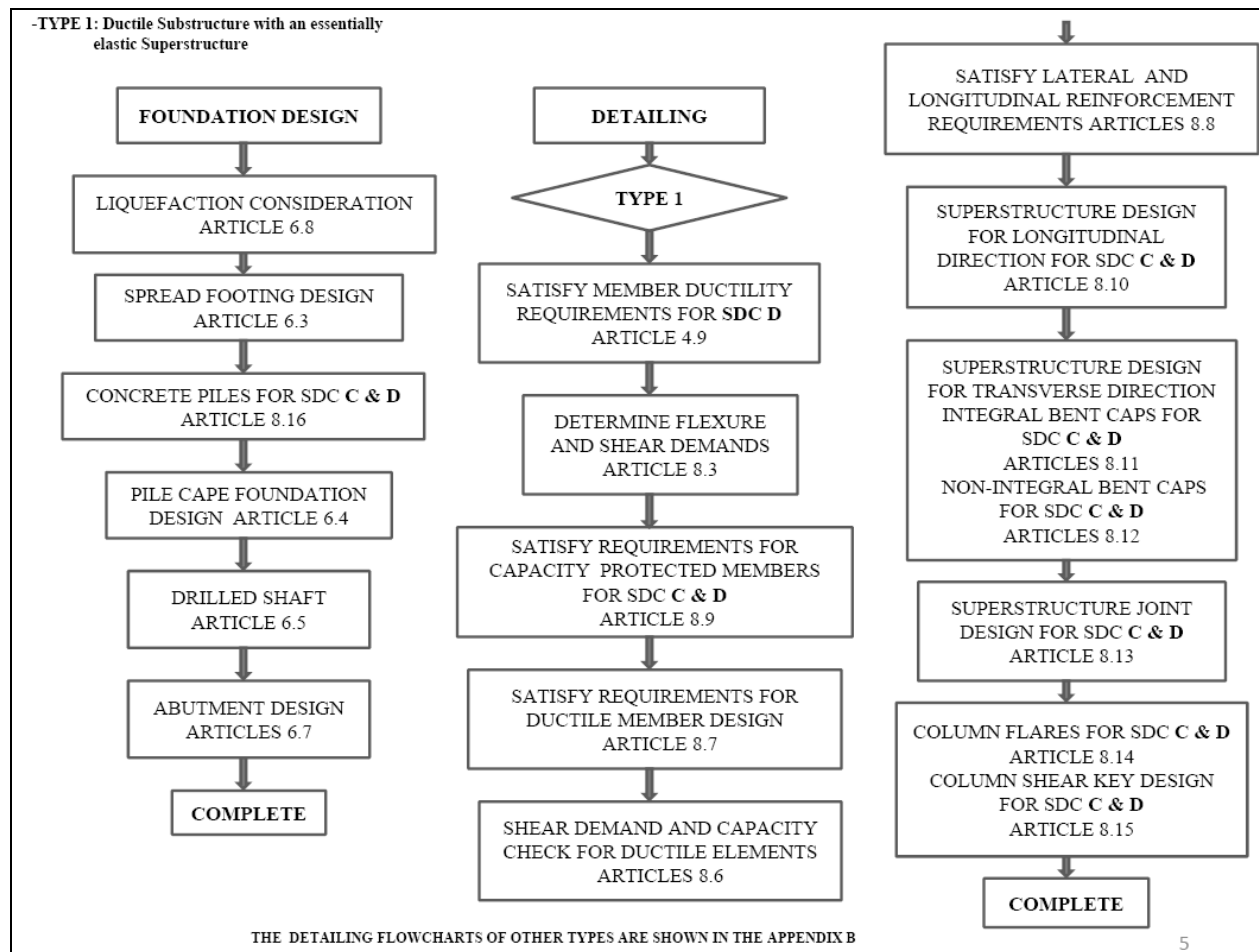


Figure 1.3-5—Foundation and Detailing Flowcharts

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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.

Oversized Pile Shaft—A drilled shaft foundation that is larger in diameter than the supported column and has a reinforcing cage larger than and independent of the column's. The ratio of the size of shaft to that of the supported column which defines a shaft as oversized is at the discretion of the Owner.

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) (8.13.4.1.2c) (8.13.4.2.2c)
A_{cap}^{top}	=	area of bent cap top flexural steel (in. ²) (8.13.4.1.2c) (8.13.4.2.2c)
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}^{fg}	=	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 >20° (in. ²) (8.13.4.1.2d) (8.13.4.2.2f)
A_s^{jh}	=	cross-sectional area of horizontal stirrups required at moment resisting joints (in. ²) (8.13.4.1.2b) (8.13.4.2.2b)
A_s^{jhc}	=	area of required horizontal cap end ties (in. ²) (8.13.4.2.2e)
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) (8.13.4.1.2a) (8.13.4.2.2a)
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}	=	area of longitudinal side reinforcement in the bent cap (in. ²) (8.13.4.1.2c) (8.13.4.2.2c)
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) (8.13.2) (8.13.3) (8.13.4.1.2a) (8.13.4.1.2b) (8.13.4.1.2d) (8.13.4.2.2a) (8.13.4.2.2b) (8.13.4.2.2g) (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_s^{U-bar}	=	area of U-shaped top and bottom longitudinal reinforcement required at moment resisting joints (in. ²) (8.13.4.2.2d) (8.13.4.2.2e)
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_{vf}	=	interface shear reinforcement (in. ²) (8.15)
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) (8.13.2)
B_{cap}	=	thickness or width 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.); effective width of the bent cap joint (in.) (8.10) (8.11) (8.13.2)
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 i th pile (kip) (6.4.2)
c	=	soil cohesion (psf or ksf) (6.2.2)
$c_{x(i)}$	=	distance from neutral axis of pile group to i th row of piles measured parallel to the y axis (ft) (6.4.2)
$c_{y(i)}$	=	distance from neutral axis of pile group to i th 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'	=	core diameter of column measured from center 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.); diameter of pile shaft (in.) (6.3.2) (C6.3.6) (6.4.7) (8.8.6) (8.10) (8.13.2) (8.13.4.1.2d) (8.13.4.2.1) (8.13.4.2.2g) (8.13.5) (8.13.5.1.3) (8.6.1)
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_{fig}	=	depth of the pile cap or footing (ft or in.) (6.4.2) (6.4.5) (6.4.7)
D_g	=	width of gap between backwall and superstructure (ft) (5.2.3.3)
D_s	=	depth of superstructure at the bent cap or depth of cap beam (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_{bl}	=	nominal diameter of longitudinal column reinforcing steel bars (in.) (4.11.6) (8.8.4) (8.8.6)
d_i	=	thickness of i th 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)
$E_c I_{eff}$	=	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) (6.4.7) (C6.4.7) (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) (8.13.2)
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)
$G_c J_{eff}$	=	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.); length of the pile shaft/column from the ground surface to the point of contraflexure in the column above the ground (in.) (4.11.6) (8.6.1)
H_s	=	length of pile shaft as determined in accordance with Article 8.6.1 (in.) (8.6.1)
h	=	web depth (in.) (7.4.2)
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.) (C4.9) (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.) (C4.9) (4.11.6) (4.11.7) (8.14.1)
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) (8.13.4.2.2g)
ℓ_d	=	bar development length (in.) (8.8.10)
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 $\epsilon_c = 0.003$ (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)
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.); extended minimum support length measured normal to the centerline of bearing (in.) (4.12) (4.12.1) (4.12.2) (4.12.3) (C4.13.1)
$\overline{N}$	=	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 ($^{\circ}$); minimum distance from the exterior girder edge at the soffit to the intersection of the bent centerline and face of the column (in.) (4.12.1) (4.12.2) (8.13.4.2)
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)
$\bar{s}_u$	=	average undrained shear strength in the top 100 ft (psf) (3.4.2)
s_{ui}	=	undrained shear strength of i th 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); net tensile force in column reinforcement associated with the column overstrength plastic hinging moment, M_{po} (kips) (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 i th 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 m th 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 <i>AASHTO LRFD Bridge Design Specifications</i> using the nominal 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 (percent) (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)
α'	=	concrete shear stress adjustment factor (8.6.2)

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_{col}^y	=	yield displacement of the column (in.) (4.8)
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.) (C3.3) (4.8) (4.8.1)
Δ_D	=	global seismic displacement demand (in.) (4.3.1) (4.11.5)
Δ_D^L	=	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) (C4.9)
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) (C4.9)
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)
$\epsilon_{ps,EE}$	=	essentially elastic prestress steel strain (8.4.3)
$\epsilon_{ps,u}$	=	ultimate prestress steel strain (8.4.3)
$\epsilon_{ps,u}^R$	=	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) (C4.9) (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; transverse reinforcement required for Class 1 knee joints (8.6.2) (8.6.5) (8.8.7) (8.13.3) (8.13.4.2.2g)
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)
θ_p	=	plastic rotation determined using Eq. 8.14.1-1 (rad.) (C4.9) (8.14.1)
θ_{pd}	=	plastic rotation determined using Eq. C4.9-1 (rad.) (C4.9)
θ_y	=	yield rotation calculated using the moment-area method (rad.) (8.14.1)
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_{col}	=	column curvature at maximum displacement demand (computed from push over analysis) (1/in.); column curvature at the demand displacement (1/in.) (C4.9) (8.14.1)
ϕ_{pd}	=	column plastic curvature demand (1/in.) (C4.9)
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 (1/in.) (7.7.6) (8.5) (8.14.1)
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) (8.14.1) (7.7.6)
ϕ_{yi}	=	idealized yield curvature (1/in.) (C4.9) (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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GENERAL REQUIREMENTS

3.1—APPLICABILITY OF GUIDE SPECIFICATIONS

C3.1

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.

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 liquefaction, liquefaction-induced lateral flow, or liquefaction-induced lateral spreading, inelastic deformation may be permitted in piles and shafts. Partial or complete replacement of the columns, piles, or shafts may be necessary.

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 exceedance (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 3.3-1a and 3.3-1b, 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 3.3-2. 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 liquefaction, liquefaction-induced lateral flow, or liquefaction-induced lateral spreading. 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 3.3-1a and 3.3-1b 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

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 C3.3-1, this assumes that Δ_C^L is approximately equal to Δ_D^L . Work by Miranda and Bertero (1994) and by Chang and Mander

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 3.3-2 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 3.3-3 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.

(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 current 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.

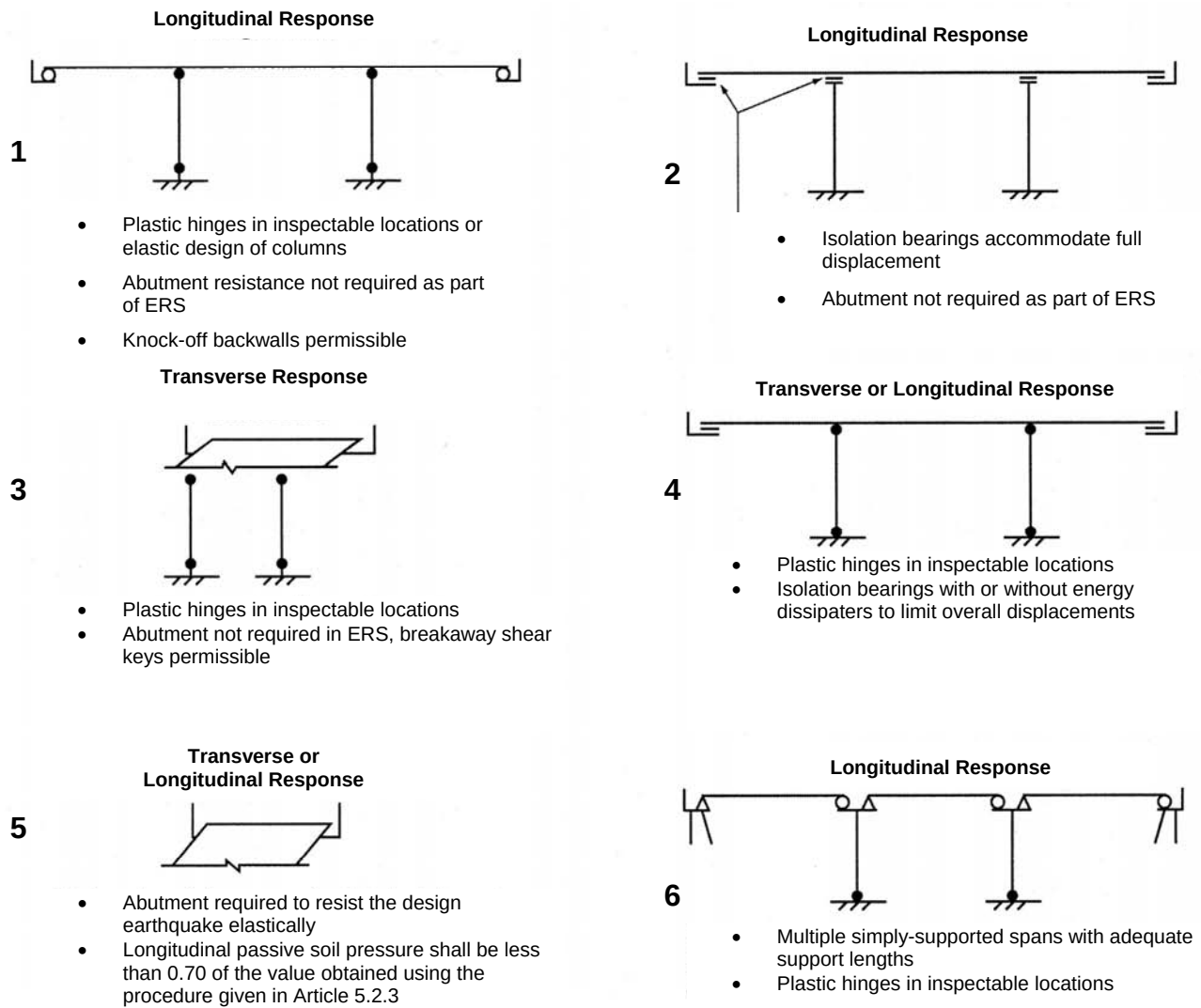


Figure 3.3-1a—Permissible Earthquake-Resisting Systems (ERSs)

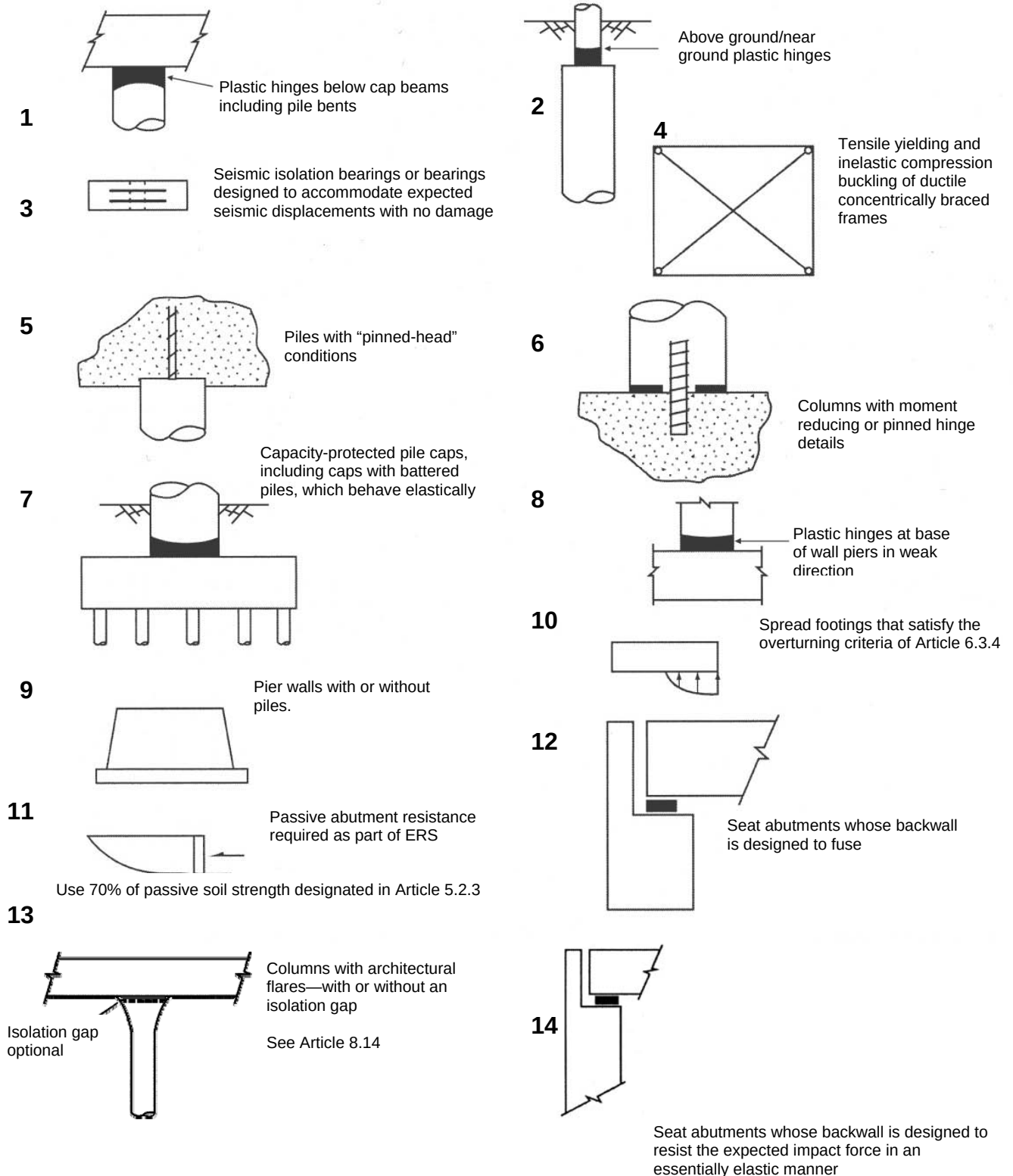


Figure 3.3-1b—Permissible Earthquake-Resisting Elements (EREs)

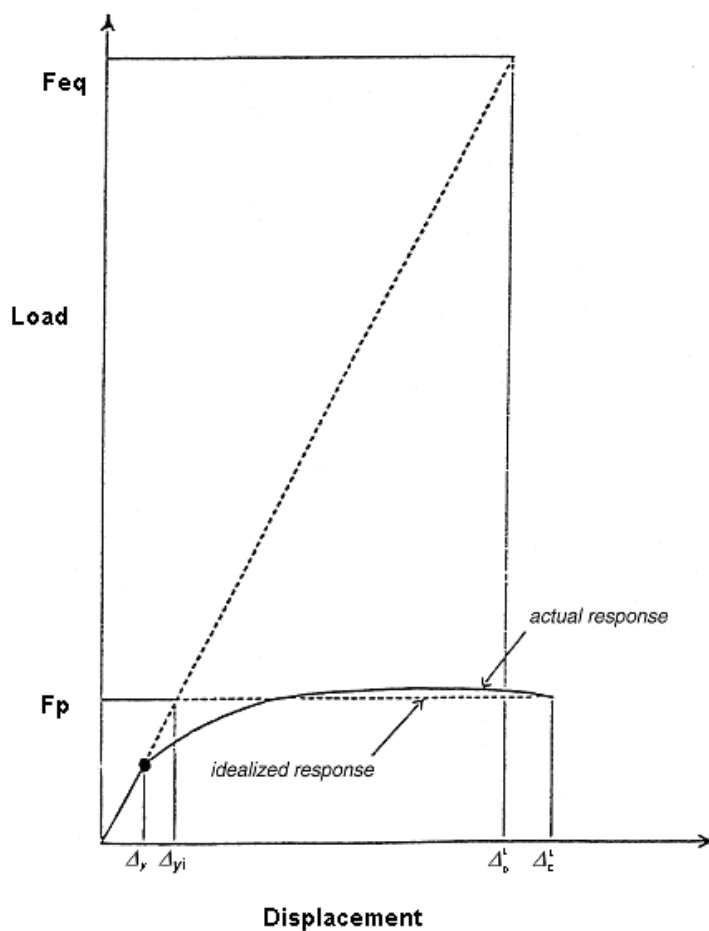


Figure C3.3-1—Design Using Strategy Type 1

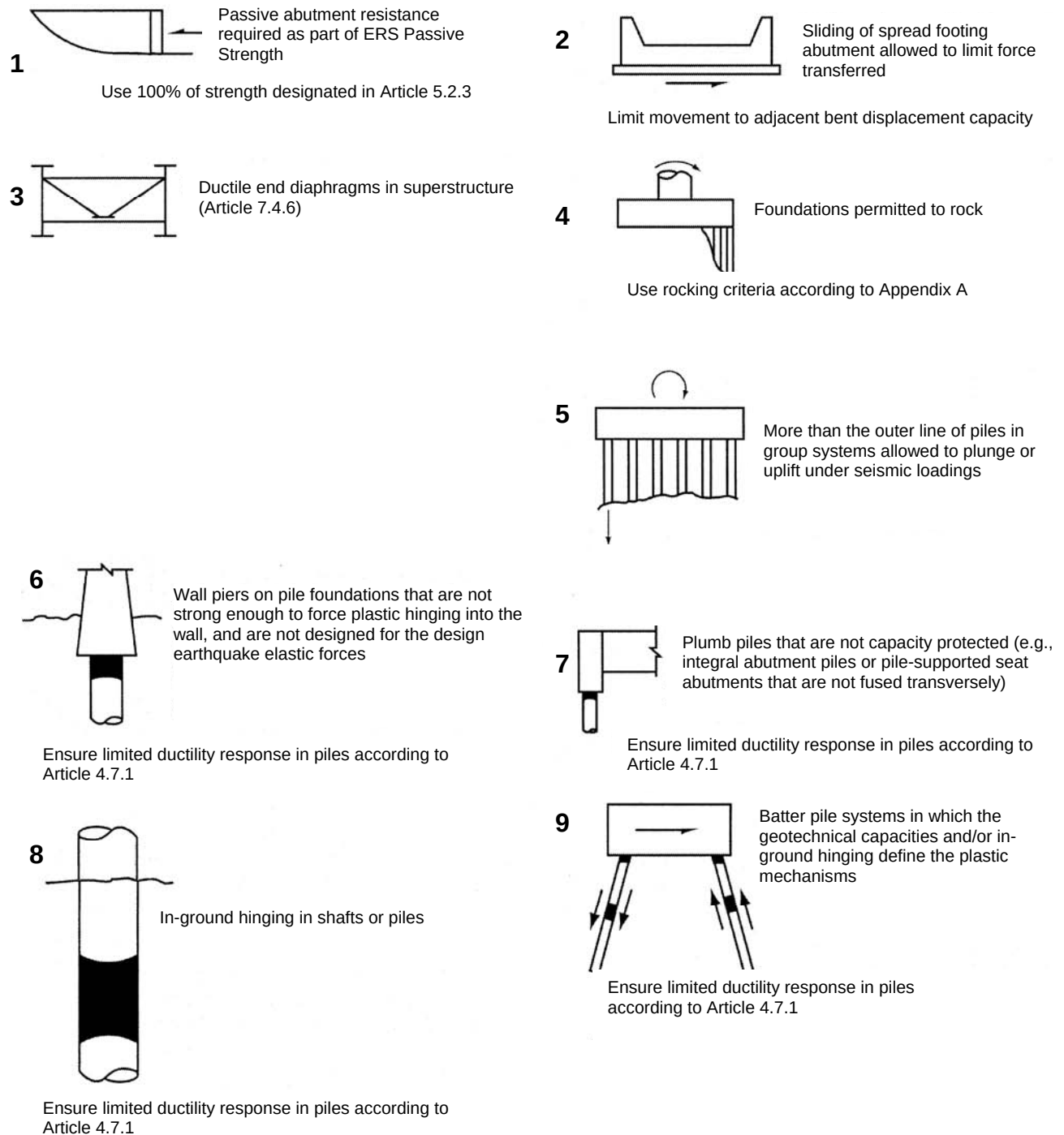


Figure 3.3-2—Permissible Earthquake-Resisting Elements that Require Owner's Approval

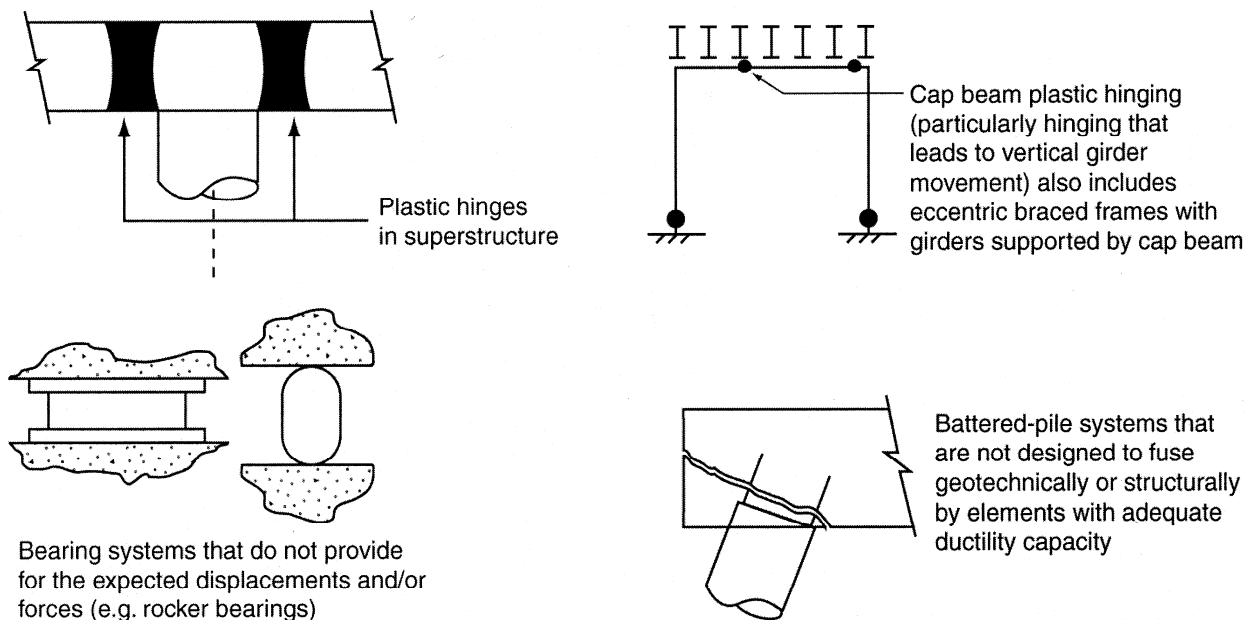


Figure 3.3-3—Earthquake-Resisting Elements that Are Not Recommended for New Bridges

3.4—SEISMIC GROUND SHAKING HAZARD

C3.4

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.

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 feasible combinations of earthquake magnitude, source-to-site distance, and

- 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.

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 3.4.1-1.

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.

An error has been identified in the Spectral Response Accelerations S_{DS} and S_{DI} results produced by the CD-ROM software. Specifically, the A_s value is erroneously calculated as $A_s = F_a PGA$. Although the corrected value for A_s is presented in the tabulated Design Spectrum table, designers

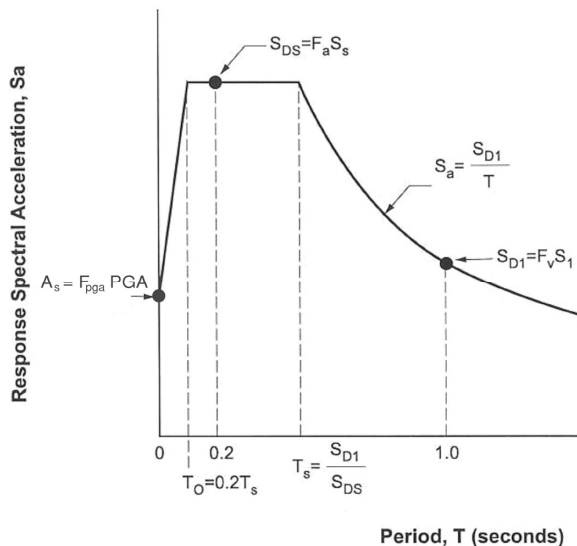


Figure 3.4.1-1—Design Response Spectrum, Construction Using Three-Point Method

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. 3.4.1-1 through 3.4.1-3, respectively:

$$A_s = F_{pga} PGA \quad (3.4.1-1)$$

$$S_{DS} = F_a S_s \quad (3.4.1-2)$$

$$S_{D1} = F_v S_1 \quad (3.4.1-3)$$

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

should be aware of this error until the problem is corrected. The software error will likely have negligible effects on bridge analysis results because:

- F_{pga} is approximately equal to F_a ,
- A_s is properly calculated and displayed in the tabulated design spectra, and
- Bridges have fundamental periods greater than the effected period range ($T < T_o$).

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. 3.4.1-8 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

- 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

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 3.4.1-1:

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

$$S_a = (S_{DS} - A_s) \frac{T}{T_o} + A_s \quad (3.4.1-4)$$

in which:

$$T_o = 0.2T_s \quad (3.4.1-5)$$

$$T_s = \frac{S_{D1}}{S_{DS}} \quad (3.4.1-6)$$

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:

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

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

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

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.

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.

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.

The value of the spectral acceleration parameters (A_s , S_{DS} , and S_{D1}) need not use more than two decimal places.

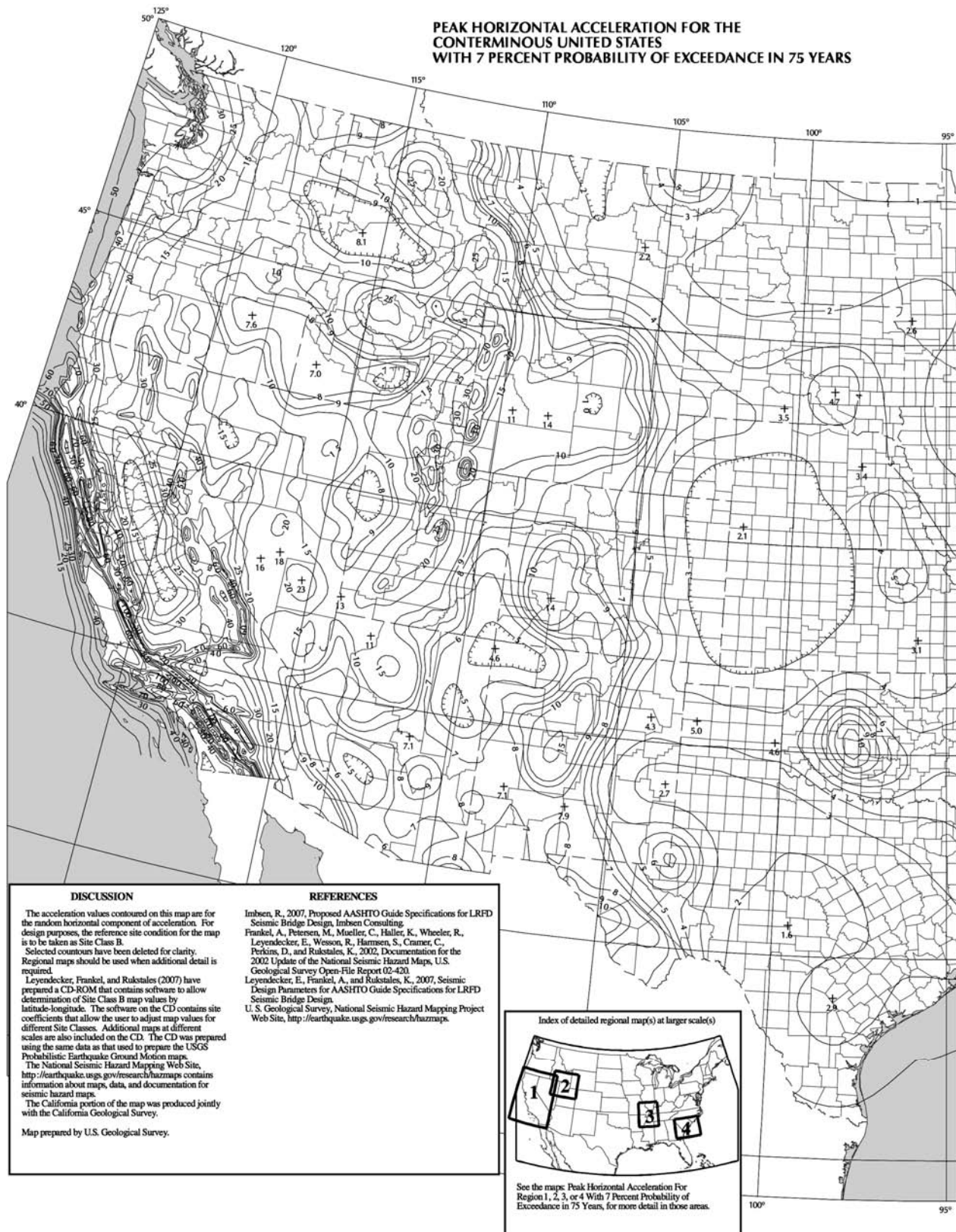


Figure 3.4.1-2a—Horizontal Peak Ground Acceleration Coefficient for the Conterminous United States (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

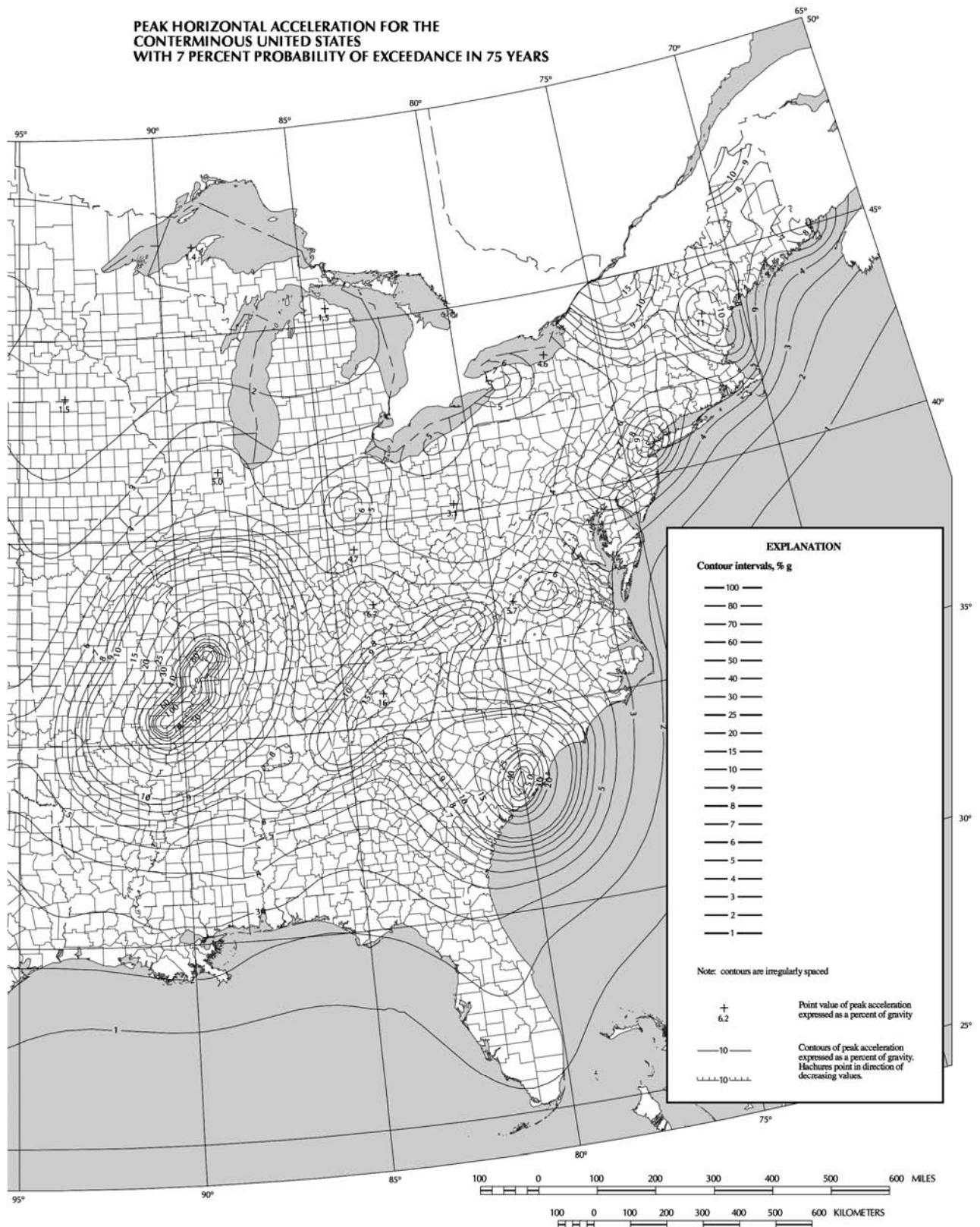


Figure 3.4.1-2b—Horizontal Peak Ground Acceleration Coefficient for the Conterminous United States (PGA) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)



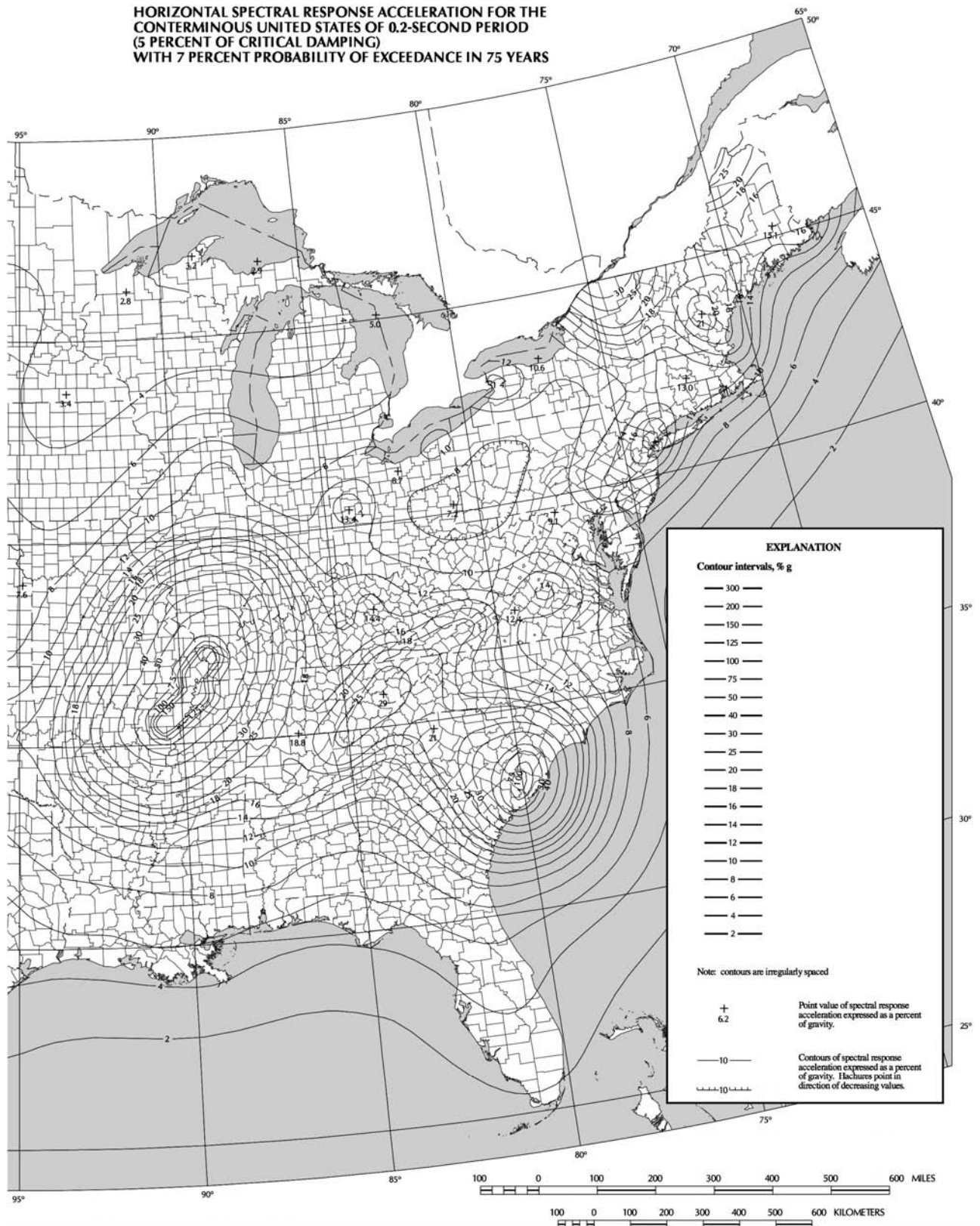


Figure 3.4.1-3b—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 0.2-sec (S_s) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping