ATC/MCEER Joint Venture
A Partnership of the Applied Technology Council and the Multidisciplinary Center for Earthquake Engineering Research

MCEER
MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
A National Center of Excellence in Advanced Technology Applications

Recommended LRFD Guidelines for the Seismic Design of Highway Bridges

Part I: Specifications

based on

NCHRP Project 12-49, FY '98

"Comprehensive Specification for the Seismic Design of Bridges"

Prepared under the MCEER Highway Project
Project 094, Task F3-1

Preliminary Report

ATC Report ATC-49a (draft) and MCEER Technical Report MCEER-02-SP01

November 2001

NOTICE

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PREFACE

The 1971 San Fernando earthquake was a major turning point in the development of seismic design criteria for bridges in the United States. Prior to 1971, the American Association of State Highway and Transportation Officials (AASHTO) specifications for the seismic design of bridges were based in part on the lateral forces requirements for buildings that had been developed by the Structural Engineers Association of California. In 1973, the California Department of Transportation (Caltrans) introduced new seismic design criteria for bridges, which included the relationship of the site to active faults, the seismic response of the soils at the site and the dynamic response characteristics of the bridge. AASHTO adopted Interim Specifications in 1975 which were a slightly modified version of the 1973 Caltrans provisions, and made them applicable to all regions of the United States. In addition to these code changes, the 1971 San Fernando earthquake stimulated research activity on seismic problems related to bridges.

In the light of these research findings, the Federal Highway Administration awarded a contract in 1978 to the Applied Technology Council (ATC) to evaluate current criteria used for seismic design of highway bridges, review available seismic research findings for design applicability and use in new specifications, develop new and improved seismic design guidelines for highway bridges applicable to all regions of the United States, and to evaluate the impact of these guidelines and modify them as appropriate. The guidelines from this ATC project (known as ATC-6) were first adopted by AASHTO as a Guide Specification in 1983. They were later adopted as seismic provisions within the AASHTO *Standard Specifications for Highway Bridges* as Division I-A in 1991.

After damaging earthquakes occurred in California (1989), Costa Rica (1991) and the Philippines (1991), AASHTO requested the Transportation Research Board to review these criteria and prepare revised specifications as appropriate. Funded through the AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) under NCHRP Project 20-7, Task 45, the Multidisciplinary Center for Earthquake Engineering Research (MCEER, formerly known as NCEER) prepared an updated set of seismic design provisions which closely followed the previous criteria but removed ambiguities and technical errors, corrected technical omissions and introduced limited new material which was based field experience and new research findings. The updated provisions were adopted into both the AASHTO *Standard Specifications* and the first and second editions of the AASHTO *LRFD Bridge Design Specifications*. However, the technical basis for the updated provisions was essentially the same as that of the ATC-6 provisions which were initially published in 1981.

Therefore, in 1998, the NCHRP initiated a subsequent study under NCHRP Project 12-49 to develop a new set of seismic design provisions for highway bridges, compatible with the AASHTO *LRFD Bridge Design Specifications*. NCHRP Project 12-49, which was conducted by a joint venture of the Applied Technology Council and the Multidisciplinary Center for Earthquake Engineering Research (the ATC/MCEER Joint Venture), had as its primary objectives the development of seismic design provisions that reflected the latest design philosophies and design approaches that would result in highway bridges with a high level of seismic performance. The results of NCHRP Project 12-49 have been re-formatted into a stand-alone set of provisions that can be more readily used for seismic design through the sponsorship of MCEER with funding from the FHWA. The provisions contained herein are the results of that effort.

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Section 1
INTRODUCTION

1.1 BACKGROUND

In the fall of 1998, the AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) initiated a project to develop a new set of seismic design provisions for highway bridges, compatible with the AASHTO *LRFD Bridge Design Specifications*. NCHRP Project 12-49 which was conducted by a joint venture of the Applied Technology Council and the Multidisciplinary Center for Earthquake Engineering Research (the ATC/MCEER Joint Venture), had as its primary objectives the development of seismic design provisions that reflected the latest design philosophies and design approaches that would result in highway bridges with a high level of seismic performance.

NCHRP Project 12-49 was intended to reflect experience gained during recent damaging earthquakes and the results of research programs conducted in the United States and elsewhere over the prior 10 years. The primary focus of the project was on the development of design provisions which reflected the latest information regarding: design philosophy and performance criteria; seismic hazard representation, loads and displacements, and site effects; advances in analysis and modeling procedures; and requirements for component design and detailing. The new specification were intended to be nationally applicable with provisions for all seismic zones, and all bridge construction types and materials.

The current provisions contained in the AASHTO *LRFD Bridge Design Specifications* are, for the most part, based on provisions and approaches carried over from Division I-A, "Seismic Design," of the AASHTO *Standard Specifications for Highway Bridges*. The Division I-A provisions were originally issued by AASHTO as a Guide Specification in 1983 and were subsequently incorporated with little modification into the *Standard Specifications* in 1991. The current *LRFD* provisions are, therefore, based on seismic hazard, and design criteria and detailing provisions, that are now considered at least 10 years and in many cases nearly 20 years out-of-date.

NCHRP Project 12-49 developed a preliminaryset of comprehensive specification provisions and commentary intended for incorporation into the AASHTO *LRFD* specifications. However, due to the amount of detail in the new provisions and the general view that the new provisions were significantly more complex than the existing provisions, the AASHTO Highway Subcommittee on Bridges and Structures recommended that the new provisions be adopted by AASHTO first as a Guide Specification. This would then allow bridge designers the opportunity to become familiar with the proposed new specifications, and for any problems such as omissions and editorial or technical errors in the new provisions to be identified and rectified, prior to formal adoption into the AASHTO *LRFD* specifications. However, the format of the provisions resulting from NCHRP Project 12-49 were not readily usable without the *LRFD* specifications, nor were they in the stand-alone format of a typical AASHTO Guide Specification.

As a result, MCEER agreed to fund the development of a Guide Specification utilizing the results of NCHRP Project 12-49. This work, which was supported via the FHWA-sponsored Highway Project at MCEER, primarily entailed a reorganization of the NCHRP material into a format more readily amenable for design use as a stand-alone document. This Guide Specification is the result of that effort.

1.2 BASIC CONCEPTS

The development of these specifications was predicated on the following basic concepts.

- Loss of life and serious injuries due to unacceptable bridge performance should be minimized.

- Bridges may suffer damage and may need to be replaced but they should have low probabilities of collapse due to earthquake motions.
- The function of essential (critical lifeline) bridges should be maintained even after a major earthquake.
- Upper level event ground motions used in design should have a low probability of being exceeded during the approximate 75-year design life of the bridge.
- The provisions should be applicable to all regions of the United States.
- The designer should not be restricted from considering and employing new and innovative design approaches and details.

1.3 NEW CONCEPTS AND MAJOR MODIFICATIONS

In comparison to the current AASHTO *Standard Specifications for Highway Bridges* and the AASHTO *LRFD Bridge Design Specifications,* these recommended Guide Specifications contain a number of new concepts and additions as well as some major modifications to the existing provisions. These are summarized as follows:

- New USGS Maps - The national earthquake ground motion map used in the existing AASHTO provisions is a probabilistic map of peak ground acceleration (PGA) on rock which was developed by the U.S. Geological Survey (USGS, 1990). The map provides contours of PGA for a probability of exceedance (PE) of 10% in 50 years, which is approximately 15% PE in the 75 year design life of a typical highway bridge.

  In 1993, the USGS embarked on a major project to prepare updated national earthquake ground motion maps. The result of that project was a set of probabilistic maps published in 1996 (Frankel et al., 1996) that cover several rock ground motion parameters and three different probability levels or return periods. The maps are available as large-scale paper maps, as small-scale paper maps obtained via the Internet, and as digitized values obtained from the Internet or a CD-ROM published by USGS. Parameters of rock ground motions that have been contour mapped by USGS include peak ground acceleration (PGA) and elastic response spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 second. Contour maps for these parameters have been prepared for three different probabilities of exceedance (PE): 10% PE in 50 years, 5% PE in 50 years, and 2% PE in 50 years (approximately 3% PE in 75 years). In addition to these contour maps, the ground motion values at any specified latitude and longitude can be obtained via the Internet for the aforementioned three probability levels for PGA and spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 seconds. In addition, the published CD-ROM contains not only the PGA and spectral acceleration values at three probability levels but also the complete hazard curves (i.e., relationships between the amplitude of a ground motion parameter and its annual frequency of exceedance at each grid point location). Therefore, the ground motion values for all of the aforementioned ground motion parameters can be obtained for any return period or probability of exceedance from the hazard curves. These maps formed the basis for seismic design using these new provisions. Upper bound limits of 1.5 times the median ground motions obtained by deterministic methods have been applied to limit probabilistic ground motions in the western United States.

- Design Earthquakes and Performance Objectives – The existing AASHTO provisions have three implied performance objectives for small, moderate and large earthquakes with detailed design provisions for a 10% PE in 50 year event (approximately 15% PE in 75 year event) to achieve the stated performance objectives. These new provisions provide more definitive performance objectives and damage states for two design earthquakes with explicit design checks to ensure the performance objectives are met. The upper-level event, termed the rare earthquake or Maximum Considered Earthquake (MCE), describes ground motions

that, for most locations, are defined probabilistically and have a probability of exceedance of 3% in 75 years. However, for locations close to highly active faults, the MCE ground motions are deterministically bounded so that the levels of ground motions do not become unreasonably high. Deterministic bound ground motions are calculated assuming the occurrence of maximum magnitude earthquakes on the highly active faults and are equal to 1.5 times median ground motions for the maximum magnitude earthquake but not less than 1.5g for the short-period spectral acceleration plateau and 0.6g for 1.0-second spectra acceleration. On the current MCE maps, deterministic bounds are applied in high-seismicity portions of California, in local areas along the California-Nevada border, along coastal Oregon and Washington, and in high-seismicity portions of Alaska and Hawaii. In areas where deterministic bounds are imposed, ground motions are lower than ground motions for 3% PE in 75 years. The MCE earthquake governs the limits on the inelastic deformation in the substructures and the design displacements for the support of the superstructure.

The lower level design event, termed the Expected Earthquake, has ground motions corresponding to 50% PE in 75 years. This event ensures that essentially elastic response is achieved in the substructures for the more frequent or "expected" earthquake. This design level is similar to the 100 year flood and has similar performance objectives. An explicit check on the strength capacity of the substructures is required. Parameter studies performed as part of the development of the provisions show that the lower level event will only impact the strength of the columns in parts of the Western United States. Background on the choice of the two design events is provided in Appendix A.

- Design Incentives – These provisions contain an incentive from a design and construction perspective for performing a more sophisticated "pushover analysis." The R-Factor increases (approximately 50%) when a pushover analysis is performed, primarily because the analysis results will provide a greater understanding of the demands on the seismic resisting elements. The analysis results are assessed using additional plastic rotation limits on the deformation of the substructure elements to ensure adequate performance.

- New Soil Factors – The site classes and site factors incorporated in these new provisions were originally recommended at a site response workshop in 1992 (Martin, ed., 1994). They were subsequently adopted in the Seismic Design Criteria of Caltrans (1999), the 1994 and 1997 NEHRP Provisions (BSSC, 1995, 1998), the 1997 Uniform Building Code (UBC) (ICBO, 1997), and the 2000 International Building Code (IBC) (ICC, 2000). This is one of the most significant changes with regard to its impact on the level of seismic design forces. It should be noted that the recommended soil factors affect both the peak (flat) portion of the response spectra as well as the declining long period (1/T) portion of the spectra. The increase in site factors with decreasing accelerations is due to the nonlinear response effects of soils. Soils are more linear in their response to lower acceleration events and display more nonlinear response as the acceleration levels increase. The effects of soil nonlinearity are also more significant for soft soils than for stiff soils.

- New Spectral Shapes – The long period portion of the current AASHTO acceleration response spectrum is governed by a spectrum shape that decays as $1/T^{2/3}$. During development of this decay function, there was considerable massaging of the factors that affect the long period portion of the spectra in order to produce a level of approximately 50% conservatism in the design spectra when compared to the ground spectra beyond a 1-second period. These new provisions remove this conservatism and provide a spectral shape that decays as 1/T for periods below 3 seconds.

- Earthquake Resisting Systems and Elements (ERS and ERE) – These provisions provide a mechanism to permit the use of some seismic resisting systems and elements that were not

permitted for use in the current AASHTO provisions. 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 facilitate the concept should be accomplished in the conceptual design or Type, Selection, and Layout (TS&L) phase of the project. Seismic performance is typically better in systems with regular configurations and evenly distributed stiffness and strength. Thus, typical geometric configuration constraints, such as skew, unequal pier heights, and sharp curves, conflict, to some degree, with the seismic design goals. For this reason, it is advisable to resolve potential conflicts between configuration and seismic performance early in the design effort. The classification of ERS and ERE into the categories of (1) permissible, (2) permissible with owner's approval, and (3) not recommended is done to trigger due consideration of seismic performance that leads to the most desirable outcome — that is, seismic performance that ensures wherever possible post-earthquake serviceability. It is not the objective of this specification 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. Common examples from each of the three categories of systems are shown in the Commentary - Figures C3.3.1-1 through C3.3.1-3.

- No Analysis Design Concept – The "no analysis" design procedure is an important new addition to the recommended provisions. It applies to regular bridges in the lower seismic hazard areas, including the expanded areas now requiring more detailed seismic design. The bridge is designed for all non-seismic loads and does not require a seismic demand analysis. Capacity design procedures are used to determine detailing requirements in columns and in the connection forces of columns to the footing and superstructure. There are no seismic design requirements for abutments, except that integral abutments need to be designed for passive pressure.

- Capacity Spectrum Design Procedure – The capacity spectrum design method is a new addition to the provisions and is conceptually the same as the Caltrans' displacement design method. The primary difference is that the capacity spectrum design procedure begins with the non-seismic capacity of the columns and then assesses the adequacy of the resulting displacements. At this time, the capacity spectrum method may be used for very regular bridges that respond essentially as single-degree-of-freedom systems, although future research should expand the range of applicability. The capacity spectrum approach uses the elastic response spectrum for the site, and this is reduced to account for the dissipation of energy in the earthquake resisting elements. The advantage of the approach is that the period of vibration does not need to be calculated, and the designer sees the explicit trade-off between the design forces and displacements.

- Displacement Capacity Verification ("Pushover") Analysis – The pushover method of analysis has seen increasing use since the early 1990's, and is widely employed in the building industry and by some transportation departments including the 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 method was used for two different purposes in these new provisions. First, it provided a mechanism under which the highest R-Factor for preliminary design of a column could be justified, because there are additional limits on the column plastic rotations that the results of the pushover analysis must satisfy. Second, it provided a mechanism to allow incorporation of earthquake resisting elements (ERE) that require owner's approval. The trade-off is the need for a more sophisticated analysis so that the expected deformations in critical elements could be assessed. The ERE could then be

used, provided that the appropriate plastic deformation limits were met.

- Foundations – The new provisions are an update of the existing AASHTO LRFD provisions incorporating explicit material that was referenced in the existing specifications and to incorporate recent research. The changes include specific guidance for the development of spring constants for spread footings and deep foundations (i.e., driven piles and drilled shafts.), as well as approaches for defining the capacity of the foundation system under overturning moments. The capacity provisions specifically address issues such as uplift and plunging (or yield) limits within the foundation. Procedures for including the pile cap in the lateral capacity and displacement evaluation are also provided. The implications of liquefaction of the soil, either below or around the foundation system, is also described. The treatment of liquefaction effects is a major technical addition to the provisions.

- Abutments – The new provisions incorporate much of the research that has been performed on abutments over the past 10 years. Current design practice varies considerably on the use of the abutments as part of the ERS. Some States design a bridge so that the substructures are capable of resisting all of the seismic loads without any contribution from the abutment. Other States use the abutment as a key component of the ERS. Both design approaches are permitted in these provisions. The abutments can be designed as part of the ERS and become an additional source for dissipating the earthquake energy. In the longitudinal direction, the abutment may be designed to resist the forces elastically utilizing the passive pressure of the backfill or, in some cases, passive pressure at the abutment is exceeded, resulting in larger soil movements in the abutment backfill. This requires a more refined analysis to determine the amount of expected movement, and procedures are provided herein to incorporate this nonlinear behavior. In the transverse direction, the abutment is generally designed to resist loads elastically. These provisions therefore recognize that the abutment can be an important part of the ERS and considerable attention is given to abutment impacts on the global response of the bridge. For the abutments to be able to effectively contribute to the ERS, a continuous superstructure is required.

- Liquefaction – Liquefaction has been one of the most significant causes of damage to bridge structures during past earthquakes. Most of the damage has been related to lateral movement of soil at the bridge abutments. However, cases involving the loss of lateral and vertical bearing support of foundations for central piers of a bridge have also occurred. Considerable research and development have occurred over the past decade in the areas of liquefaction potential and effects, and much of this information has been incorporated in these new provisions. For example, the new provisions outline procedures for estimating liquefaction potential using methods developed in 1997, as part of a national workshop on the evaluation of liquefaction. Procedures for quantifying the consequences of liquefaction, such as lateral flow or spreading of approach fills and settlement of liquefied soils, are also given. The provisions also provide specific reference to methods for treating deep foundations extending through soils that are spreading or flowing laterally as a result of liquefaction.

  For sites with mean earthquake magnitudes less than 6.0, the effects of liquefaction on dynamic response can be neglected.

  When liquefaction occurs, vibration and permanent movement occur simultaneously during a seismic event. The recommended methodology in these provisions is to consider the two effects independently; i.e., de-coupled.

  If lateral flow occurs, significant movement of the abutment and foundation systems can result and this can be a difficult problem to mitigate. The range of design options include (1) designing the piles for the flow forces to (2) an acceptance of the predicted lateral flow movements, provided inelastic hinge rotations in the piles remain within a specified limit. The acceptance of plastic hinging in the piles is a deviation from past provisions in that

damage to piles is accepted when lateral flow occurs, thereby acknowledging that the bridge may need to replaced if this option is selected.

Structural or soil mitigation measures to minimize the amount of movement to meet higher performance objectives are also outlined. Due to the concerns on the cost impact of the liquefaction resulting from the higher level design events, two detailed case studies on the application of the recommended design methods for both liquefaction and lateral flow design were performed and summarized in Appendix H. These examples demonstrated that for the soil profiles considered, the new provisions would not be significantly more costly than the application of the more conservative current provisions.

- Steel Design Requirements – The existing AASHTO Specifications do not have seismic requirements for steel bridges, except for the provision of a continuous load path to be identified and designed (for strength) by the engineer. Consequently a comprehensive set of special detailing requirements for steel components expected to yield and dissipate energy in a stable and ductile manner during earthquakes were developed, including provisions for ductile moment-resisting frame substructures, concentrically-braced frame substructures, and end-diaphragms for steel girder and truss superstructures. These provisions now provide a comprehensive set of guidance on steel structures, drafts of which have been reviewed by engineers knowledgeable in steel design and construction practice.

- Concrete Design Requirements – There are no major additions to the concrete provisions contained herein, but there are important updates for key design parameters based on research conducted over the past decade. The minimum amount of longitudinal steel was reduced from 1% to 0.8%, which will result in cost savings when used with the capacity design procedures. An implicit shear equation was also added where no seismic demand has been determined. Modifications to the explicit shear equation and confinement requirements were made, and a global buckling provision was added, as were plastic rotation limits for the pushover analysis.

- Superstructure Design Requirements – Detailed design requirements are not included in the current AASHTO seismic design provisions, other than those required by the generic load path requirement. Therefore, for the higher hazard levels, explicit design requirements have been added since the current provisions result in a wide discrepancy in their application.

- Bearing Design Requirements – One of the significant issues that arose during development of the steel provisions, and was subsequently endorsed by the NCHRP Project Panel and the ATC/MCEER Joint Venture Project Team (PT) and Project Engineering Panel (PEP), was the critical importance of bearings as part of the overall bridge load path. The 1995 Kobe, Japan earthquake (and other more recent earthquakes) clearly showed the very poor performance of some bearing types and the disastrous consequence that a bearing failure can have on the overall performance of the bridge. Three design options are included to address the issue; these are (1) testing of the bearings, (2) ensuring restraint of the bearings, and (3) a design concept that permits the girders to slide on a flat surface if the bearings fail.

- Seismic Isolation Provisions – The *Guide Specifications for Seismic Isolation Design* were first adopted by AASHTO in 1991; they were significantly revised and reissued in 1999. Under the NCHRP 12-49 project, the 1999 Guide Specification provisions were incorporated into the recommended LRFD provisions. This resulted in the addition of a new chapter 15 for the recommended NCHRP 12-49 LRFD provisions, based on issues related to seismic isolation design. That new recommended chapter is included in this Guide Specification as Section 15, and it is essentially the same as the 1999 AASHTO *Guide Specifications for Seismic Isolation Design*.

- Cost Implications – A parameter study was performed as part of the NCHRP 12-49 project and the results are summarized in Appendix

G. In brief, they show that the net effect on the cost of a column and spread footing system is on the average 2% less than the current Division I-A provisions for multi-column bents and 16% less than Division I-A provisions for single column bents. These cost comparisons are based on the use of the more refined method for calculating overstrength factors and 2400 different column configurations including the seismic input of five different cities.

One factor that caused a cost increase in some of the lower period configurations was the short period modifier of Article 4.7 of this *Guide Specification*. Since this provision needs to be a part of any new code and is not part of the current Division I-A provisions, the cumulative effect of all the other changes (including the 3% PE in 75 year/1.5 median deterministic event, new soil factors, new spectral shape, new R-Factors, new phi-factors, cracked section properties for analysis, etc.) would have resulted in even lower average costs had the short period modifier been a part of Division I-A.

Appendix G provides a breakdown of how the accumulation of the new design parameters provides a lower design force for the Seattle, Washington area.

1.4 PROJECT ORGANIZATION

Development of the original NCHRP Project 12-49 provisions (from which this Guide Specification was generated) was done by the ATC/MCEER Joint Venture. Ian Friedland of ATC (and formerly MCEER) was the project principal investigator and Ronald Mayes was the project technical director. Christopher Rojahn of ATC was the project administrative officer on behalf of the ATC/MCEER Joint Venture. The Project team members working on NCHRP Project 12-49 included:

- Donald Anderson, CH2M Hill, Inc.
- Michel Bruneau, University at Buffalo
- Gregory Fenves, University of California at Berkeley
- John Kulicki, Modjeski and Masters, Inc.
- John Mander, University of Canterbury (formerly University at Buffalo)
- Lee Marsh, BERGER/ABAM Engineers
- Ronald Mayes, Simpson, Gumpertz & Heger Consultants
- Geoffrey Martin, University of Southern California
- Andrzej Nowak, University of Michigan
- Richard Nutt, bridge consultant
- Maurice Power, Geomatrix Consultants, Inc.
- Andrei Reinhorn, University at Buffalo

The project also included a distinguished advisory committee through ATC (the ATC Project Engineering Panel); Ian Buckle, of the University of Nevada at Reno, co-chaired this committee with Christopher Rojahn of ATC. Other members included:

- Serafim Arzoumanidis, Steinman Engineers
- Mark Capron, Sverdrup Civil Inc.
- Ignatius Po Lam, Earth Mechanics
- Paul Liles, Georgia DOT
- Brian Maroney, California DOT
- Joseph Nicoletti, URS Greiner Woodward Clyde
- Charles Roeder, University of Washington
- Frieder Seible, University of California at San Diego
- Theodore Zoli, HNTB Corporation

NCHRP Project Panel C12-49, under the direction of NCHRP Senior Program Officer David Beal and chaired by Harry Capers of the New Jersey Department of Transportation, also provided a significant amount of input and guidance during the conduct of the project. The other members of the NCHRP Project Panel were:

- D.W. Dearasaugh, Transportation Research Board
- Gongkang Fu, Wayne State University
- C. Stewart Gloyd, Parsons Brinckerhoff
- Manoucher Karshenas, Illinois DOT
- Richard Land, California DOT
- Bryan Millar, Montana DOT
- Amir Mirmirman, University of Central Florida
- Charles Ruth, Washington State DOT
- Steven Starkey, Oregon DOT

- Phillip Yen, FHWA

Three drafts of the Project 12-49 specifications and commentary were prepared and reviewed by the ATC Project Engineering Panel, NCHRP Project Panel 12-49, and the AASHTO Highway Subcommittee on Bridges and Structures seismic design technical committee (T-3), which was chaired by James Roberts of Caltrans.

The development of this Guide Specification was conducted as a task in the FHWA-sponsored MCEER Highway project following completion of the original NCHRP 12-49 project. The leaders of the effort to reorganize the NCHRP material were:

- Ronald Mayes, Simpson, Gumpertz & Heger Consultants
- Richard Nutt, bridge consultant
- Maurice Power, Geomatrix Consultants, Inc.

These individuals condensed the original draft specifications prepared by the Project Team to this two-volume manual. In addition to making the document more amenable for design, the two volumes address issues identified during final project review and provide additional commentary for some of the studies that were carried out in support of the original specification development

The Applied Technology Council (ATC) provided editorial and desktop publishing services during the preparation of this Guide Specification.

Section 2
DEFINITIONS AND NOTATIONS

2.1 DEFINITIONS

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.

Capacity Spectrum Design – SDAP C – A design and analysis procedure that combines a demand and capacity analysis (See Article 3.10.3.4.1)

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, 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 – SDAP E – A design and analysis procedure that requires the designer to verify that his or her structure has sufficient displacement capacity. It generally involves a non-linear static (i.e. "pushover") analysis.

Earthquake Resisting System – 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.

Expected Earthquake – The largest earthquake that is likely to occur during the life of a bridge. It has a 50 percent chance of being exceeded during a 75 year period.

Lateral Ground Movement – Seismically induced permanent horizontal ground movement

Life Safety Performance Level – The minimum acceptable level of seismic performance allowed by this specification. It is 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 build up 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 porewater pressure (without inertial loading from earthquake). Lateral flow 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.

Maximum Considered Earthquake (MCE) – The upper level, or rare, design earthquake that has a 3 % chance of being exceeded in 75 years.

In areas near highly-active faults, the MCE is deterministically bounded to ground motions that are lower than those having a 3% chance of being exceeded in 75 years.

Minimum Seat Width – The minimum prescribed width of a bearing seat that must be provided in a new bridge designed according to these specifications.

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.

Pushover Analysis – See Displacement Capacity Verification

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

Rare Earthquake – The upper level design event, or maximum considered earthquake (MCE). It has a 3 percent probability of being exceeded during a 75 year period. In areas near highly-active faults, the MCE is deterministically bounded to ground motions that are lower than those having a 3% chance of being exceeded in 75 years.

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

Seismic Design and Analysis Procedure (SDAP) – One of five defined procedures for conducting seismic design and analysis. Minimum requirements are based on seismic hazard level, performance objective, structural configuration, and the type of ERS and/or ERE's.

Seismic Detailing Requirements (SDR) – One of six categories of minimum detailing requirements based on the seismic hazard level and the performance objective.

Seismic Hazard Level – One of four levels of seismic ground shaking exposure measured in terms of the rare earthquake design spectral accelerations for 0.2 and 1.0 seconds.

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.

Square Root of the Sum of the Squares (SRSS) Combination – In this specification, this classical statistical combination rule is used in two ways. The first is for combining forces resulting from two or three orthogonal ground motion components. The second use is for establishing orthogonal moments for biaxial design.

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 NOTATIONS

A_g = gross cross-sectional area of column

B = factor that sets the shape of the interaction diagram for concrete-filled steel pipe, as defined in Article 6.15.4.3.4.b

B_L = capacity spectrum response reduction factor for constant-velocity portion of design response spectrum curve

B_S = capacity spectrum response reduction factor for short-period portion of design response spectrum curve

C_c = seismic capacity coefficient

C_d = seismic demand coefficient

C_{sm} = elastic seismic response coefficient for the mth mode of vibration

C_v = dead load multiplier coefficient for vertical earthquake effects

D = transverse dimension of a column or pile

D' = effective depth of reinforced concrete column

D_p = pile dimension about the weak axis at ground line

D_{eff} = effective gap width at abutment after passive soil resistance is mobilized (m) (Fig 11.6.5.2)

D_g = gap width at abutment (m) (11.6.5.1.1b)

d_b = longitudinal reinforcing bar diameter

d_c = total thickness of cohesive soil at a site

d_i = thickness of soil layer "i"

d_s = total thickness of cohesionless soil at a site

EI_{eff} = effective flexural rigidity, including effect of concrete cracking of reinforced concrete members

F_a = site coefficient for short-period portion of design response spectrum curve

F_v = site coefficient for long-period portion of design response spectrum curve

F_{ye} = Expected yield strength of steel to be used (MPa)

g = acceleration due to gravity, 32.2 ft/sec^2 or 9.81 m/sec^2

K = lateral stiffness of bridge in uniform load method

K_{cr} = lateral stiffness of a concrete pier based on the cracked section properties

K_{eff} = effective lateral stiffness at design displacement

K_{eff1} = effective initial stiffness of abutment backwall and soil including the initial gap (kN/m) (Fig 11.6.5.2)

K_{eff2} = secant stiffness of abutment backwall and soil at maximum EQ displacement (kN/m) (Fig 11.6.5.2)

K_i = initial stiffness of abutment backfill based on soil resistance alone (kN/m) (Fig 11.6.5.2)

K_{rv} = rotational stiffness of pile (MN/mm) (10.7.4.3.1)

K_{sec} = secant stiffness of a column based on the nominal moment capacity and the elastic displacement

L = length of bridge

M_n = nominal moment capacity of a column

M_{po} = plastic overstrength capacity of a column

M_{rc} = factored moment resistance of a concrete filled steel pipe for Article 6.15.4.3.4.2 (kN-m)

M_X = maximum moment about the "x" axis due to earthquake load applied in all directions

M_X^L = maximum moment about the "x" axis due to earthquake load applied in the longitudinal direction

M_X^{LC1} = maximum moment about the "x" axis due to earthquake load case 1

M_X^{LC2} = maximum moment about the "x" axis due to earthquake load case 2

M_X^T = maximum moment about the "x" axis due to earthquake load applied in the transverse direction

M_Y = maximum moment about the "y" axis due to earthquake load applied in all directions

M_Y^L = maximum moment about the "y" axis due to earthquake load applied in the longitudinal direction

M_Y^{LC1} = maximum moment about the "y" axis due to earthquake load case 1

M_Y^{LC2} = maximum moment about the "y" axis due to earthquake load case 2

M_Y^T = maximum moment about the "y" axis due to earthquake load applied in the transverse direction

$\bar{N}$ = average standard penetration test blow count for the top 100 ft (30 m) of a site

$\bar{N}_{ch}$ = average standard penetration test blow count for cohesionless layers of top 100 ft (30 m) of a site

N = minimum seat width

N_i = standard penetration test blow count of soil layer "i"

PI = plasticity index of soil

P_C = axial compression capacity of timber pile

P_e = column axial load

p_e = uniform load on superstructure for uniform load method for design response spectrum curve

p_0 = unit uniform load on superstructure for uniform load method

p_p = passive pressure acting against the abutment backwall under EQ loading (MPa) (11.6.5.1.1b)

P_p = passive force acting against abutment backwall under EQ loading kN (11.6.5.1.1b)

P_{rc} = factored compressive resistance of the concrete core of a concrete-filled steel pipe (Articles 6.9.2.1 and 6.9.5.1) with $\lambda = 0$ (kN)

P_{ro} = factored compressive resistance of concrete-filled steel pipe (Articles 6.9.2.1 and 6.9.5.1) with $\lambda = 0$ (kN)

P_y = axial yield force of steel pile

Q = total factored force effect

Q_i = force effect from specified load

R = response modification factor

R_B = base response modification factor

R_d = ratio of estimated actual displacement to displacement determined from elastic analysis

R_y = ratio of the expected yield strength F_{ye} to the minimum specified yield strength F_y

S_a = design response spectral acceleration

S_{DS} = design earthquake response spectral acceleration at short periods

S_{DI} = design earthquake response spectral acceleration at 1 second period

S_s = 0.2-second period spectral acceleration on Class B rock from national ground motion maps

S_1 = 1-second period spectral acceleration on Class B rock from national ground motion maps

$\bar{s}_u$ = average undrained shear strength of cohesive soil layers in the top 100 ft (30 m) of a site

S_{ui} = undrained shear strength of cohesive soil layer "*i*"

T = period of vibration

T_s = period at the end of constant design spectral acceleration plateau

T_0 = period at beginning of constant design spectral acceleration plateau

T^* = period used to calculate R and R_d

t = thickness of pier wall

T_{eff} = effective vibration period at design displacement

T_m = vibration period for uniform load method

V = equivalent static lateral force for uniform load method

$\overline{v}_s$ = average shear wave velocity for the top 100 ft (30 m) of a site

$v_{s,\max}$ = maximum displacement of bridge under uniform load

v_{si} = shear wave velocity of soil layer "i"

W = weight of bridge

w = moisture content of soil in percent

α_{skew} = skew angle of the bridge, (0 degrees being the angle for a right bridge)

β = damping ratio in percent

Δ = displacement from an elastic seismic analysis

Δ_m = estimated actual displacement at the center of mass

ε_y = yield strain of longitudinal reinforcement

Λ = fixity factor used for the calculation of shear forces

λ_v = empirical factor used to adjust resistance factors for ultimate capacity determination based on the method of construction supervision or monitoring during pile installation (DIM) (10.5.5)

θ = principal crack angle in reinforced concrete column

θ_p = plastic rotation at a plastic hinge

ρ_ℓ = longitudinal reinforcement ratio of a column or pier

Section 3
GENERAL REQUIREMENTS

3.1 APPLICABILITY

The provisions herein shall apply to bridges of conventional slab, beam girder, box girder, and truss superstructure construction. For other types of construction (i.e. cable stayed and suspension), the Owner shall specify and/or approve appropriate provisions. Seismic effects for box culverts and buried structures need not be considered, except where they cross active faults. The potential for soil liquefaction and slope movements shall be considered.

3.2 SEISMIC PERFORMANCE OBJECTIVES

Bridges shall be designed to satisfy the performance criteria given in Table 3.2-1 As a minimum, bridges shall be designed for the life safety level of performance. Higher levels of performance may be required at the discretion of the bridge owner. Development of design earthquake ground motions for the probabilities of exceedance in Table 3.2-1 are given in Article 3.4.

When required by the provisions of this specification, seismic performance shall be assured by verifying that displacements are limited to satisfy geometric, structural and foundation constraints on performance.

Table 3.2-1 Design Earthquakes and Seismic Performance Objectives

		Performance Level[1]	
Probability of Exceedance For Design Earthquake Ground Motions[4]		**Life Safety**	**Operational**
Rare Earthquake (MCE) 3% PE in 75 years/1.5 Median Deterministic	Service[2]	Significant Disruption	Immediate
	Damage[3]	Significant	Minimal
Expected Earthquake 50% PE in 75 years	Service	Immediate	Immediate
	Damage	Minimal	Minimal to None

Notes:

(1) Performance Levels

These are defined in terms of their anticipated performance objectives in the upper level earthquake. Life safety in the MCE event means that the bridge should not collapse but partial or complete replacement may be required. Since a dual level design is required the Life Safety performance level will have immediate service and minimal damage for the expected design earthquake. For the operational performance level the intent is that there will be immediate service and minimal damage for both the rare and expected earthquakes.

(2) Service Levels*:

- *Immediate* – Full access to normal traffic shall be available following an inspection of the bridge.
- *Significant Disruption* – Limited access (Reduced lanes, light emergency traffic) may be possible after shoring, however the bridge may need to be replaced.

(3) Damage Levels*:

- None – *Evidence* of movement may be present but no notable damage.
- Minimal – Some visible signs of damage. Minor inelastic response may occur, but post-earthquake damage is limited to narrow flexural cracking in concrete and the onset of yielding in steel. Permanent deformations are not apparent, and any repairs could be made under non-emergency conditions with the exception of superstructure joints.
- Significant – Although there is no collapse, permanent offsets may occur and damage consisting of cracking, reinforcement yield, and major spalling of concrete and 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 on the seismic load path is possible. 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, significant inelastic deformation is permitted in the piles, whereas for all other sites the foundations are capacity-protected and no damage is anticipated. 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 approaches producing minimal or moderate damage (Figure C3.3-1) such as seismic isolation or the control and repairability design concept should be assessed.

* See commentary and design sections for geometric and structural constraints on displacements and deformations.

(4) The upper-level earthquake considered in these provisions is designated the Maximum Considered Earthquake, or MCE. In general the ground motions on national MCE ground motion maps have a probability of exceedance (PE) of approximately 3% PE in 75 years. However, adjacent to highly active faults, ground motions on MCE maps are bounded deterministically as described in the commentary for Article 3.2. When bounded deterministically, MCE ground motions are lower than ground motions having 3% PE in 75 years. The performance objective for the expected earthquake is either explicitly included as an essentially elastic design for the 50% PE in 75 year force level or results implicitly from design for the 3% PE in 75 year force level.

3.3 SEISMIC DESIGN APPROACH

All bridges and their foundations shall have a clearly identifiable earthquake resisting system (ERS) selected to achieve the performance objectives defined in Table 3.2-1. The ERS shall provide 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. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen mechanism of seismic resistance and other structural requirements.

3.3.1 Earthquake Resisting Systems (ERS)

For the purposes of encouraging the use of appropriate systems and of ensuring due consideration of performance by the owner, the ERS and earthquake resisting elements (ERE) are categorized as follows:

- Permissible
- Permissible with Owner Approval
- Not Recommended for New Bridges

These terms apply to both systems and elements. For a system to be in the permissible category, its primary ERE must all be in the permissible category. If any ERE are not permissible, then the entire system is not permissible.

Permissible systems (Figure C3.3.1-1a and -1b) and elements have the following characteristics:

1. 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 with the owners' approval. If all structural elements of a bridge are designed elastically (R=1.0 and Article 4.10) then no inelastic deformation is anticipated and elastic

elements are permissible, but ductile detailing is required.

2. 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 systems that require owner approval (Figure C3.3.1-2) are those systems that do not meet either item (1) or (2), above. Such systems may only be used with the owners' approval. Additionally, these systems will require the use of the highest level of analysis (Seismic Design and Analysis Procedures E – SDAP E), as outlined in the flow chart shown in Figure 3.3.1-1. The minimum Seismic Design and Analysis Procedures (SDAP) are defined in Article 3.7.

In general, systems that do not fall in either of the two permissible categories (Figure C3.3.1-3) are not allowed. 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.

The interrelationship between the performance objective and the ERS is given in Table 3.3.1-1. Abutment design issues are amplified in Table 3.3.1-2.

3.4 DESIGN GROUND MOTION

Design response spectra acceleration parameters shall be obtained using either a general procedure (Article 3.4.1) or a site-specific procedure (Article 3.4.3). A site-specific procedure shall be used if any of the following apply:

- Soils at the site require site-specific evaluation (i.e. Site Class F soils, Article 3.4.2.1), unless a determination is made that the presence of such soils would not result in a significantly higher response of the bridge.

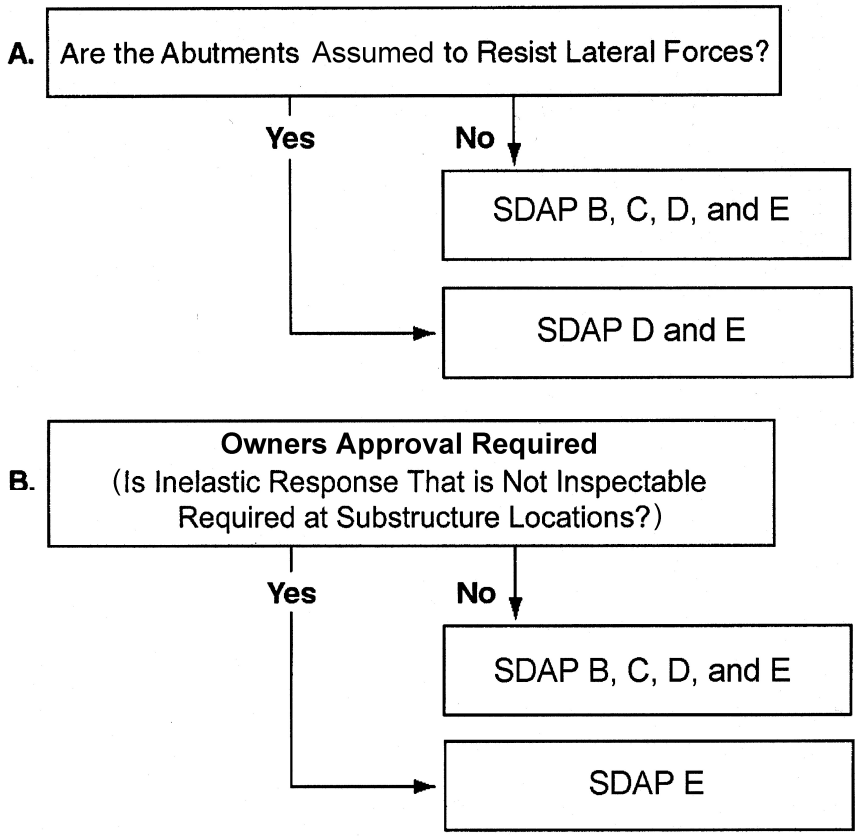


Figure 3.3.1-1 Classification of ERS

- The bridge is considered to be a major or very important structure for which a higher degree of confidence of meeting the seismic performance objectives of Article 3.2 is desired.
- The site is located within 10 km (6.25 miles) of a known active fault and its response could be significantly and adversely influenced by near-fault ground motion characteristics.

3.4.1 Design Spectra Based on General Procedure

Design response spectra for the rare earthquake (MCE) and expected earthquake shall be constructed using the accelerations from national ground motion maps described in this section 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.

Design earthquake response spectral acceleration at short periods, S_{DS}, and at 1 second period, S_{D1}, shall be determined from Eq. 3.4.1-1 and 3.4.1-2, respectively:

$$S_{DS} = F_a S_s \qquad (3.4.1\text{-}1)$$

and

$$S_{DI} = F_v S_1 \qquad (3.4.1\text{-}2)$$

where S_s and S_1 are the 0.2-second period spectral acceleration and 1-second period spectral acceleration, respectively, on Class B rock from ground motion maps described below and F_a and F_v are site coefficients described in Article 3.4.2.3. Values of S_s and S_1 may be obtained by the following methods:

1. For the MCE
   (a) S_s and S_1 may be obtained from national ground motion maps (Figures 3.4.1-2(a) through 3.4.1-2(l).
   (b) S_s and S_1 may be obtained from the CD-ROM published by the U.S. Geological Survey (Leyendecker et al., 2000a) for site coordinates specified by latitude and longitude, or alternatively, by zip code.
2. For the expected earthquake, S_s and S_1 may be obtained by linear interpolation from hazard curves on the CD-ROM published by the U.S. Geological Survey (Frankel and Leyendecker, 2000) for site coordinates specified by latitude and longitude or alternatively by zip code.

Table 3.3.1-1 Performance Levels and Earthquake Resisting Systems

Performance Level	Expected Element Behavior	Earthquake Resisting System	Abutment Performance	
			50% in 75 Years	3% in 75 Years
Operational	Linear Elastic Nonlinear Elastic	Permissible elements designed to resist all seismic loads within displacement constraints. Elements requiring owner approval should not be used.	No damage. Soil passive mobilization is OK if $\Delta \leq 0.01H_E$	No damage. Soil passive mobilization is O.K. if $\Delta \leq 0.02H_E$
Life Safety	Linear Elastic Nonlinear Elastic Nonlinear Inelastic	Permissible elements designed to resist all seismic loads within displacement constraints. Elements requiring owner approval are OK.	Limited damage and soil passive mobilization O.K.	Significant damage. Soil passive mobilization is O.K.

Table 3.3.1-2 Abutment Design Issues

No Damage		Significant Damage Accepted	
Longitudinal	Transverse	ERS does not Include Abutment Contribution	ERS Includes Abutment Contribution
Alternate 1 – Abutment resists forces by mobilizing passive soil for 3% in 75-year event and displacement constraints of Table 3.10.1-2 are acceptable ($\Delta \leq 0.02H_E$). Needs sufficient backwall clearance for 50% in 75-year event.	Alternate 1 – Design abutments to resist full 3% in 75-year transverse loads within acceptable displacement limits of Table 3.10.1-2 ($\Delta \leq 0.02H_E$)	The ERS is designed to resist all seismic loads without any contribution from abutments (SDAP B and C). Abutments then limit displacement and provide additional capacity and better performance. The bridge is safe even if serious problems occur at the abutments. For SDAP D and E and the 50% in 75-year event, the bridge should be analyzed with the abutments and the abutments are designed for the 50% in 75-year forces and displacements. If sacrificial concrete shear keys are used to protect the piles, the bridge shall be analysed with all combinations of shear key failure considered (i.e. at each abutment separately and both abutments simultaneously).	The ERS is designed with the abutments as a key element of the ERS. Abutment are designed and analyzed for the 3% in 75-year forces and displacements.
Alternate 2 – Abutment does not mobilize passive soil in 3% in 75-year event. Need sufficient clearance to backwall or use top of backwall knockoff detail.	Alternate 2 – Provide capacity protection (force-limiting devices) for abutment, plus sufficient clearance. Transverse force capacity governed by 50% in 75-year forces. Capacity protection by shear key or bearings that provide sufficient nonseismic lateral capacity and then have sufficient displacement capacity for 3% in 75-year event. If sacrificial concrete shear keys are used to protect the piles, the bridge shall be analysed with all combinations of shear key failure considered (i.e. at each abutment separately and both abutments simultaneously).		
Alternate 3 – With either of above alternatives, use displacement-limiting devices (isolation bearing or energy dissipation devices) to limit overall deck displacements. Displacements can be reduced by up to a factor of 2 with 30% damping.	Alternate 3 – Provide sufficient clearance in the transverse direction to permit the deck to move. The movement can be limited with isolation bearings or energy dissipation devices		

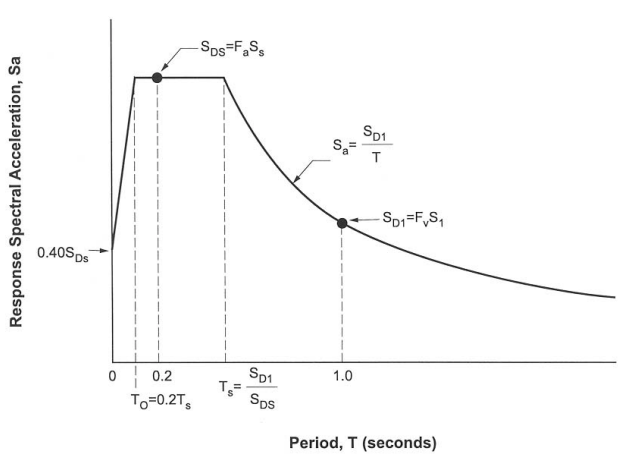


Figure 3.4.1-1 Design Response Spectrum, Construction Using Two-Point Method

The design response spectrum curve shall be developed as indicated in Figure 3.4.1-1 and as follows:

1. For periods less than or equal to T_0, the design response spectral acceleration, S_a, shall be defined by Equation 3.4.1-3:

$$S_a = 0.60\frac{S_{DS}}{T_0}T + 0.40S_{DS} \qquad (3.4.1\text{-}3)$$

T and T_0 are defined in 2 below.

Note that for $T = 0$ seconds, the resulting value of S_a is equal to peak ground acceleration, PGA.

2. For periods greater than or equal to T_0 and less than or equal to T_s, the design response spectral acceleration, S_a, shall be defined by Equation 3.4.1-4:

$$S_a = S_{DS} \qquad (3.4.1\text{-}4)$$

where $T_0 = 0.2T_s$, and $T_s = S_{D1}/S_{DS}$, and T =period of vibration (sec).

3. For periods greater than T_s, the design response spectral acceleration, S_a, shall be defined by Equation 3.4.1-5:

$$S_a = \frac{S_{D1}}{T} \qquad (3.4.1\text{-}5)$$

Response spectra constructed using maps and procedures described in Article 3.4.1 are for a damping ratio of 5%.

3.4.2 Site Effects on Ground Motions

The generalized site classes and site factors described in this section 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.

3.4.2.1 Site Class Definitions

The site shall be classified as one of the following classes according to the average shear wave velocity, SPT blow count (N-value), or undrained shear strength in the upper 30 m (100 ft) of site profile. Procedures given in Article 3.4.2.2 shall be used to determine the average condition.

A Hard rock with measured shear wave velocity, $\bar{v}_s$ > 1500 m/s (5000 ft/sec)

B Rock with 760 m/s < $\bar{v}_s$ ≤ 1500 m/s (2500 ft/sec < $\bar{v}_s$ ≤ 5000 ft/sec)

C Very dense soil and soft rock with 360 m/s < $\bar{v}_s$ ≤ 760 m/s (1200 ft/sec < $\bar{v}_s$ ≤ 2500 ft/sec) or with either $\bar{N}$ > 50 blows/0.30 m (blows/ft) or $\bar{s}_u$ > 100 kPa (2000 psf)

D Stiff soil with 180 m/s ≤ $\bar{v}_s$ ≤ 360 m/s (600 ft/sec ≤ $\bar{v}_s$ ≤ 1200 ft/sec) or with either 15 ≤ $\bar{N}$ ≤ 50 blows/0.30 m (blows/ft) or 50 kPa ≤ $\bar{s}_u$ ≤ 100 kPa (1000 psf ≤ $\bar{s}_u$ ≤ 2000 psf)

E A soil profile with $\bar{v}_s$ < 180 m/s (600 ft/sec) or with either $\bar{N}$ < 15 blows/0.30 m (blows/ft) or $\bar{s}_u$ < 50 kPa (1000 psf), or any profile with more than 3 m (10 ft) of soft clay defined as soil with PI > 20, w ≥ 40 percent, and $\bar{s}_u$ < 25 kPa (500 psf)

Table 3.4.2-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A	> 1500 m/sec (> 5000 ft/sec)	–	–
B	760 to 1500 m/sec (2500 to 5000 ft/sec)	–	–
C	360 to 760 m/sec (1200 to 2500 ft/sec)	> 50	> 100 kPa (> 2000 psf)
D	180 to 360 m/sec (600 to 1200 ft/sec)	15 to 50	50 to 100 kPa (1000 to 2000 psf)
E	< 180 m/sec (<600 ft/sec)	<15 blows/0.30 m (15 blows/ft)	< 50 kPa (<1000 psf)

NOTE: If the $\bar{s}_u$ method is used and the $\bar{N}_{ch}$ and $\bar{s}_u$ criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

F Soils requiring site-specific evaluations:

1. Peats and/or highly organic clays (H > 3 m [10 ft] of peat and/or highly organic clay where H = thickness of soil)
2. Very high plasticity clays (H > 8 m [25 ft] with PI > 75)
3. Very thick soft/medium stiff clays (H > 36 m [120 ft])

When the soil properties are not known in sufficient detail to determine the Site Class, Site Class D may be used. Consequently Site Classes E or F need not be assumed unless the authority having jurisdiction determines that Site Classes E or F could be present at the site or in the event that Site Classes E or F are 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 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 30 m (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 3 m (10 ft) of soil between the rock surface and the bottom of the spread footing or mat foundation.

3.4.2.2 Definitions of Site Class Parameters

The definitions presented below apply to the upper 30 m (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 30 m (100 ft). The subscript i then refers to any one of the layers between 1 and n.

The average $\bar{v}_s$ for the layer is as follows:

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

where $\sum_{i=1}^{n} d_i$ is equal to 30 m (100 ft), v_{si} is the shear wave velocity in m/s (ft/sec) of the layer, and d_i is the thickness of any layer between 0 and 30 m (100 ft).

N_i is the Standard Penetration Resistance (ASTM D1586-84) not to exceed 100 blows/0.30 m (100 blows/ft) as directly measured in the field without corrections.

$\bar{N}$ is:

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

$\bar{N}_{ch}$ is

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

where $\sum_{i=1}^{m} d_i = d_s$

In Equation 3.4.2.2-3, d_i and N_i are for cohesionless soils only and d_s is the total thickness of cohesionless soil layers in the top 30 m (100 ft).

s_{ul} is the undrained shear strength in kPa (psf), not to exceed 250 kPa (5,000 psf), ASTM D2166-91 or D2850-87.

$\bar{s}_u$ is:

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

where $\sum_{i=1}^{k} d_i = d_c$

d_c is the total thickness [(30-d_s) m (100-d_s)ft] of cohesive soil layers in the top 30 m (100 ft).

PI is the plasticity index, ASTM D4318-93.

w is the moisture content in percent, ASTM D2216-92.

3.4.2.3 Site Coefficients

Site coefficients for the short-period range (F_a) and for the long-period range (F_v) are given in Tables 3.4.2.3-1 and 3.4.2.3-2, respectively. Application of these coefficients to determine elastic seismic response coefficients of ground motions is described in Article 3.4.1

Table 3.4.2.3-1 Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at Short Periods				
	$S_s \leq 0.25$ g	$S_s = 0.50$ g	$S_s = 0.75$ g	$S_s = 1.00$ g	$S_s \geq 1.25$ g
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 S_s, where S_s is the spectral acceleration at 0.2 seconds obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3). For the purpose of defining Seismic Hazard Levels in Article 3.7 Type E values may be used for Type F soils.

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

Site Class	Mapped Spectral Response Acceleration at 1 Second Periods				
	$S_1 \leq 0.1$ g	$S_1 = 0.2$ g	$S_1 = 0.3$ g	$S_1 = 0.4$ g	$S_1 \geq 0.5$ g
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 at 1.0 second obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3). For the purpose of defining Seismic Hazard Levels in Article 3.7 Type E values may be used for Type F soils.

3.4.3 Response Spectra Based on Site-Specific Procedure

A site-specific procedure to develop design response spectra of earthquake ground motions shall be performed when required by Article 3.4 and may be performed for any site. A site-specific probabilistic ground motion analysis shall include the following: characterization of seismic sources and ground motion attenuation that incorporates current scientific interpretations, including uncertainties in seismic source and ground motion models and parameter values; detailed documentation; and detailed peer review (Article C3.4.1).

Where analyses to determine site soil response effects are required by Articles 3.4 and 3.4.2.1 for Site Class F soils, the influence of the local soil conditions shall be determined based on site-specific geotechnical investigations and dynamic site response analyses.

For sites located within 10km (6 miles) of an active fault (as defined in Article 3.4), studies shall be considered to quantify near-fault effects on ground motions to determine if these could significantly influence the bridge response.

In cases where the 0.2-second or 1.0-second response spectral accelerations of the site-specific probabilistic response spectrum for the MCE exceeds the response spectrum shown in Figure 3.4.3-1, a deterministic spectrum may be utilized in regions having known active faults if the deterministic spectrum is lower than the probabilistic spectrum. The deterministic spectrum shall be the envelope of median-plus-one standard-deviation spectra calculated for characteristic maximum magnitude earthquakes on known active faults, but shall not be lower than the spectrum shown in Figure 3.4.3-1. If there is more than one active fault in the site region, the deterministic spectrum shall be calculated as the envelope of spectra for the different faults. Alternatively, deterministic spectra may be defined for each fault, and each spectrum, or the spectrum that governs bridge response, may be used for the analysis of the bridge.

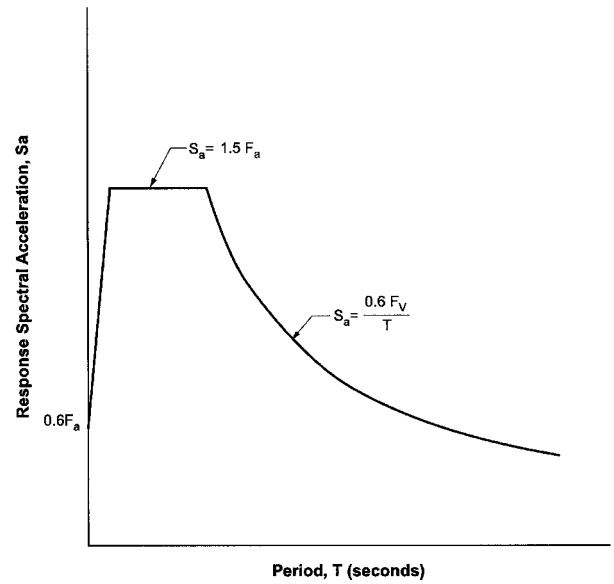


Figure 3.4.3-1 Minimum Deterministic Response Spectrum

When response spectra are determined from a site-specific study, the spectra shall not be lower than two-thirds of the response spectra determined using the general procedure in Article 3.4.1.

3.4.4 Acceleration Time Histories

The development of time histories shall meet the requirements of this section. The developed time histories shall have characteristics that are representative of the seismic environment of the site and the local site conditions.

Time histories may be either recorded time histories or spectrum-matched time histories. If sufficient recorded motions are not available, simulated-recorded time histories may be developed using theoretical ground motion modeling methods that simulate the earthquake rupture and the source-to-site seismic wave propagation.

If spectrum-matched time histories are developed, the initial time histories to be spectrum matched shall be representative recorded or simulated-recorded 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.

When using recorded or simulated-recorded time histories, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. For each component of motion, an aggregate match of the design response spectrum shall be achieved for the set of acceleration time histories used. A mean spectrum of the individual spectra of the time histories shall be calculated period-by-period. Over the defined period range of significance, the mean spectrum shall not be more than 15% lower than the design spectrum at any period, and the average of the ratios of the mean spectrum to the design spectrum shall be equal to or greater than unity. When developing spectrum-matched time histories, before the matching process, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. Thereafter, the set of time histories for each component shall be spectrum-matched to achieve the aggregate fit requirement stated above.

At least three time histories shall be used for each component of motion for use in nonlinear inelastic time history analysis using either recorded, simulated-recorded, or spectrum-matched motions for either the 3% PE in 75 yr/1.5 mean deterministic or 50% PE in 75 yr event. 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 are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction.

3.4.5 Vertical Acceleration Effects

The impact of vertical ground motion may be ignored if the bridge site is greater than 50km (32 miles) from an active fault as defined in Article 3.4 and may be ignored for all bridges in the central and Eastern U.S. as well as those areas impacted by subduction earthquakes in the Pacific Northwest. If the bridge site is located within 10km (6 miles) of an active fault then a site specific study is required if the response of the bridge could be significantly and adversely affected by vertical ground motion characteristics. In such cases response spectra and acceleration time histories as appropriate shall be developed and shall include appropriate vertical ground motions for inclusion in the design and analysis of the bridge. For vertical design forces the linear analysis shall use the CQC modal combination method and the SRSS directional combination method.

If the bridge site is located between 10km (6 miles) and 50km (32 miles) of an active fault a site specific study may be performed including the effects of appropriate vertical ground motion.

In lieu of a dynamic analysis that incorporates the effect of vertical ground motions, the following variations in column axial loads and superstructure moments and shears shall be included in the design of the columns and the superstructure to account for the effects of vertical ground motion.

Column Axial Loads (AL) = DL Axial Force $\pm$ C_V (DL Axial Force)

Superstructure Bending Moments = DL Moment $\pm$ C_V (DL Moment)

Superstructure Shears = DL Shear $\pm$ C_V (DL Shear)

C_V is the coefficient given in Table 3.4.5-1 if the maximum magnitude of the design earthquake is 7.0 or less, or Table 3.4.5-2 if the maximum magnitude of the design earthquake is greater than 7.0. Note that the coefficient C_V for the superstructure has a value specified at the mid-span location and at the column/pier support. Linear interpolation is used to determine C_V for points on the superstructure between these locations.

Table 3.4.5-1 Fault distance zones and corresponding dead load multiplier (C_v) for all bridges for rock and soil site conditions and a magnitude 7.0 event or less.

Response Quantity	Fault Distance Zones (km)				
	0-10	10-20	20-30	30-40	40-50
Pier Axial Force DL Multiplier	0.7	0.3	0.20	0.1	0.1
Superstructure Shear Force at Pier DL Multiplier	0.7	0.4	0.2	0.1	0.1
Superstructure Bending Moment at Pier DL Multiplier	0.6	0.3	0.2	0.1	0.1
Superstructure Shear Force at Mid-Span DL Multiplier	0.1	0.1	0.1	0.1	0.1
Superstructure Bending Moment at Mid-Span DL Multiplier	1.4	0.7	0.4	0.3	0.2

Footnotes

(1) The DL Multiplier values given above are in addition to the dead load; thus, an actual "load factor" would be 1.0 plus/minus the above numbers.

(2) The Live Load (LL) typically used in the design of bridge types shown in this study is in the range of 20-30% of the Dead Load (DL).

Table 3.4.5-2 Fault distance zones and corresponding dead load multiplier (C_v) for all bridges for rock and soil site conditions and an event magnitude greater than 7.0.

Response Quantity	Fault Distance Zones (km)				
	0-10	10-20	20-30	30-40	40-50
Pier Axial Force DL Multiplier	0.9	0.4	0.2	0.2	0.1
Superstructure Shear Force at Pier DL Multiplier	1.0	0.5	0.3	0.2	0.2
Superstructure Bending Moment at Pier DL Multiplier	1.0	0.5	0.3	0.2	0.2
Superstructure Shear Force at Mid-Span DL Multiplier	0.2	0.1	0.1	0.1	0.1
Superstructure Bending Moment at Mid-Span DL Multiplier	1.9	1.0	0.6	0.5	0.3

Footnotes

(1) The DL Multiplier values given above are in addition to the dead load; thus, an actual "load factor" would be 1.0 plus/minus the above numbers.

(2) The Live Load (LL) typically used in the design of bridge types shown in this study is in the range of 20-30% of the Dead Load (DL).

3.5 LOAD FACTORS

EXTREME EVENT-I (Table 3.5-1) – Load combination including rare and expected earthquakes.

The load factor for live load in Extreme Event Load Combination I, γ_{EQ}, shall be determined on a project specific basis. The inertia effects of live load do not need to be considered when performing a dynamic analysis. It is generally not necessary to consider the gravity effects of live load for Extreme Event-I except for bridges with heavy truck traffic (i.e. high ADTT) and/or elements particularly sensitive to gravity loading such as C-bents, outrigger bents or superstructures with nonsymmetrical geometry.

Table 3.5-1 Load Combinations and Load Factors

Load Combination Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH-I (unless noted)	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-IV EH, EV, ES, DW DC ONLY	γ_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH-V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME EVENT-I	1.00	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME EVENT-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

3.6 COMBINATION OF SEISMIC FORCE EFFECTS

The maximum seismic force due to seismic load in any one direction shall be based on the CQC combination of modal responses due to ground motion in that direction. The maximum force due to two or three orthogonal ground motion components shall be obtained either by the SRSS combination or the 100% - 40% combination forces due to the individual seismic loads.

3.6.1 SRSS Combination Rule

The maximum response quantity of interest is the SRSS combination of the response quantity from each of the orthogonal directions. (i.e., $M_x = \sqrt{(M_x^T)^2 + (M_x^L)^2}$ where M_x^T and M_x^L are the x-component moments from a transverse and longitudinal analysis)

If biaxial design of an element is important (e.g. circular columns) and the bridge has a maximum skew angle less than 10 degrees and/or a subtended angle less than 10 degrees then the maximum response quantities in the two orthogonal directions (Mx, My) may use the 100% - 40% rule prior to obtaining the vector sum. The maximum vector moment is the maximum of:

$$\sqrt{M_x^2 + (0.4M_y)^2} \text{ or } \sqrt{(0.4M_x)^2 + M_y^2} \qquad (3.6\text{-}1)$$

If the maximum skew angle or the subtended angle in a horizontally curved bridge exceeds 10 degrees then the maximum response quantities in the two horizontal directions shall be combined as the vector sum:

$$\sqrt{M_x^2 + M_y^2} \qquad (3.6\text{-}2)$$

3.6.2 100% - 40% Combination Rule

The maximum response quantity of interest shall be obtained from the maximum of two load cases.

Load Case 1 (LC1) – 100% of the absolute value of the response quantity resulting from the analysis in one orthogonal direction (transverse) added to 40% of the response quantity resulting from the analyses in the other orthogonal direction(s) (longitudinal).

$$M_x^{LC1} = 1.0M_x^T + 0.4M_x^L \qquad (3.6\text{-}3)$$

Load Case 2 (LC2) – 100% of the absolute value of the response quantity resulting from an analysis in the other orthogonal direction (longitudinal) added to 40% of the response quantity resulting from an analysis in the original direction (transverse).

$$M_x^{LC2} = 0.4M_x^T + 1.0M_x^L \qquad (3.6\text{-}4)$$

If biaxial design of an element is important then the maximum response quantities in the two orthogonal directions from each load case shall be combined to obtain a vectorial sum and the maximum vector from the two load cases shall be used for design, i.e., the maximum of:

$$\sqrt{(M_x^{LC1})^2 + (M_y^{LC1})^2} \text{ or } \sqrt{(M_x^{LC2})^2 + (M_y^{LC2})^2} \qquad (3.6\text{-}5)$$

3.7 SEISMIC HAZARD LEVEL (SHL), SEISMIC DESIGN AND ANALYSIS PROCEDURE (SDAP) AND SEISMIC DESIGN REQUIREMENT (SDR)

Each bridge shall be assigned a Seismic Hazard Level that shall be the highest level determined by the value of F_vS_1 or F_aS_s from Table 3.7-1.

Table 3.7-1 Seismic Hazard Levels

Seismic Hazard Level	Value of F_vS_1	Value of F_aS_s
I	$F_vS_1 \le 0.15$	$F_aS_s \le 0.15$
II	$0.15 < F_vS_1 \le 0.25$	$0.15 < F_aS_s \le 0.35$
III	$0.25 < F_vS_1 \le 0.40$	$0.35 < F_aS_s \le 0.60$
IV	$0.40 < F_vS_1$	$0.60 < F_aS_s$

Notes:

1. For the purposes of determining the Seismic Hazard Level for Site Class E Soils (Article 3.4.2.3) the value of F_v and F_a need not be taken larger than 2.4 and 1.6 respectively when S_1 is less than or equal to 0.10 and S_S is less than 0.25.
2. For the purposes of determining the Seismic Hazard Level for Site Class F Soils (Article 3.4.2.3) F_v and F_a values for Site Class E soils may be used with the adjustment described in Note 1 above.

Each bridge shall be designed, analyzed and detailed for seismic effects in accordance with Table 3.7-2. Seismic Design and Analysis Procedures (SDAP) are described in Section 4. Minimum seismic design requirements (SDR) for SDR 1 and 2, SDR 3 and SDR 4, 5, and 6 are given in Sections 6, 7 and 8, respectively.

Table 3.7-2 Seismic Design and Analysis Procedures (SDAP) and Seismic Design Requirements (SDR)

Seismic Hazard Level	Life Safety		Operational	
	SDAP	SDR	SDAP	SDR
I	A1	1	A2	2
II	A2	2	C/D/E	3
III	B/C/D/E	3	C/D/E	5
IV	C/D/E	4	C/D/E	6

Notes:

1. SDAP B/C – The use of these two design/analysis procedures is governed by regularity requirements as defined in Articles 4.3.2 and 4.4.2 respectively.
2. SDAP D – The use of the uniform load method is only permitted for the life safety performance level and limits on its use are given in Article 5.4.2.1.
3. If abutments are required to deform inelastically and act as part of the ERS then only SDAP D or E can be used and the uniform load method (ULM) is not permitted.
4. If owners approval of an ERE is required (Article 3.3.1 – i.e. inelastic behavior that is not inspectable occurs in a substructure) then SDAP E must be used.
5. Seismic Design Requirements (SDR) 1 and 2 are given in Section 6, SDR 3 are given in Section 7, and SDR 4, 5, and 6 are given in Section 8.

3.8 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.

The requirement that an earthquake shall not cause collapse of all or part of a bridge, as stated in Article 3.2, shall apply to temporary bridges expected to carry traffic. 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 ground response spectra given in Article 3.4 may be reduced by a factor of not more than 2 in order to calculate the component elastic forces and displacements. Response spectra for construction sites that are close to active faults shall be the subject of special study. The response modification factors given in Article 4.7 may be increased by a factor of not more than 1.5 in order to calculate the design forces. This factor shall not be applied to connections as defined in Table 4.7-2.

The minimum seat width provisions of Article 7.3 or 8.3 shall apply to all temporary bridges and staged construction.

Figure 3.4.1-2(a) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

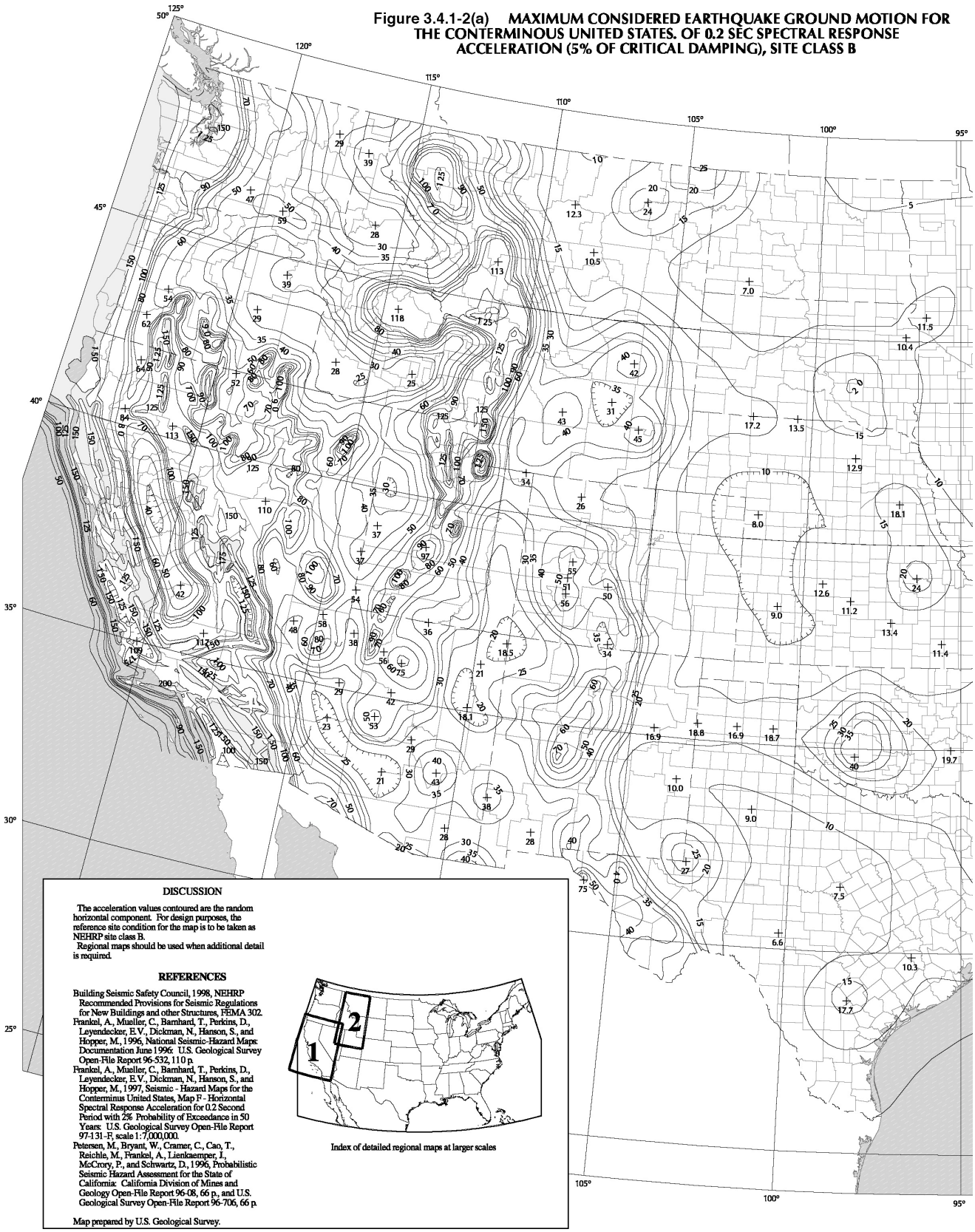


Figure 3.4.1-2(a) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

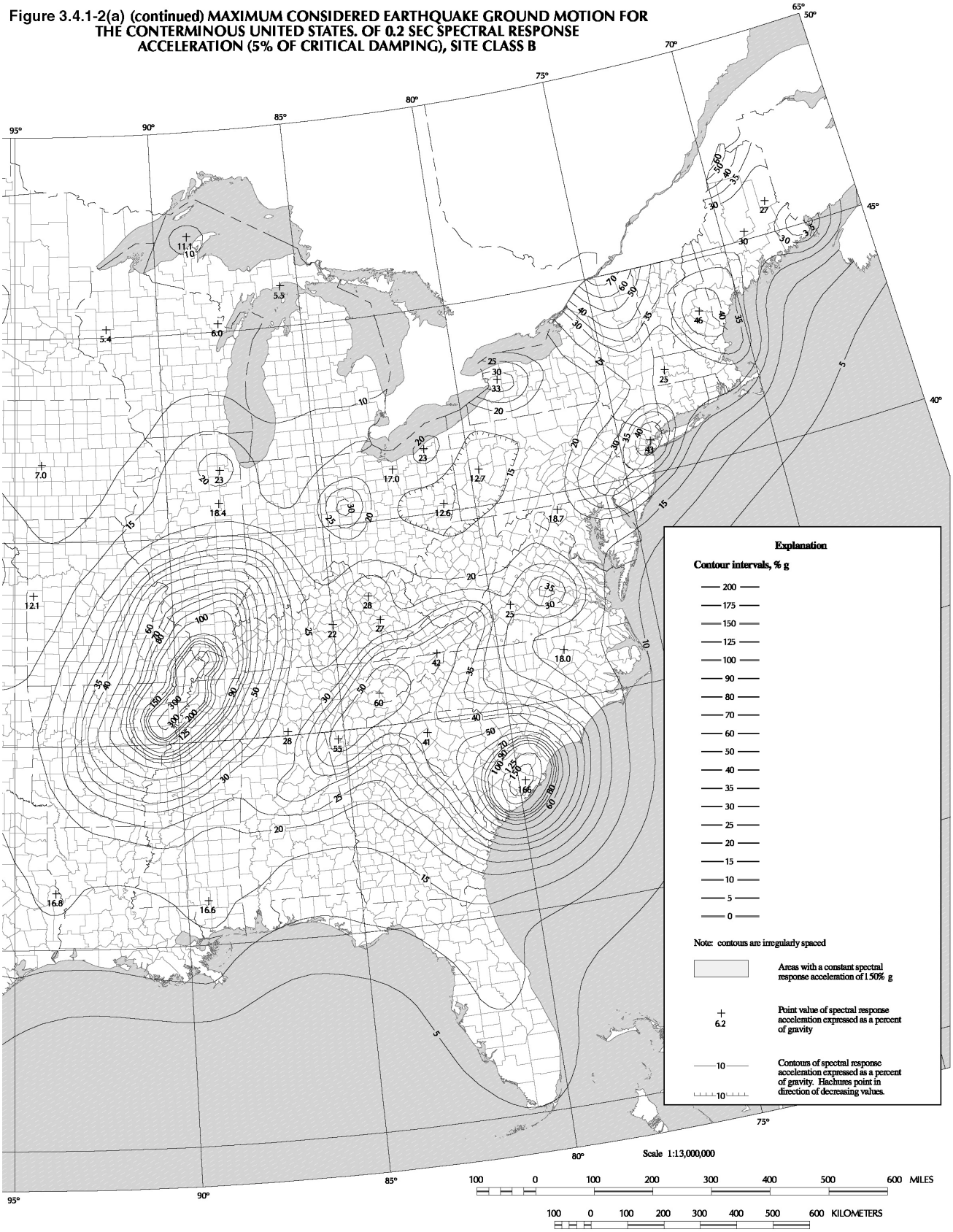


Figure 3.4.1-2(b) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

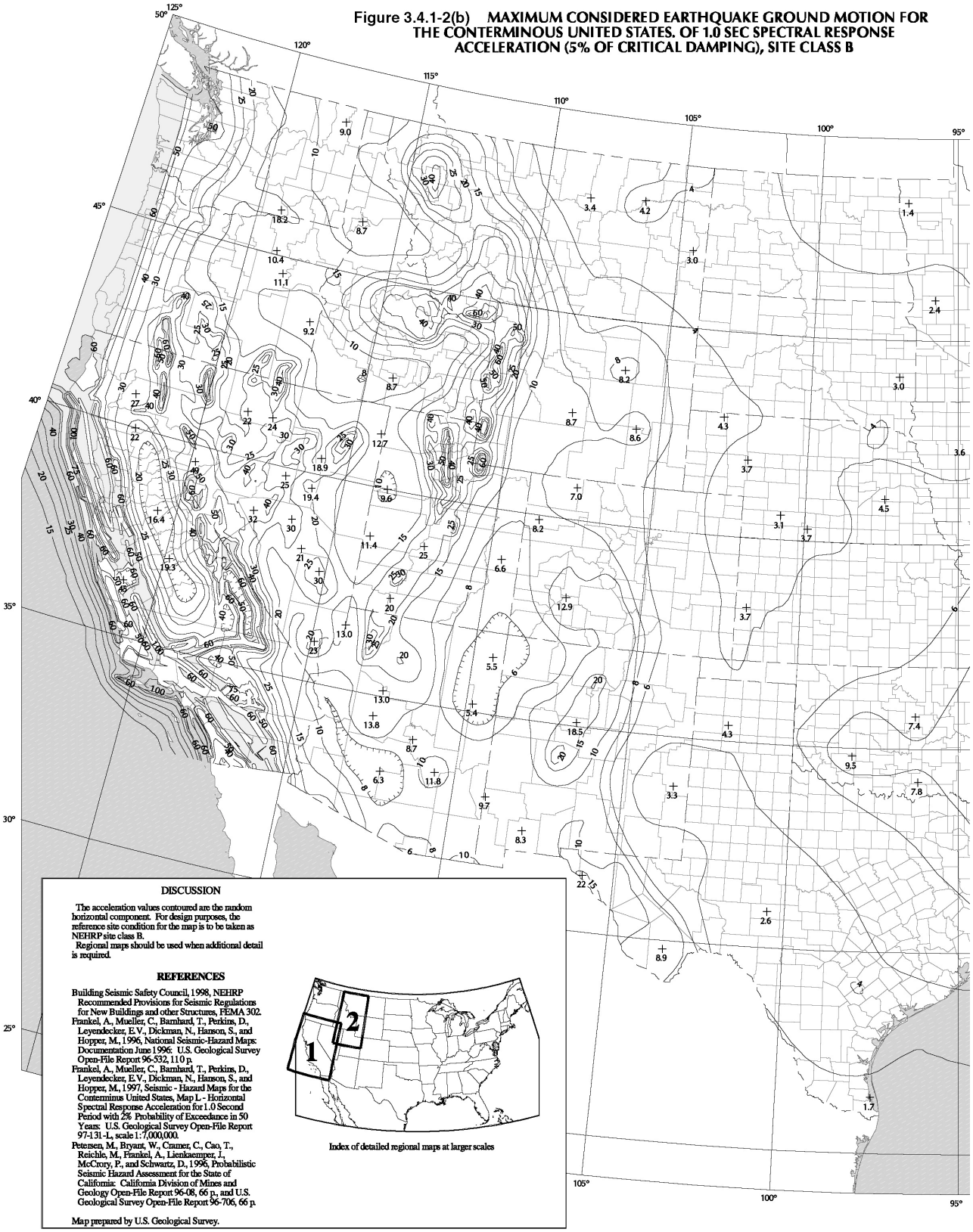
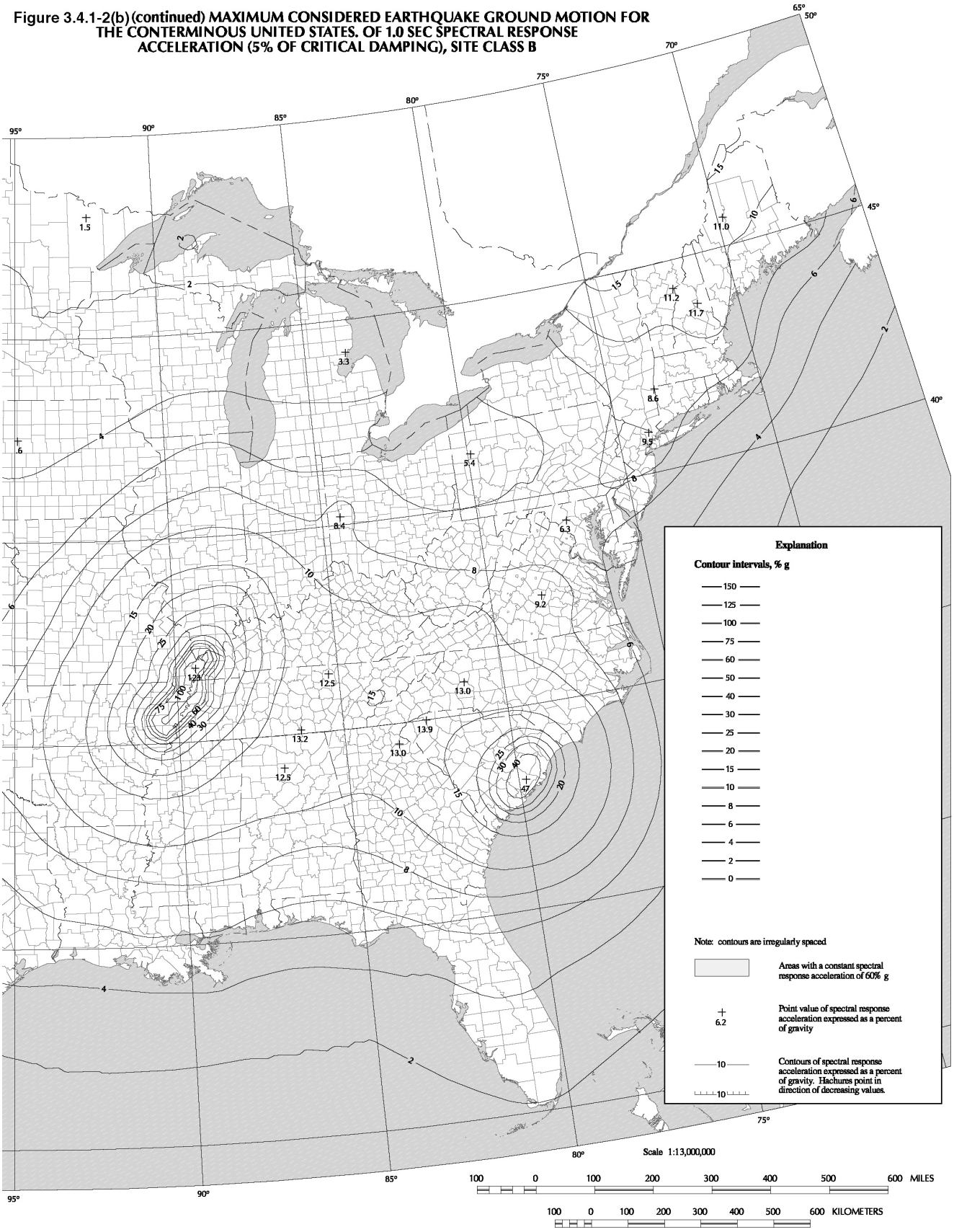


Figure 3.4.1-2(b) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



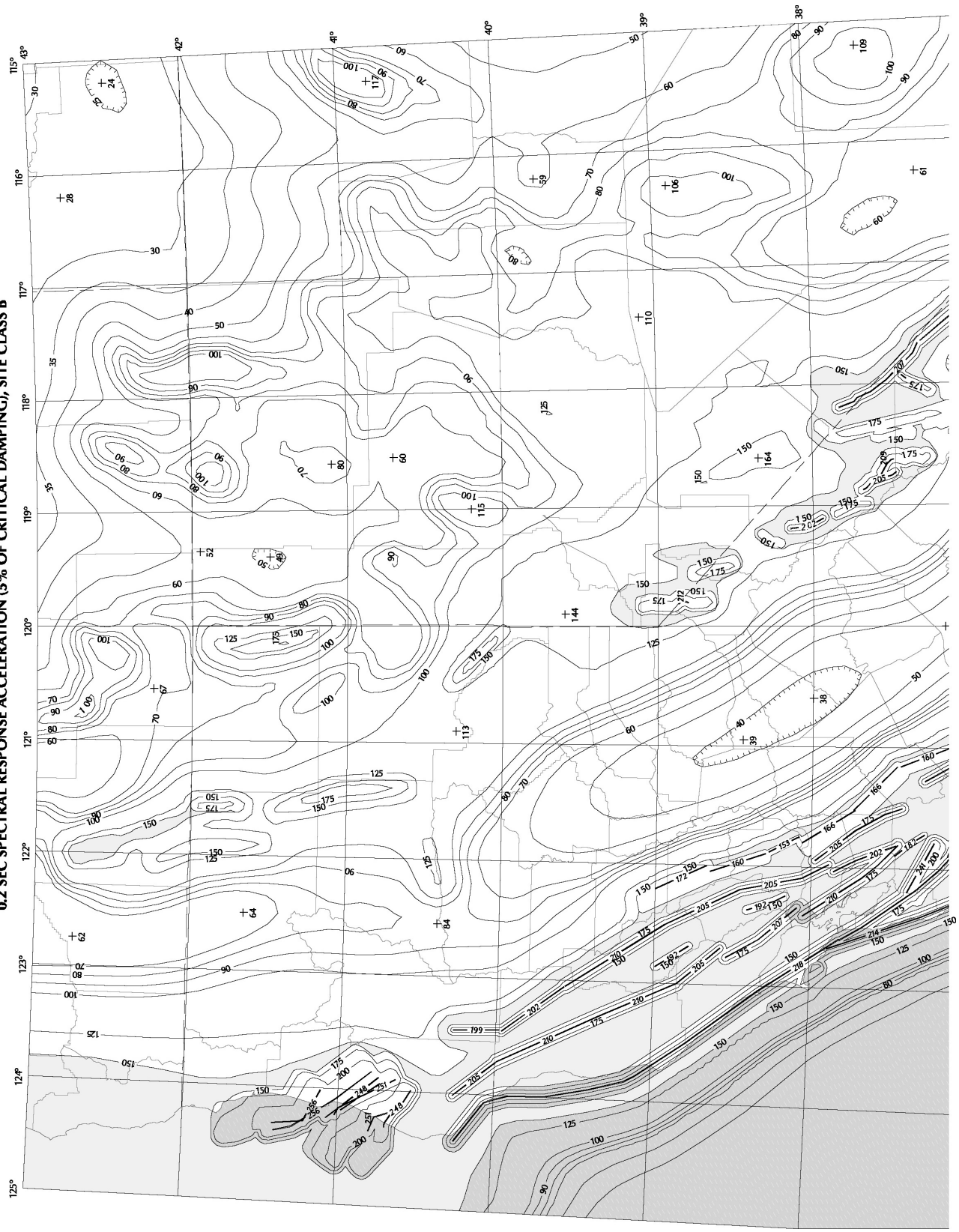


Figure 3.4.1-2(c) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

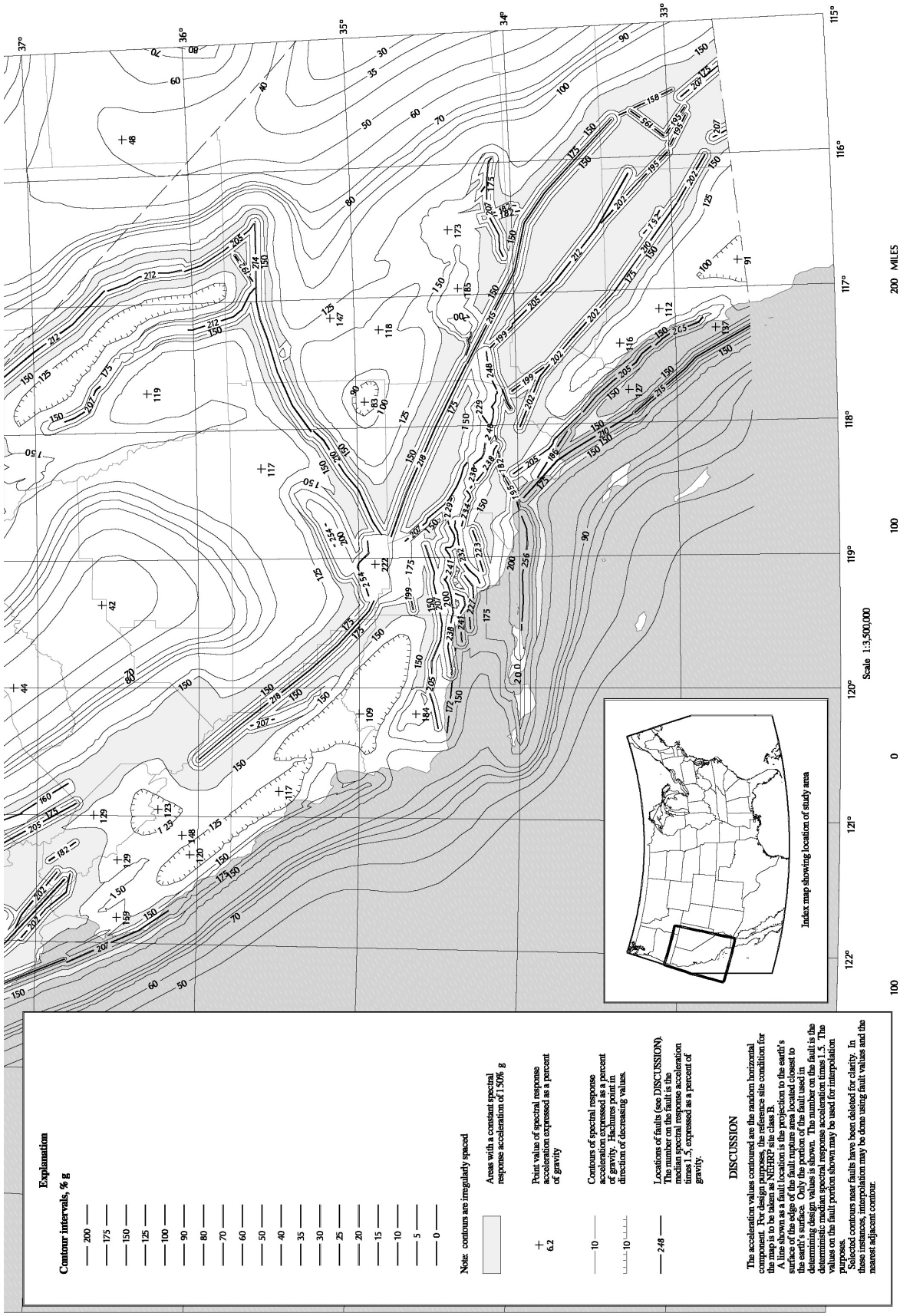


Figure 3.4.1-2(c) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

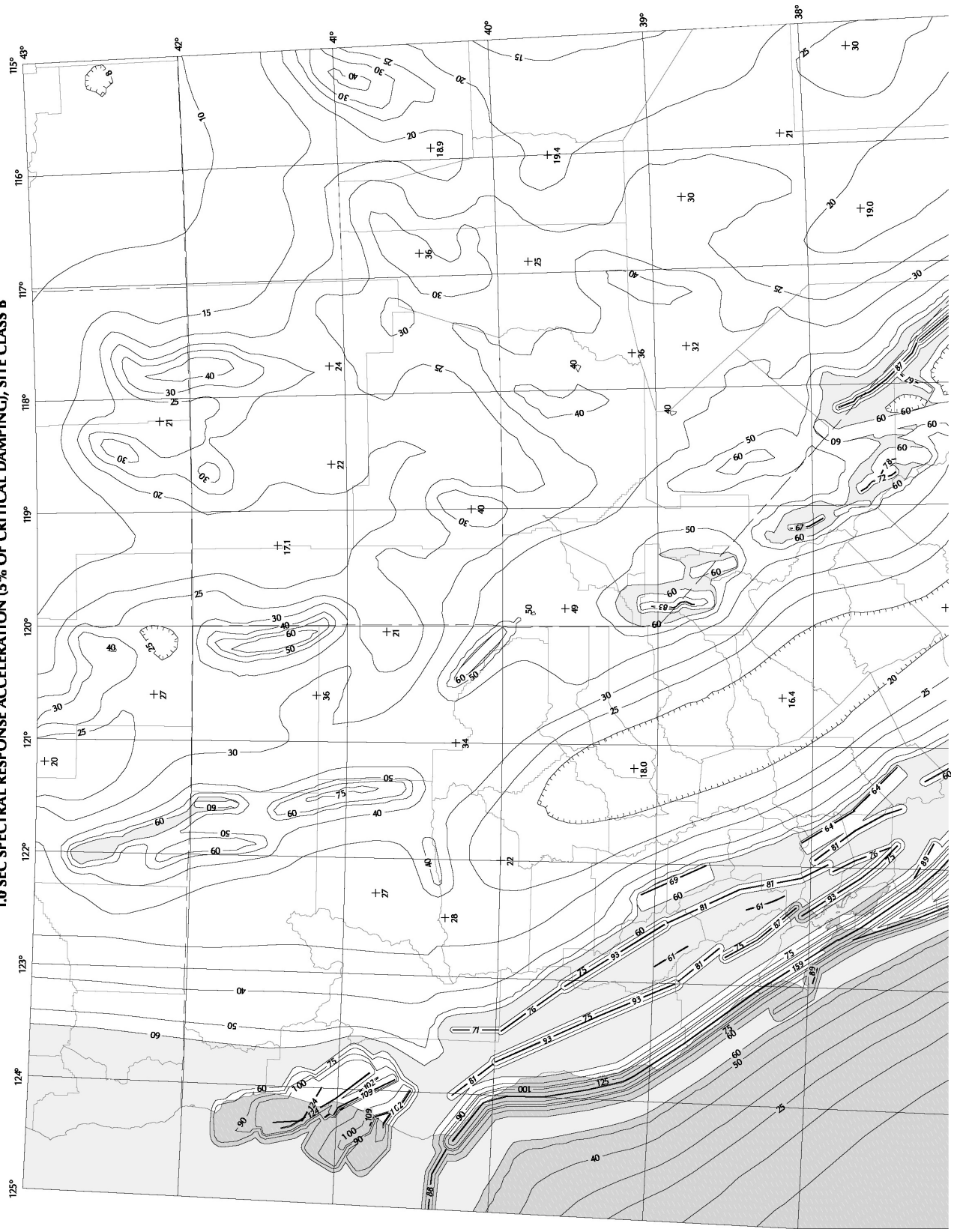


Figure 3.4.1-2(d) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

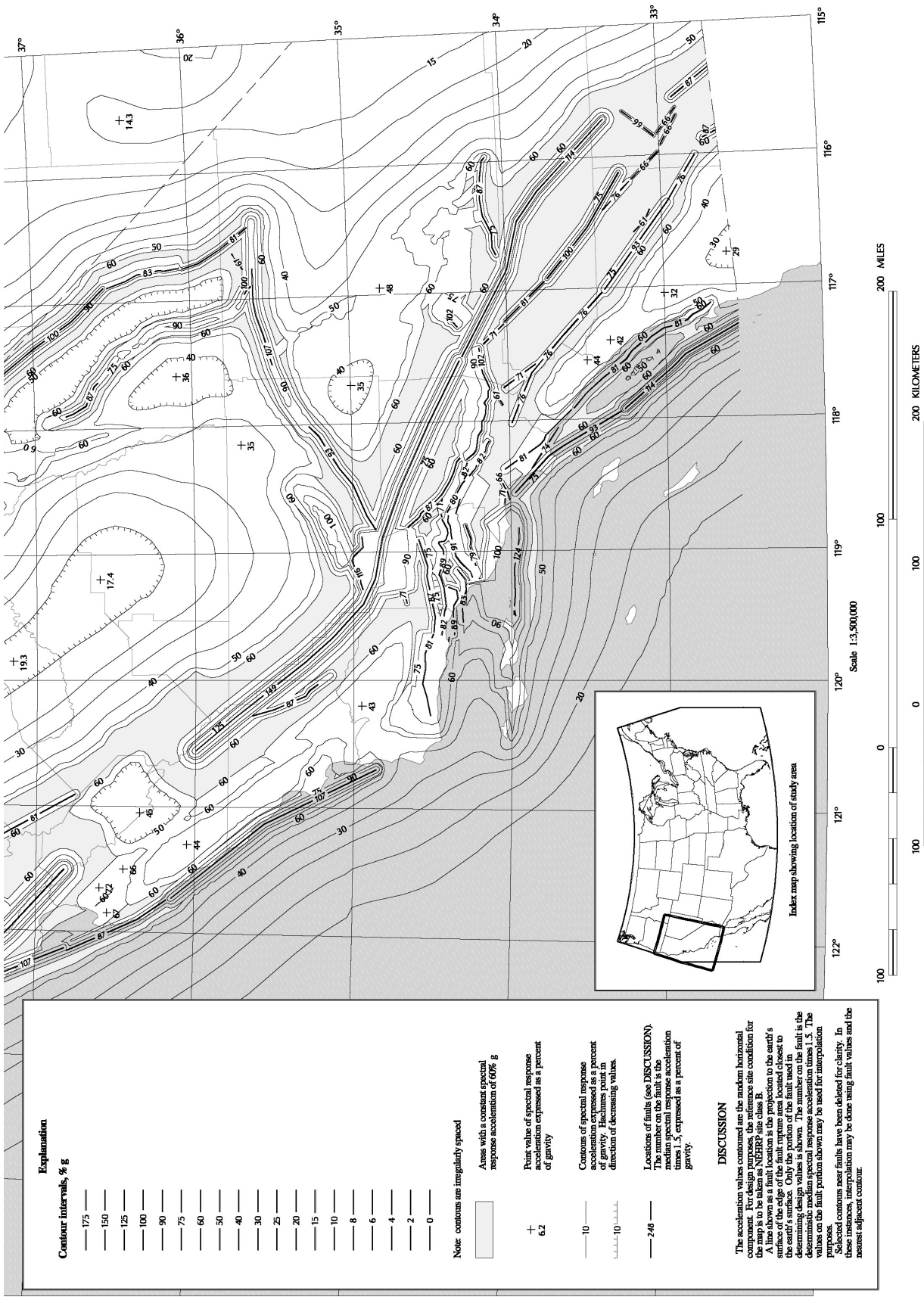
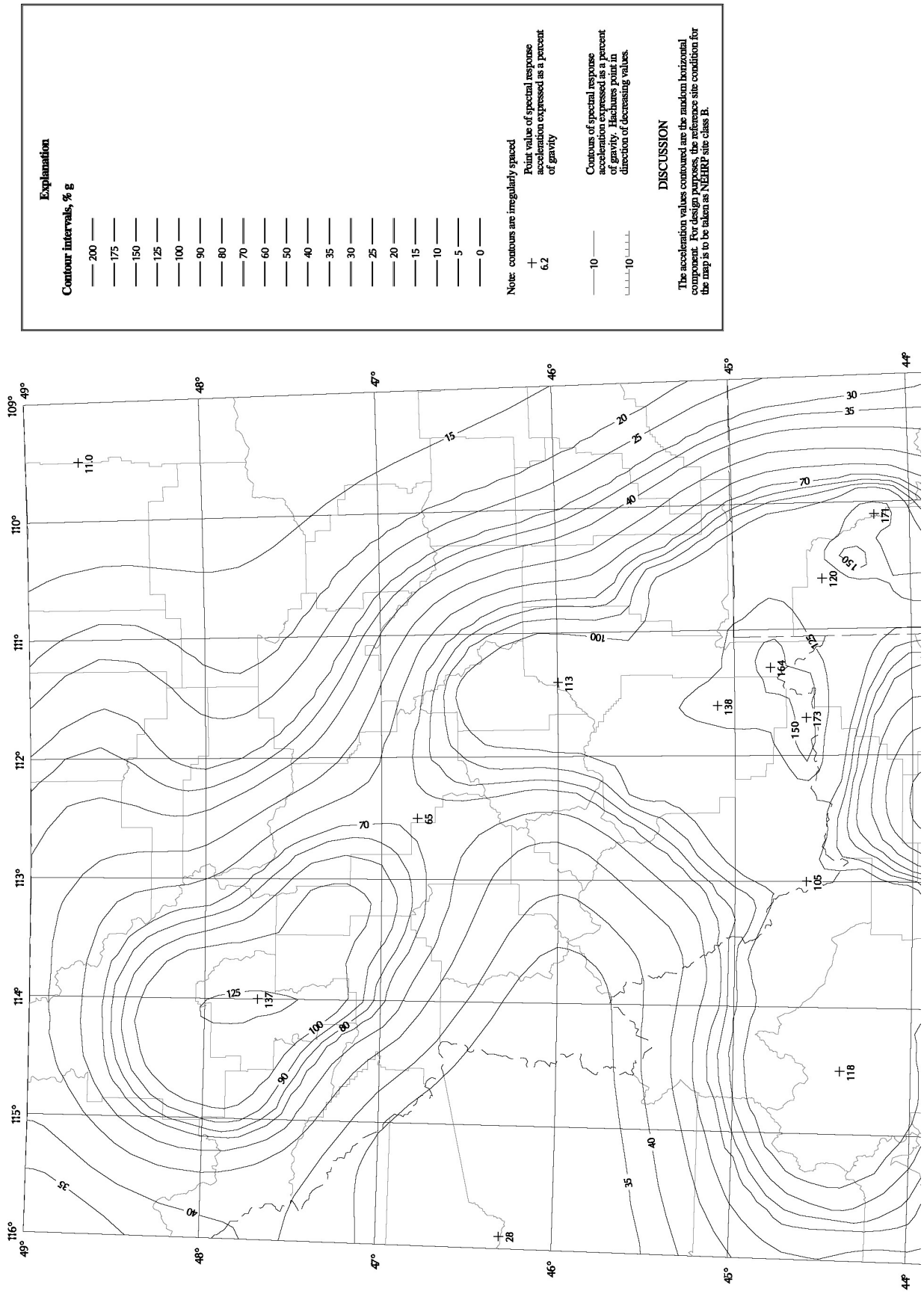


Figure 3.4.1-2(d) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Figure 3.4.1-2(e) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



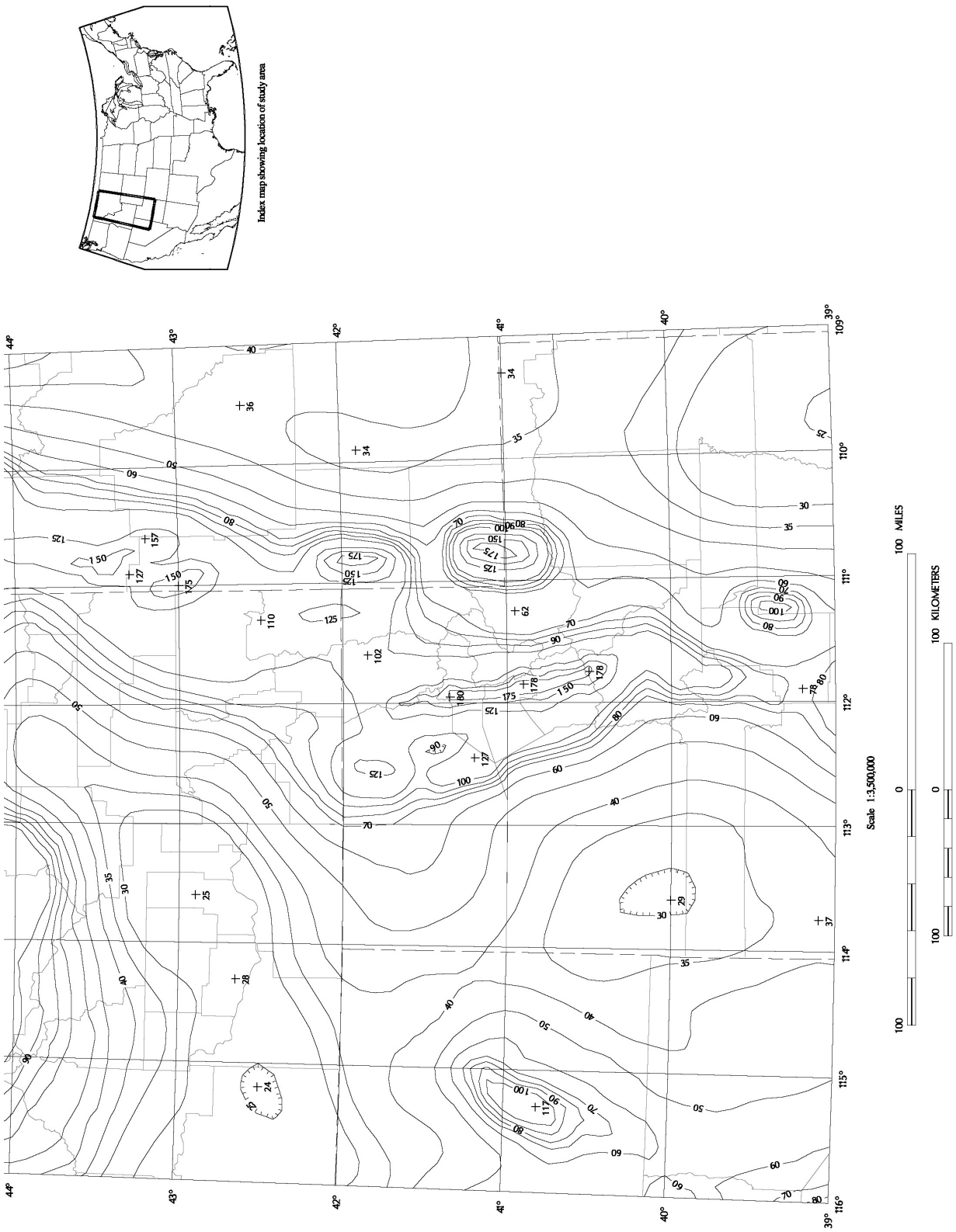
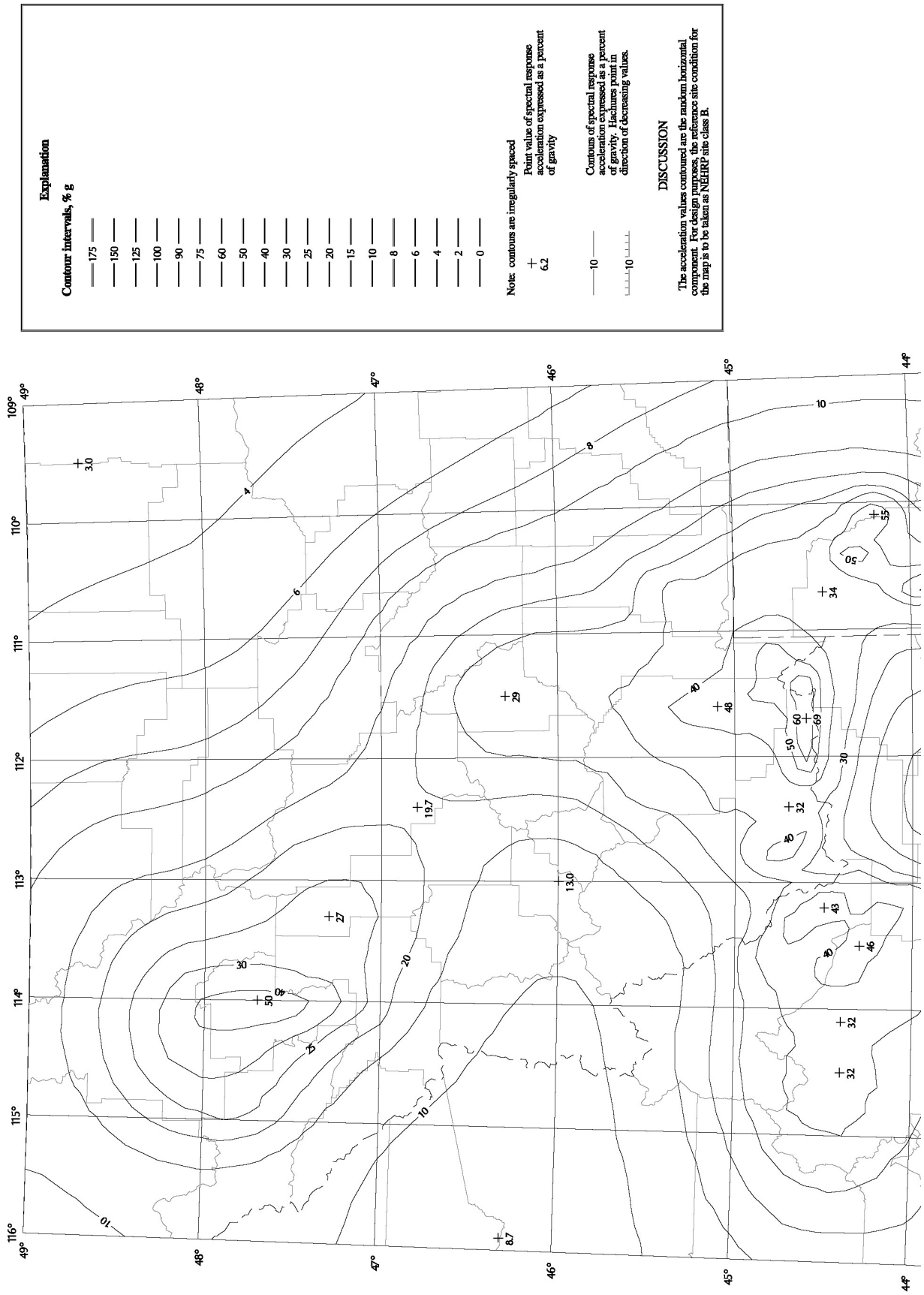


FIGURE 1615 (5) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Figure 3.4.1-2(f) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



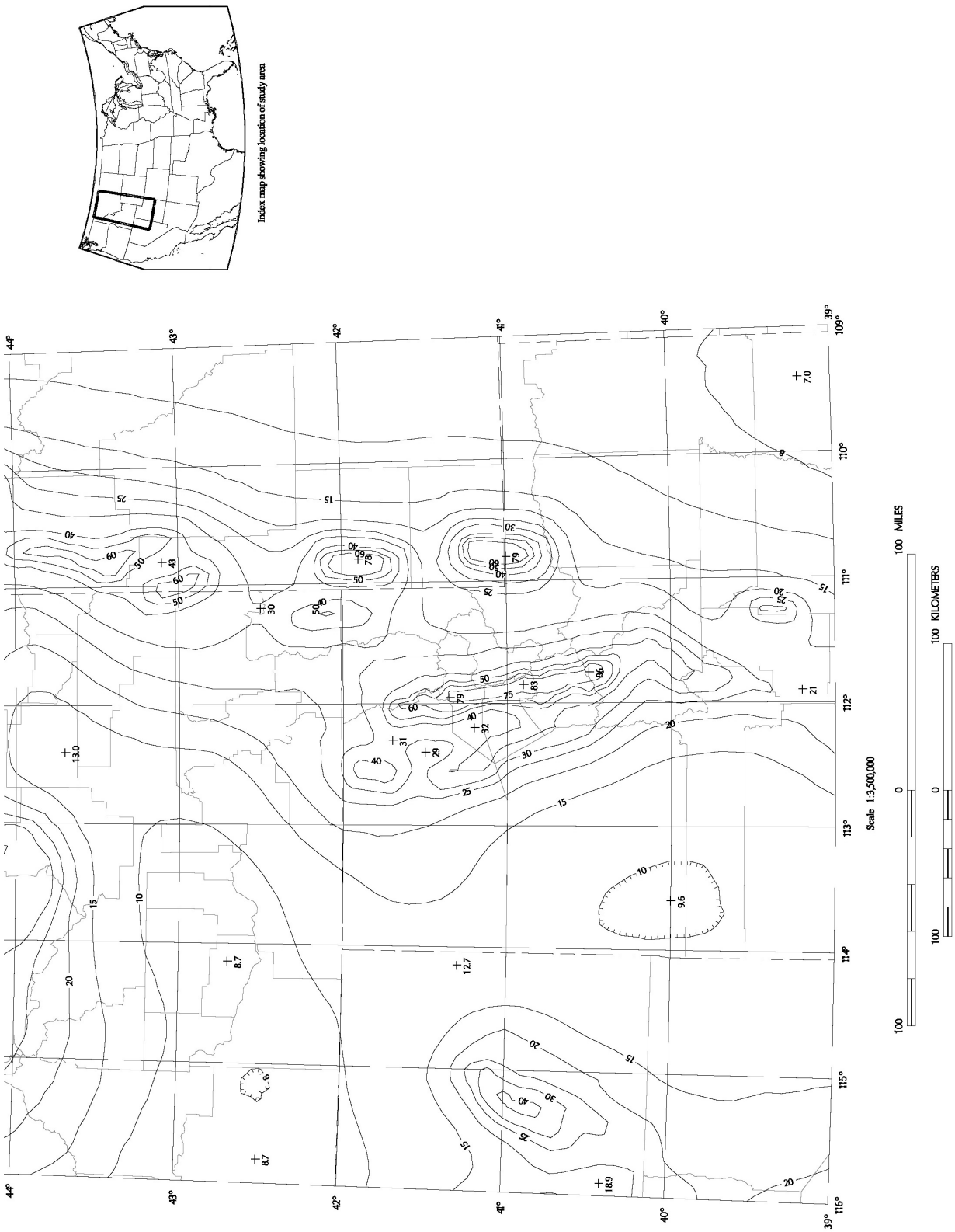


Figure 3.4.1-2(f) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Figure 3.4.1-2(g) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR ALASKA OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

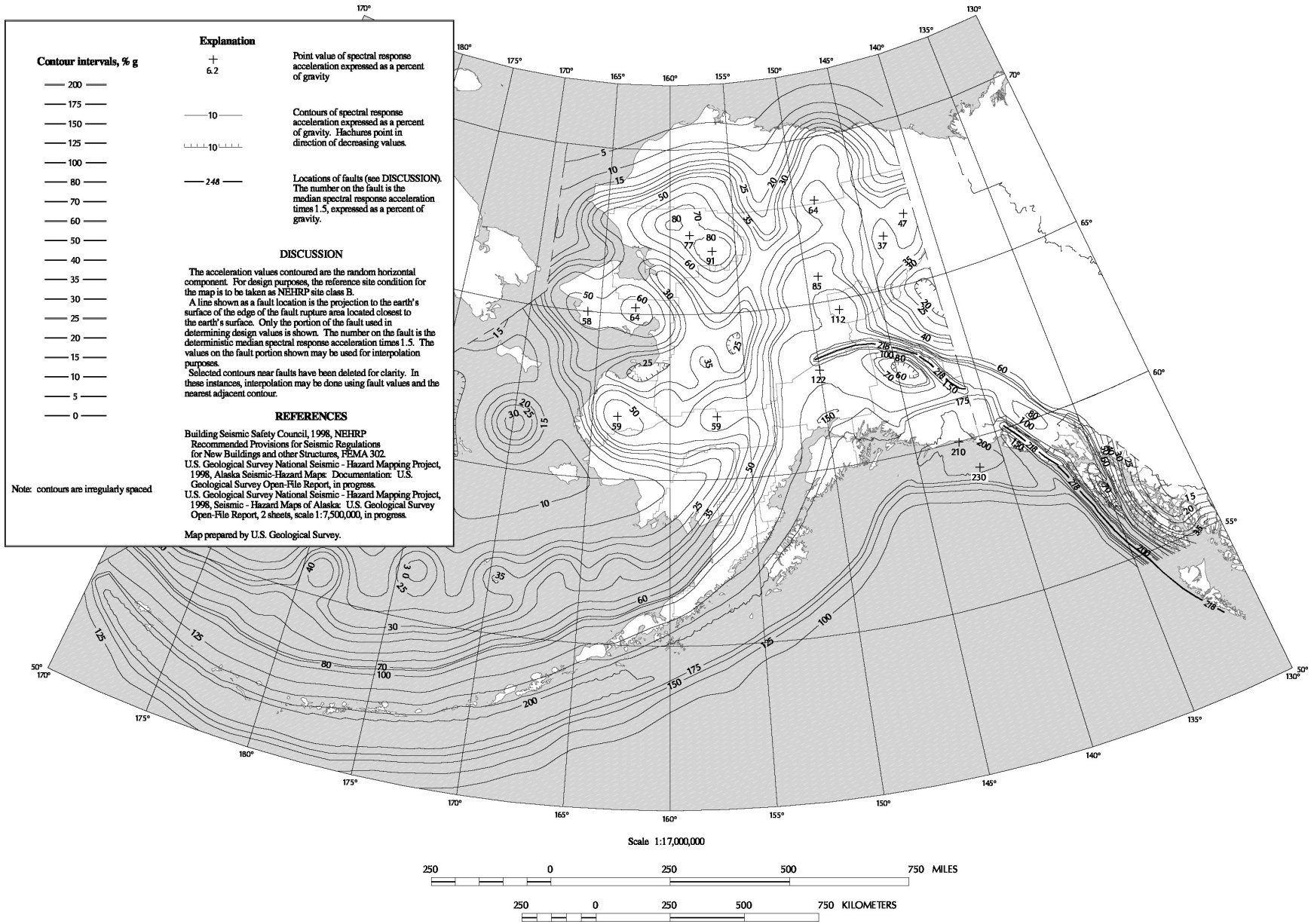


Figure 3.4.1-2(h) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR ALASKA OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

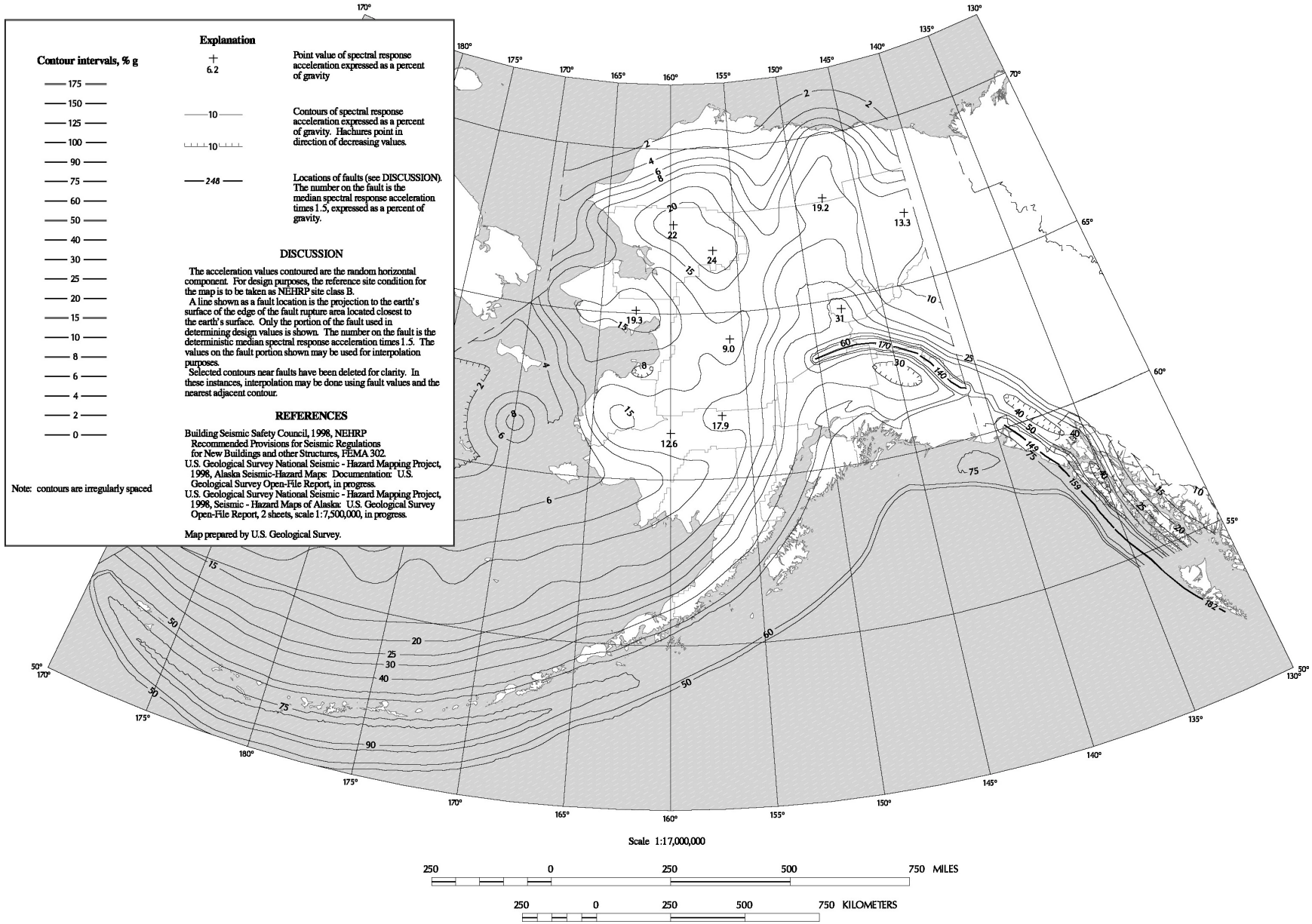


Figure 3.4.1-2(i) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR HAWAII OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

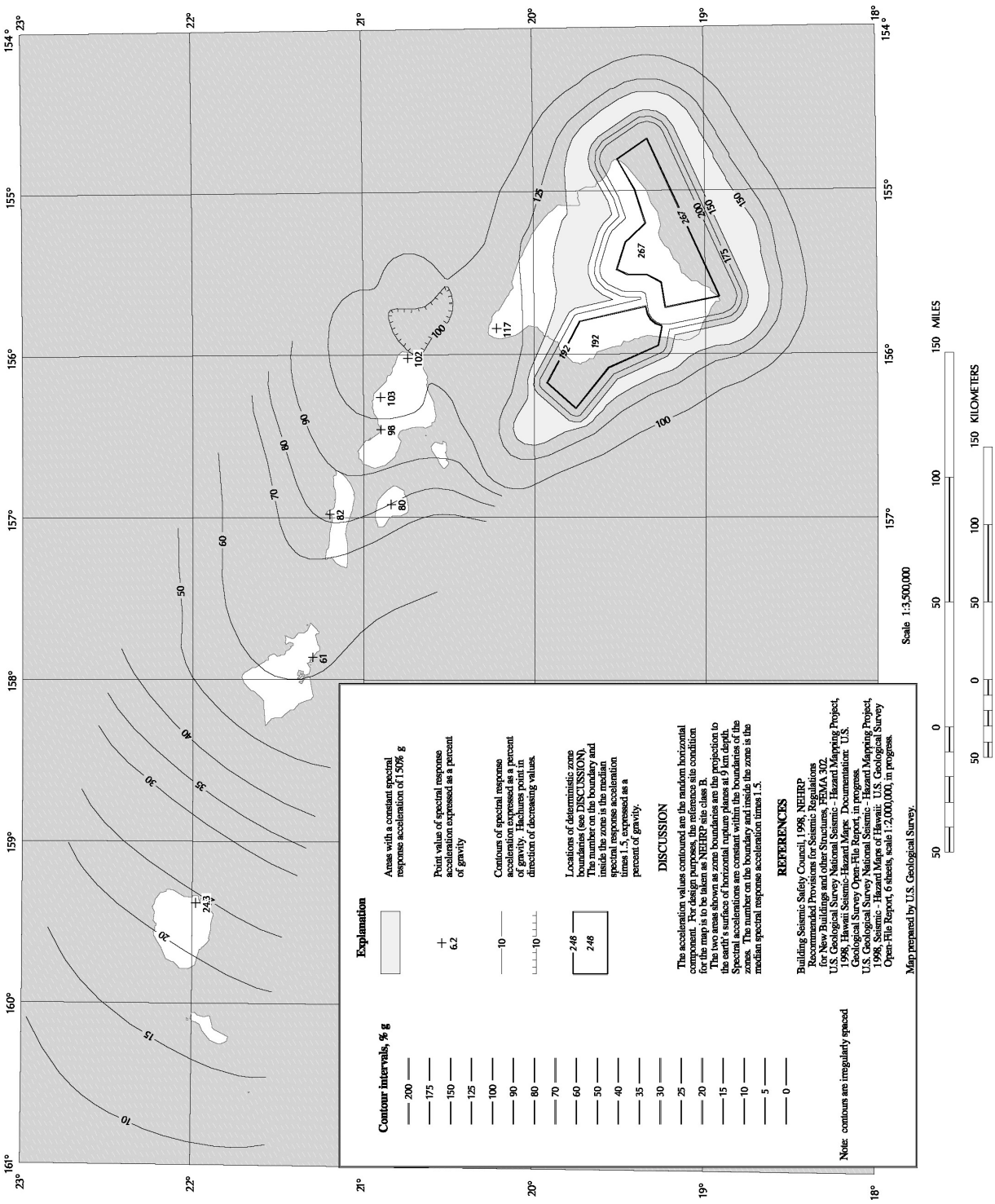


Figure 3.4.1-2(j) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR HAWAII OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

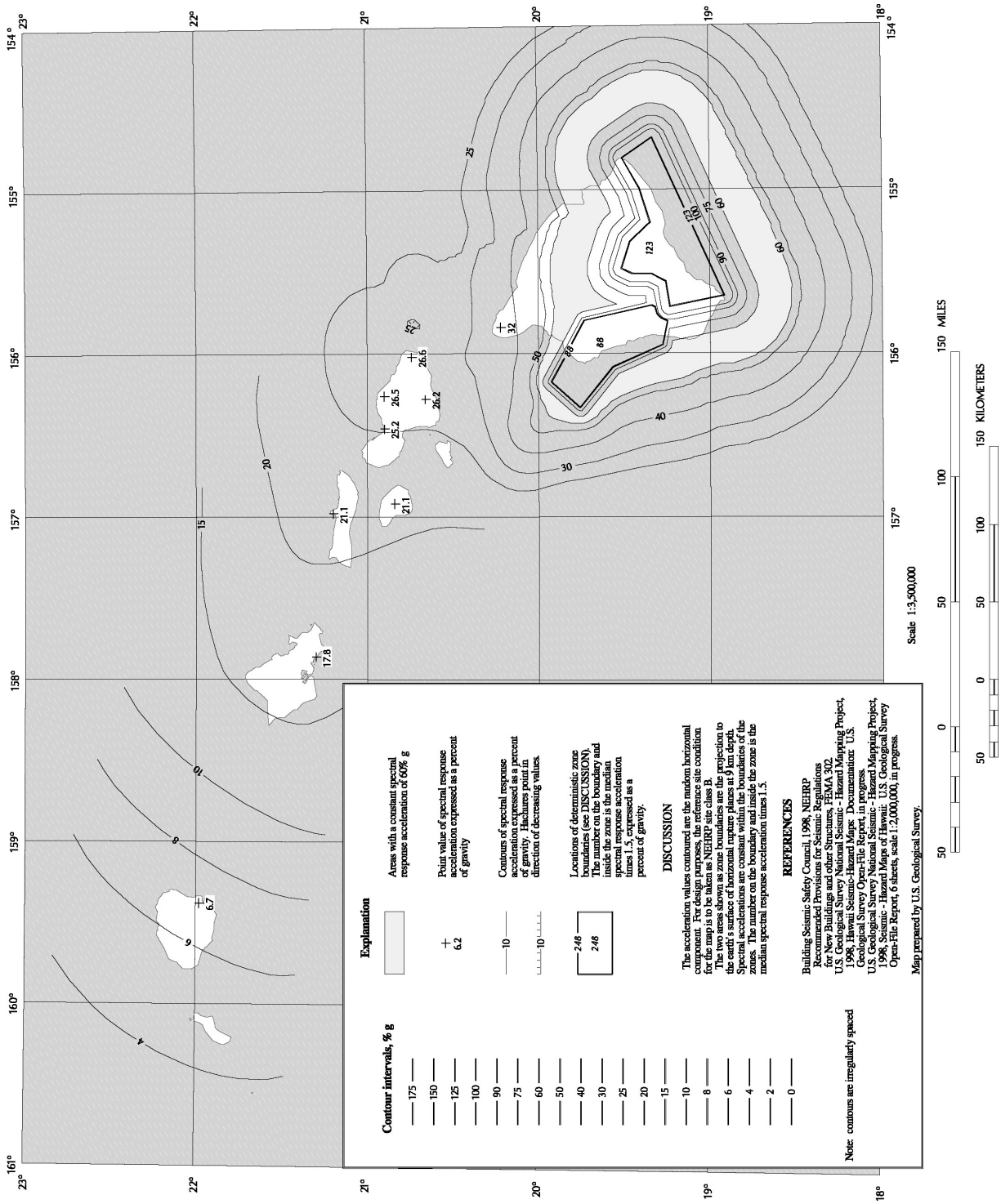
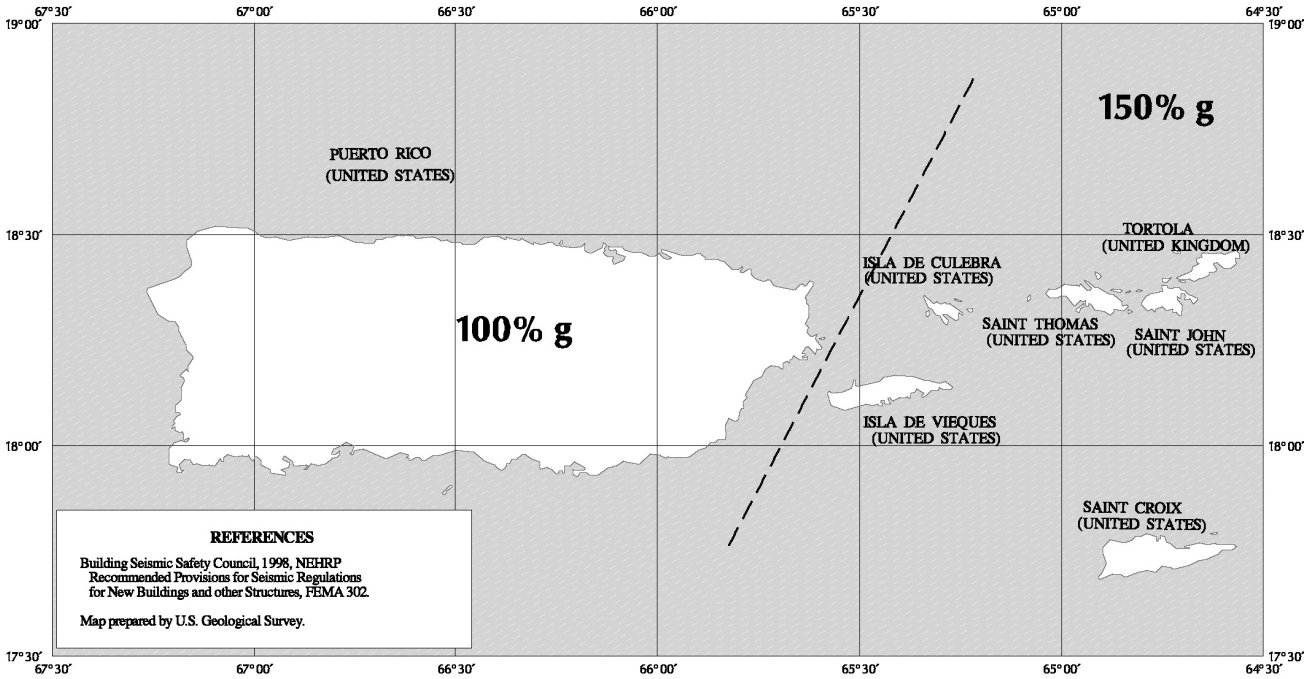
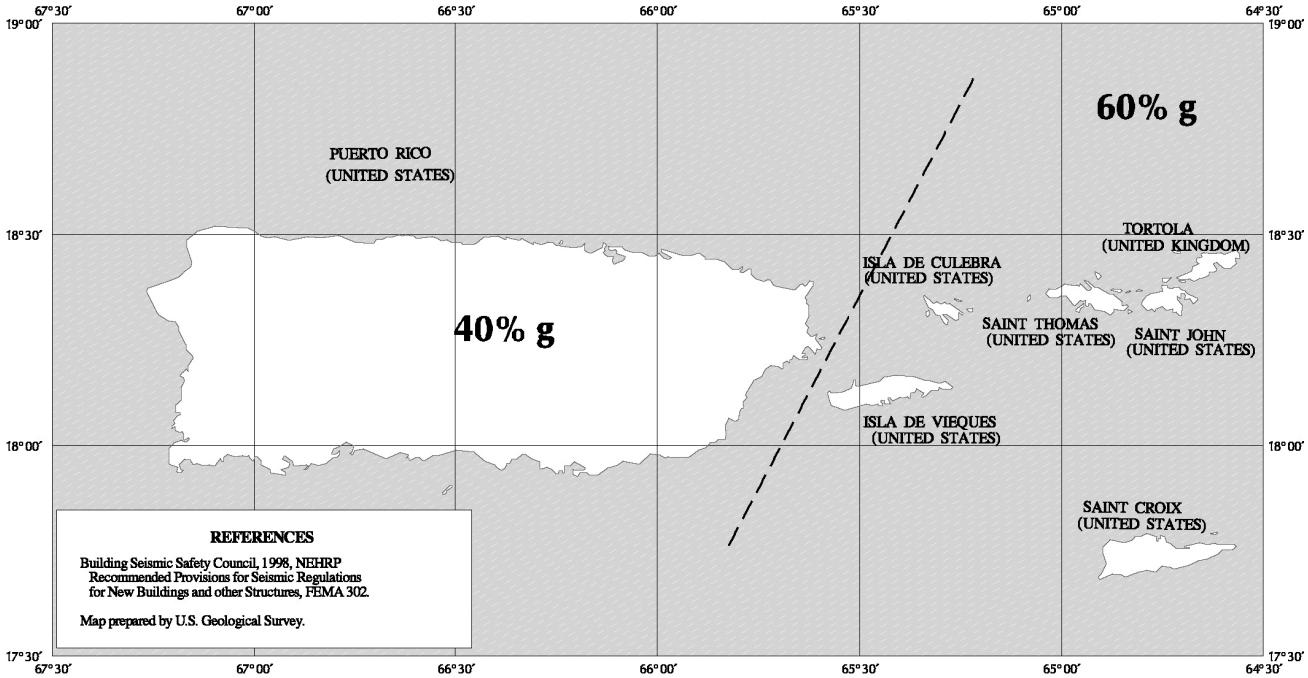


Figure 3.4.1-2(k) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR PUERTO RICO, CULEBRA, VIEQUES, ST. THOMAS, ST. JOHN, AND ST. CROIX OF 0.2 AND 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)

Scale 1:2,000,000

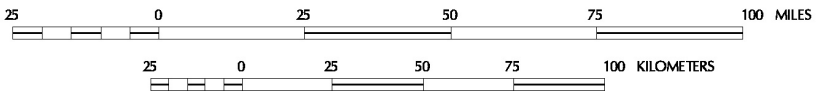
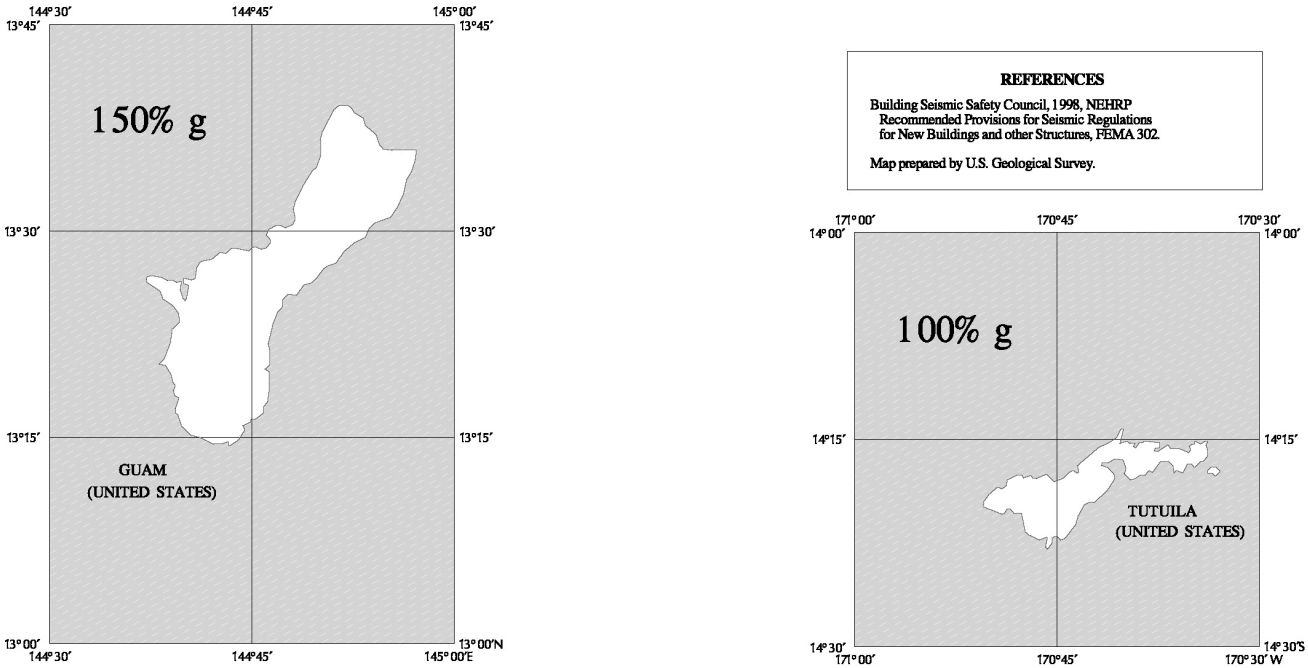
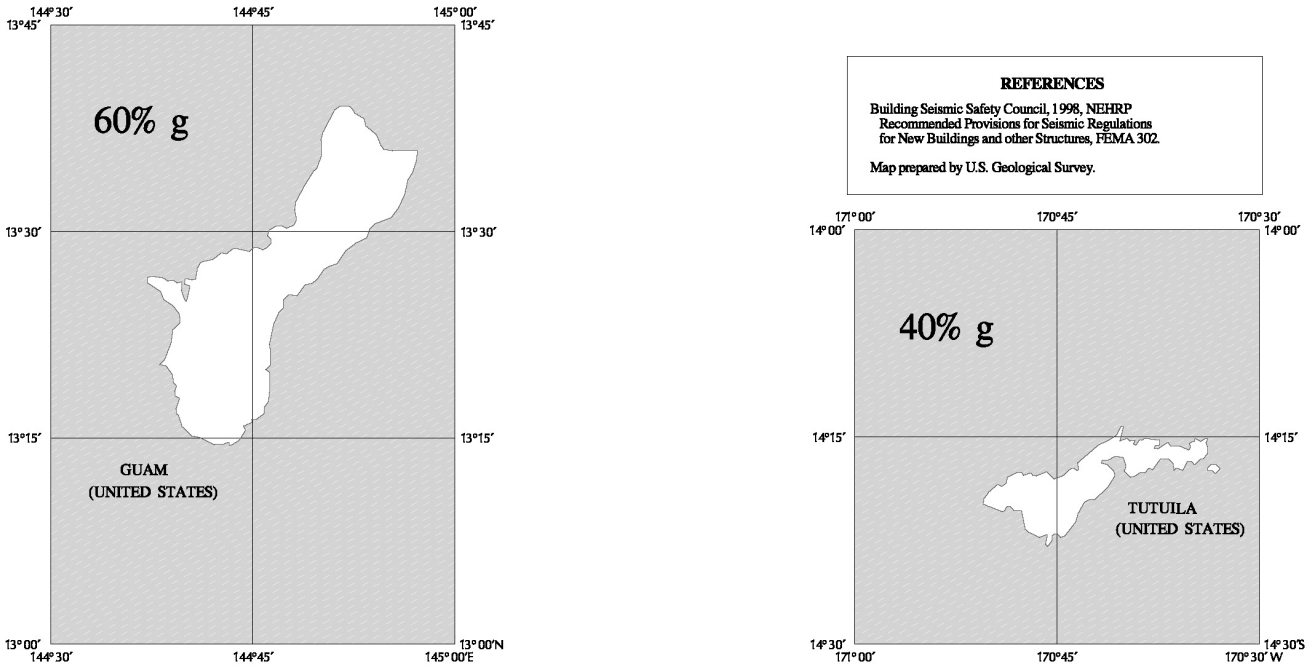


Figure 3.4.1-2(l) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR GUAM AND TUTUILLA OF 0.2 AND 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)

Scale 1:1,000,000

25 0 25 MILES

25 0 25 KILOMETERS

Section 4
DESIGN AND ANALYSIS PROCEDURES (SDAP)

4.1 SINGLE SPAN BRIDGES

For single-span bridges, regardless of seismic zone and in lieu of a rigorous analysis, the minimum design force at the connections in the restrained direction between the superstructure and the substructure shall not be less than the product of $F_a S_S / 2.5$, and the tributary permanent load.

4.2 SDAP A1 AND A2

The design requirements for SDAP A1 and A2 are specified in Section 6. There are no dynamic analysis requirements specified for SDAP A1 and A2

4.3 SDAP B — NO SEISMIC DEMAND ANALYSIS

Bridges qualifying for SDAP B do not require a seismic demand analysis but capacity design principles and minimum design details are required. The capacity design forces are covered in more detail in Article 4.8.

4.3.1 No Analysis Approach

SDAP B consists of the following steps:

- Step 1 - Check Article 4.3.2 for restrictions on structural and site characteristics to determine if SDAP B is applicable. The bridge site must not exceed F_vS_1 limitations and the structure must meet certain regularity requirements as defined in Section 4.3.2.
- Step 2 - Reinforced concrete columns shall be designed using non-seismic loading cases and checked for minimum longitudinal reinforcement (0.8%).
- Step 3 - Reinforced concrete columns shall be detailed to meet the shear, confinement and bar restraint reinforcement requirements of Article 7.8.2.3 through 7.8.2.6 in the plastic hinge zones defined in Article 4.9.
- Step 4 - Steel columns shall be designed using non-seismic loading cases and checked for minimum width to thickness ratios as described in Article 7.7.4. Plastic hinge zone forces shall be those from capacity design procedures of Article 4.8.
- Step 5 -Members connecting to columns shall be designed to resist column plastic moments and shears using the principles of capacity design described in Article 4.8.
- Step 6 - Foundations (soils and piles) shall be designed to resist column moment and shears using the principles of capacity design described in Article 4.8 using an overstrength ratio of 1.0 for all columns.

4.3.2 Limitations

SDAP B shall be used only at sites where:

$$F_v S_1 < 0.4 \cos \alpha_{skew} \qquad (4.3.2\text{-}1)$$

where α_{skew} = the skew angle of the bridge, (0 degrees being the angle for a right bridge).

Additionally, SDAP B shall be used only on structures that comply with the following restrictions:

For concrete column and pile bents

- $P_e < 0.15 f'_c A_g$
- $\rho_\ell > 0.008$
- $D > 300\text{mm}$ (12 inches)
- $\dfrac{M}{VD} < 6$

where P_e = column dead plus seismic live load

f'_c = nominal 28 day concrete strength

A_g = gross cross-sectional area of column
ρ_ℓ = longitudinal reinforcement ratio
D = column transverse dimension
M = maximum column moment
V = maximum column shear

For concrete wall piers with low volumes of longitudinal steel:

- $P_e < 0.1 f'_c A_g$
- $\rho_\ell > 0.0025$
- $\frac{M}{Vt} < 10$
- $t > 300\text{mm (12 inches)}$

where t = wall thickness, or smallest cross-sectional dimension.

For steel pile bents framing into reinforced concrete caps:

- $P_e < 0.15 P_y$
- $D_p \geq 250\text{mm (10 inches)}$
- $L/b < 10$

where D_p = pile dimension about the weak axis bending at ground line.

b = flange width or pipe diameter

P_y = axial yield force of steel pile

L = length from the point of maximum moment to the inflexion point of the column when subjected to a pure transverse load.

For timber piles framing into reinforced concrete caps or steel moment-frame columns:

- $P_e < 0.1 P_c$
- $D_p \geq 250\text{mm (10 inches)}$
- $\frac{M}{VD_p} < 10$

where P_c = axial compression capacity of the pile.

SDAP B shall NOT be used for bridges where:

- Individual interior bent stiffnesses vary by more than a factor of 2 with respect to the average bent stiffness of the bridge.
- The maximum span exceeds 80 m.
- The maximum span length is more than 50 percent longer than the average span length.
- The maximum skew angle exceeds 30 degrees
- For horizontally curved bridges the subtended angle exceeds 30 degrees.
- For frames in which the superstructure is continuous over the bents and for which some bents do not participate in the ERS, F_vS_1 factored by the ratio of the total number of bents in the frame divided by the number of bents in the frame that participate in the ERS in the longitudinal direction exceeds $0.4\cos\alpha_{skew}$
- If the bridge site has a potential for liquefaction and the piers are seated on spread footings.
- The bridge site has a potential for liquefaction and the piers are seated on piled foundations unless the piles shall be detailed for ductility, in accordance with these provisions over the length passing through the liquifiable soil layer plus an additional length of three-pile diameters or 3 m (10 ft) whichever is larger, above and below the liquefiable soil layer.

4.3.3 Capacity Design and Strength Requirements of Members Framing into Columns

Except for the geotechnical design of foundations, SDAP B requires the use of capacity design for all components connected to the columns (Article 4.8). For the geotechnical design of foundations, the moment overstrength capacity of columns that frame into the foundations need not be taken as greater than:

$$M_{po} = 1.0\, M_n$$

Where

M_{po} = plastic overstrength capacity of a column

M_n = nominal moment capacity of a column

4.4 SDAP C — CAPACITY SPECTRUM DESIGN METHOD

4.4.1 Capacity Spectrum Design Approach

SDAP C combines a demand and capacity analysis, including the effect of inelastic behavior of ductile earthquake resisting elements. The procedure applies only to bridges that behave essentially as a single degree-of-freedom system. SDAP C is restricted to bridges with a very regular configuration as described in Article 4.4.2 and with the recommended earthquake resisting systems (ERS) as described in Article 3.3.1.

The major steps in applying the capacity spectrum method for the two levels of earthquake are as follows:

- *Step 1* - Design the bridge for the non-seismic load combinations. Determine the applicability of SDAP C.
- *Step 2* - Check if the design for non-seismic loads satisfies the requirements for the 50% PE in 75-year earthquake event.
- *Step 3* - Design for the 50% PE in 75-year earthquake event if necessary from Step 2.
- *Step 4* - With a design that satisfies the non-seismic load combinations and the 50% PE in 75-year earthquake event, check that the requirements for the 3% PE in 75 year/1.5 mean deterministic earthquake event are satisfied.
- *Step 5* - If necessary from Step 4, modify the design to satisfy the requirements for the 3% PE in 75-year/1.5 mean deterministic event.
- *Step 6* - Design and detail the columns, the connections of the columns to the foundation, and superstructure or column bent using the capacity design procedures of Article 4.8. For bridges in SDR 3, the requirements of Article 4.3.3 are applicable. If the connection capacity design forces are excessive, then SDAP D shall be used to determine the elastic connection forces.

Details for each of these steps are discussed in the Commentary.

4.4.2 Limitations

SDAP C shall only be used on bridges that satisfy the following requirements:

- The number of spans per frame or unit shall not exceed six.
- The number of spans per frame or unit shall be at least three, unless seismic isolation bearings are utilized at the abutments.
- Abutments shall not be assumed to resist significant seismic forces in the transverse or longitudinal directions.
- Span length shall not exceed 60 m (200 feet).
- The ratio of span lengths in a frame or unit shall not exceed 1.5.
- Pier wall substructures must have bearings that permit transverse movement.
- The maximum skew angle shall not exceed 30 degrees, and skew of piers or bents shall not differ by more than 5 degrees in the same direction.
- For horizontally curved bridges, the subtended angle of the frame of all the bridge types shall not exceed 20 degrees.
- The ratio of bent or pier stiffness shall not vary by more than 2 with respect to the average bent stiffness, including the effect of foundation stiffness.
- The ratio of lateral strength (or seismic coefficient) shall not exceed 1.5 of the average bent strength.
- For concrete columns and pile bents:
  - $P \leq 0.20 f'_c A_g$
  - $\rho_\ell > 0.008$
  - $D \geq 300\text{mm}$ (12 inches)
- When liquefaction potential is determined to exist according to the requirements in Article 7.6 or 8.6, the piers or bents must have pile foundations.

Bridges that satisfy the above and have elastomeric, sliding, or isolation bearings at each pier and abutment shall use the provisions of Article 5.4.1.1.

4.5 SDAP D — ELASTIC RESPONSE SPECTRUM METHOD

SDAP D is a one step design procedure using an elastic (cracked section properties) analysis. Either the Uniform Load or Multimode method of analysis may be used as per Article 5.4.2. The analysis shall be performed for the governing design spectra (either the 50% PE in 75-year or the 3% PE in 75-year/1.5 mean deterministic) and the R-Factors given in Tables 4.7-1 and 4.7-2 shall be used to modify elastic response values. The analysis shall determine the elastic moment demand at all plastic hinge locations in the columns. Capacity design principles shall be used for column shear design and the design of all column connections and foundation design. If sacrificial elements are part of the design (i.e. shear keys) they shall be sized to resist the 50% PE in 75-year forces and the bridge shall be capable of resisting the 3% PE in 75-year/1.5 mean deterministic forces without the sacrificial elements (i.e. two analyses are required if sacrificial elements exist in a bridge).

This design procedure consists of the following steps:

- *Step 1* - Design the bridge for non-seismic loading conditions.
- *Step 2* - Perform an elastic dynamic analysis as described in Article 5.4.2 for the 3% PE in 75-year/1.5 mean deterministic earthquake loading to determine displacement demands. Analysis shall reflect the anticipated condition of the structure and the foundation during this earthquake.
- *Step 3* - Determine controlling seismic design forces for the moment design of all columns from an elastic dynamic analysis using either the 50% PE in 75- or 3% PE in 75-year/1.5 mean deterministic earthquake. Analyses shall reflect the anticipated condition of the structure and the foundation during each of these earthquakes. Elastic forces from the analyses shall be modified using the appropriate R-Factors from Tables 4.7-1 and 4.7-2.
- *Step 4* – Check the minimum design base shear for each column using the *P-Δ* requirements from Article 7.3.4 or 8.3.4 using the elastic displacements obtained in Step 2. Modify column design as necessary.
- *Step 5* - Determine the design forces for other structural actions using capacity design principles as described in Article 4.8.
- *Step 6* - Design sacrificial elements to resist forces generated by the 50% PE in 75-year earthquake.

4.6 SDAP E — ELASTIC RESPONSE SPECTRUM METHOD WITH DISPLACEMENT CAPACITY VERIFICATION

SDAP E requires an elastic (cracked section properties) response spectrum analysis for the governing design spectra (50% PE in 75-year or 3% PE in 75-year/1.5 mean deterministic) and *P-Δ* design check. The results of these analyses shall be used to perform preliminary flexural design of plastic hinges in columns and to determine the displacement of the structure. To take advantage of the higher R-Factors in Table 4.7-1, displacement capacities shall be verified using two-dimensional nonlinear static (pushover) analyses in the principal structural directions. Design forces on substructure elements may be reduced below those obtained for the 3% PE in 75-year event/1.5 mean deterministic divided by the R-Factor, as described in Step 2 below. Capacity design principles of Article 4.8 shall be used to design the connection of the columns to the superstructure and foundation and for column shear design. SDAP E is required when owner approved ERE are used that have inelastic action that cannot be inspected.

This design procedure shall consist of the following steps:

- *Step 1* - Perform Steps 1 through 4 for SDAP D except that the appropriate R-Factors from Tables 4.7-1 and 4.7-2 shall be used.
- *Step 2* - Perform a displacement capacity verification analysis using the procedures described in Article 5.4.3. If sufficient displacement capacity exists the substructure design forces may be further reduced from those of Step 1, but not less than 70% of the Step 1 forces nor less than design forces from the 50% PE in 75-year event. If column sizes are reduced, repeat Step 2 of SDAP D and these displacements shall be used in a repeat of this step in SDAP E.
- *Step 3* - Perform Steps 5 and 6 for SDAP D.

Table 4.7-1 Base Response Modification Factors , R_B, for Substructure

Substructure Element	Performance Objective			
	Life Safety		Operational	
	SDAP D	SDAP E	SDAP D	SDAP E
Wall Piers – larger dimension	2	3	1	1.5
Columns – Single and Multiple	4	6	1.5	2.5
Pile Bents and Drilled Shafts – Vertical Piles – above ground	4	6	1.5	2.5
Pile Bents and Drilled Shafts – Vertical Piles – 2 diameters below ground level-No owners approval required.	1	1.5	1	1
Pile Bents and Drilled Shafts – Vertical Piles – in ground - Owners approval required.	N/A	2.5	N/A	1.5
Pile Bents with Batter Piles	N/A	2	N/A	1.5
Seismically Isolated Structures	1.5	1.5	1	1.5
Steel Braced Frame – Ductile Components	3	4.5	1	1.5
Steel Braced frame – Nominally Ductile Components	1.5	2	1	1
All Elements for Expected Earthquake	1.3	1.3	0.9	0.9

Notes:

1. The substructure design forces resulting from the elastic analysis divided by the appropriate R-Factor for SDAP E cannot be reduced below 70% of these R-Factor reduced forces or the 50% PE in 75 year design forces as part of the pushover analysis.
2. There maybe design situations (e.g architecturally oversized columns) where a designer opts to design the column for an R=1.0 (i.e. elastic design) – Article 4.10. In concrete columns the associated elastic design shear force may be obtained from the elastic analysis forces using an R-Factor of 0.67 or by calculating the design shear by capacity design procedures using a flexural overstrength factor of 1.0. In steel braced frames if an R=1.0 is used the connection design forces shall be obtained using an R=0.67. If an R=1.0 is used in any design the foundations shall be designed for the elastic forces plus the SDR 2 detailing requirements are required for concrete piles. (i.e. minimum shear requirements) – Article 4.10.
3. Unless specifically stated, the R-Factors apply to both steel and concrete.
4. N/A in this case means that owners approval is required and thus SDAP E is required to use this design option.

Table 4.7-2 Response Modification Factors, R — Connections

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

Note: These factors are not intended for those cases where capacity design principles are used to develop the design forces to design the connections.

4.7 RESPONSE MODIFICATION FACTORS

Structures that are designed using SDAP D or E shall use the response modification factors defined in this article.

To apply the R-Factors specified herein, the structural details shall satisfy the provisions of Articles 7.7 and 7.8 or 8.7 and 8.8 as appropriate.

Except as noted herein, seismic design force effects for flexural design of the primary plastic hinges in substructures shall be determined by dividing the force effects resulting from elastic analysis by the appropriate R-Factor, R, as given by

$$R = 1 + (R_B - 1)\frac{T}{1.25T_s} \le R_B \qquad (4.7\text{-}1)$$

where R_B is given in Table 4.7-1., T is the period of vibration of the bridge, and T_s is defined in Figure 3.4.1-1.

4.8 CAPACITY DESIGN

Capacity design principles require that those elements not participating as part of the primary energy dissipating system (flexural hinging in columns), such as column shear, joints and cap beams, spread footings, pile caps and foundations be "capacity protected". This is achieved by ensuring the maximum moment and shear from plastic hinges in the columns (overstrength) can be dependably resisted by adjoining elements.

Exception: Elastic design of all substructure elements (Article 4.10), seismic isolation design (Article 7.10 and 8.10) and in the transverse direction of a column when a ductile diaphragm (Article 7.7.8 or 8.7.8) is used.

4.8.1 Inelastic Hinging Forces

Inelastic hinges shall form before any other failure due to overstress or instability in the structure and/or in the foundation. Except for pile bents and drilled shafts, and with owners' approval, inelastic hinges shall only be permitted at locations in columns where they can be readily inspected and/or repaired.

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 yielding members. Except for the geotechnical aspects of design of foundations in SDR 3, the moment overstrength capacity (M_{po}) of column/pier/pile members that form part of the primary mechanism resisting seismic loads shall be assessed using one of the following approaches:

- $M_{po} = \lambda_{mo} M_n$ where
  λ_{mo} = 1.5 for concrete columns
  = 1.2 for steel columns
  = 1.3 for concrete filled steel tubes
  = 1.5 for steel piles in weak axis bending and, for steel members in shear (e.g. eccentrically braced frames),
  = 1.0 for geotechnical design forces in SDR 3

  where M_n is the nominal moment strength in which expected yield strengths are used for steel members (Articles 7.7.2 and 8.7.2) and λ_{mo} is the overstrength factor.
- For reinforced concrete columns the plastic analysis approach given by Article 7.8.2.8.
- For reinforced concrete columns a compatibility section analysis, taking into account the expected strengths of the materials and the confined concrete properties and the strain hardening effects of the longitudinal reinforcement.

These 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 developing. Typical methods of applying capacity design at a bent in the longitudinal and transverse directions are shown in Figure 4.8.1-1.

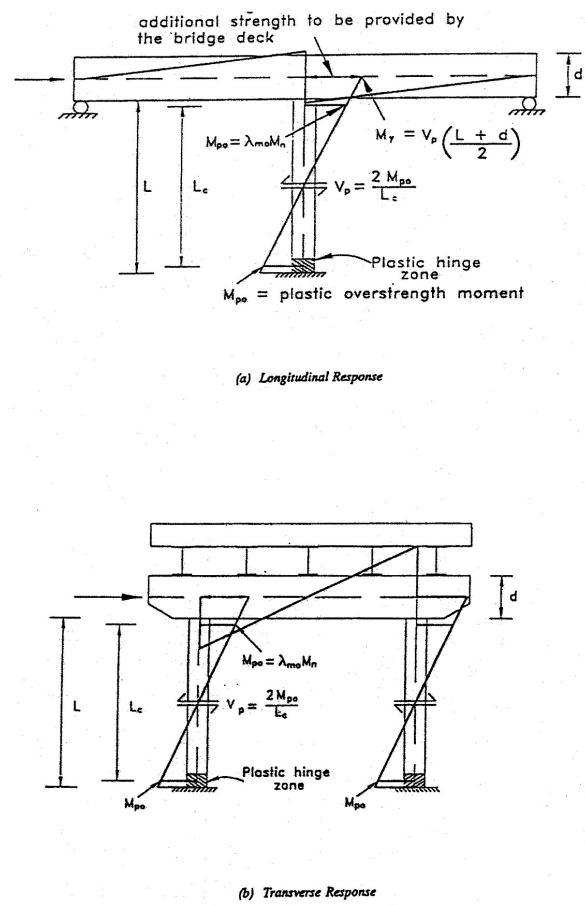


Figure 4.8.1-1 Capacity Design of Bridges Using Overstrength Concepts

4.8.1.1 Single Column and Piers

Column shear forces and design moments in the superstructure, bent caps, and the foundation structure shall be calculated 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 given in Article 4.8.1 times the nominal moment. The nominal moment for steel members is calculated using the expected yield strengths of Article 7.7.2. For both materials use the maximum elastic column axial load from Article 3.6 added to the column dead load. Column overstrength moments should be distributed to the connecting structural elements. (Exception: when calculating the design forces for the geotechnical aspects of foundations in SDR 3, 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 the superstructure or with isolation gaps less than required by Article 7.8.2 or 8.8.2, the shear shall be calculated as the greatest shear obtained from using:

  a. The overstrength moment at both the top of the flare and the top of the foundation with the appropriate column height.
  b. The overstrength moment at both the bottom of the flare and the top of the foundation with the reduced column height.

If the foundation of a column is more than 1-column diameter below ground level, the column height for the capacity shear force calculation shall be based on the mud or ground line, not the top of the foundation.

For pile bents or drilled shafts, the length of the pile or drilled shaft shall be not lower than the ground line for the purpose of calculating the capacity design shear force.

The forces corresponding to a single column hinging are:

- Axial Forces —unreduced maximum and minimum seismic axial load of Article 3.6 plus the dead load.
- Moments—those calculated in Step 1. Exception: An overstrength factor of 1.0 is required for geotechnical design forces in SDR 3.
- Shear Force—that calculated in Step 2.

4.8.1.2 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 4.8.1.1. In the plane of the bent the forces shall be calculated as follows:

- *Step 1.* Determine the column overstrength moments. Use overstrength factors given in Article 4.8.1 on the nominal strength calculated using the expected yield strength for structural steel. For both materials use the axial load corresponding to the dead load. Exception: When calculating the design forces for the geotechnical aspects of foundations in SDR 3 use an overstrength factor of 1.0 on the nominal moment.
- *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 4.8.1.1 - Step 2. For pile bents and drilled shafts, the length of pile from the pile cap to the mud or ground line shall be used to calculate the capacity design shear force.
- *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% 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, are:

- 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 corresponding to the maximum compressive axial load specified in (1) with an overstrength factor specified in Article 4.8.1 Exception: An overstrength factor of 1.0 is required for geotechnical design forces in SDR 3.
- Shear Force—the shear force corresponding to the final column overstrength moments in Step 4 above.

4.8.1.3 Capacity Design Forces

Design forces for columns and pile bents shall be determined using the provisions of Article 4.8.1.1 or 4.8.1.2 or the elastic design forces specified in Article 4.10. Capacity design forces for pier walls in the weak direction shall be determined using the provisions of Article 4.8.1.1 and those in the strong direction using Article 4.10. The capacity design forces for the shear design of individual columns, pile bents or drilled shafts shall be those determined using Article 4.8.1.1 or 4.8.1.2 as appropriate. The capacity design forces for the connection of the column to the foundation, cap beam or superstructure shall be the axial forces, moments and shears determined using the provisions of Article 4.8.1.1 or 4.8.1.2. The bearing supporting a superstructure shall be capable of transferring the shear forces determined using the provisions of Article 4.8.1.1 or 4.8.1.2 in both the longitudinal and transverse directions as per Article 7.9 or 8.9. Sacrificial elements are designed for the elastic forces from the 50% PE in 75 year earthquake. The capacity design forces for superstructure design (Article 8.11) shall either be the elastic forces from the analysis or where appropriate the moments and shears from Article 4.8.1.1 or 4.8.1.2. The abutment forces associated with the superstructure design shall be the elastic forces from the analysis.

4.9 PLASTIC HINGE ZONES

Columns, piers, pile bents/caissons and piles that participate in the ERS will have plastic hinges occurring and special detailing in these zones is specified in Articles 7.7, 7.8, 8.7, and 8.8 as appropriate. The plastic hinge zones defined below cover the potential range of locations where a plastic hinge may occur.

4.9.1 Top Zone of Columns, Pile Bents and Drilled Shafts

For concrete and steel columns, pile bents and drilled shafts the plastic hinge zone at the top of the member is defined as the length of the member below the soffit of the superstructure for monolithic construction and below the soffit of girders or cap beams for bents. The plastic hinge zone length shall be the maximum of the following.

- The maximum cross-sectional dimension of a reinforced concrete column
- One sixth of the clear height of a reinforced concrete column
- One eighth of the clear height of a steel column
- 450mm
- The following additional criteria shall determine the maximum plastic hinge length in reinforced concrete columns

$$D\left(\cot\theta + \tfrac{1}{2}\tan\theta\right) \qquad (4.9.1\text{-}1)$$

$$1.5\left(0.08\,M/V + 4400\varepsilon_y d_b\right) \qquad (4.9.1\text{-}2)$$

$$M/V\left(1 - M_y/M_{po}\right) \qquad (4.9.1\text{-}3)$$

where

D = transverse column dimension in direction of bending
θ = principal crack angle from Equation. 8.8.2.3-4
ε_y = yield strain of longitudinal reinforcement
d_b = longitudinal bar diameter
M = maximum column moment
V = maximum column shear
M_y = column yield moment
M_{po} = column plastic overstrength moment

For flared columns the plastic hinge zone shall extend from the top of the column to a distance equal to the maximum of the above criteria below the bottom of the flare.

4.9.2 Bottom Zone of a Column Above a Footing or Above an Oversized In-ground Drilled Shaft

The plastic hinge zone above the top of the footing of a column or a drilled shaft designed so that the maximum moment is above ground shall be the maximum of the items given in Article 4.9.1 unless the footing or the transition between in ground and above ground drilled shafts is below the ground level in which case it shall extend from the top of the footing or the transition between the two shafts to a distance above the mud or ground line equal to the maximum of the items given in Article 4.9.1.

4.9.3 Bottom Zone of Pile Bents and Drilled Shafts/Caissons

The plastic hinge zone at the bottom of a pile bent or a uniform diameter drilled shaft/caisson shall extend a distance above the mud or ground line equal to the maximum of the items specified in 4.9.1 to a distance 10D below the mud or ground line or 5 m (15 ft.) whichever is greater. It need not exceed 3D below the point of maximum moment in the ground. If scour or liquefaction may occur the plastic hinge zone as a minimum shall extend a distance of 3D below the mean scour depth or 3D below the lowest liquefiable layer. If a drilled shaft has an oversized in-ground shaft the top 10D of the oversized shaft shall be treated like the zone of a pile below the pile cap.

4.10 ELASTIC DESIGN OF SUBSTRUCTURES

There may be instances where a designer chooses to design all of the substructure supports elastically (i.e., R=1.0 for all substructures) or in some cases a limited number of substructure elements are designed elastically

4.10.1 All Substructure Supports are Designed Elastically

The elastic design forces for all elements are obtained from SDAP D using either an R=1.0 or 0.8 as specified in Table 4.7-2. The design force for any elements that could result in a brittle mode of failure (e.g., shear in concrete columns and pile bents, connections in braced frames) shall use an R-Factor of 0.67 with the elastic force. As an alternate to the use of the elastic forces, all elements connected to the column can be designed using the capacity design procedures of Article 4.8 using an overstrength ratio of 1.0 times the nominal moment capacities.

4.10.2 Selected Substructure Supports are Designed Elastically

If selected substructure supports are designed elastically then the moment demand can be established using an R=1.0 from the SDAP D analysis. The column or pile bent shear force and all connecting elements shall be designed using the capacity design procedures of Article 4.8 or the requirements of Article 4.10.1.

Exception: The component design procedures of Article 4.10.1 may be used, provided the SDAP D analytical model uses the secant modulus of columns that are not designed elastically. The secant stiffness (K_{sec}) of the columns shall be based on the elastic displacements from an iterated analysis as shown in Figure 4.10-1 where M_n is the nominal moment capacity and Δ_e is the elastic displacement of the bent as defined in Article 7.3.4 or 8.3.4

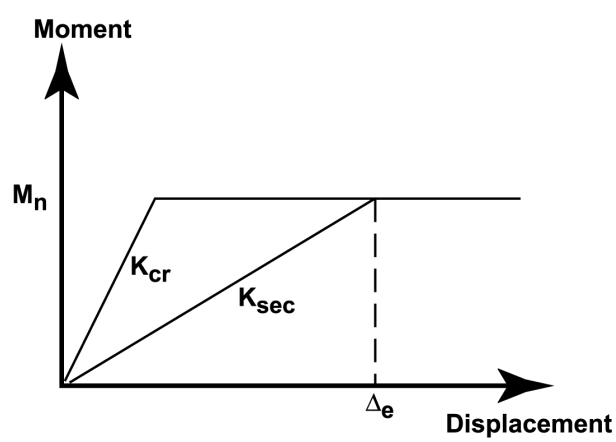


Figure 4.10-1 Characterization of the Secant stiffness of a column

Section 5
ANALYSIS REQUIREMENTS

5.1 DEFINITION OF PROCEDURES

5.1.1 General

When seismic analysis is required for Seismic Design and Analysis Procedure (SDAP) C, D, and E, the bridge shall be analyzed using a mathematical model that considers the geometry, boundary conditions and material behavior of the structure. The Engineer should consider the force and deformation effects being quantified and the accuracy required when defining a mathematical model.

A representation of the foundation and soil that supports the bridge may be included in the mathematical model of the foundations depending on the type of foundation, the Seismic Design and Analysis Procedure (SDAP), and the Seismic Design Requirement (SDR). When the foundations and abutments are included in the mathematical model, the assumed properties shall be consistent with the expected deformations of the soil.

In the case of seismic design, gross soil movement and liquefaction shall also be considered in the analysis when applicable.

5.1.2 Selection of Analysis Procedure

For seismic design the choice of the mathematical model and analysis procedure shall be based on the requirements of Article 3.7.

Table 3.7-2 identifies the Seismic Design and Analysis Procedure. When required, the Seismic Design and Analysis Procedures use the following seismic demand analysis and/or seismic displacement capacity verification procedures in order of increasingly higher-level of ability to represent structural behavior. A higher level analysis may be used in place of a lower-level analysis.

- *Capacity Spectrum Analysis* - Seismic response of a very regular structure is modeled as a single degree-of-freedom system, and the demand analysis and capacity evaluation are combined in a single procedure. The capacity spectrum analysis may be used for seismically isolated bridges.

- *Elastic Response Spectrum Analysis* - Seismic demands are determined by a response spectrum analysis using the spectrum defined in Article 3.4. For bridges with a regular configuration, the uniform load method may be used, otherwise a multi-mode dynamic analysis is required.

- *Nonlinear Static Displacement Capacity Verification ("Pushover" Analysis)* - The displacement capacity of individual piers or bents is determined by a lateral load-displacement analysis accounting for the nonlinear behavior of the inelastic components.

- *Nonlinear Dynamic Analysis* – Nonlinear dynamic analysis using earthquake ground motion records to evaluate the displacement and force demands accounting for the inelastic behavior of the components.

A nonlinear dynamic analysis is required for structures with seismic isolation systems with (1) an effective vibration period greater than 3 seconds, or (2) effective damping greater than 30 percent.

The displacements from any demand analysis must satisfy the requirements in Articles 7.3 or 8.3.

5.2 SEISMIC LATERAL LOAD DISTRIBUTION

5.2.1 Applicability

These provisions shall apply to decks, girders, diaphragms (cross-frames), lateral bracing, and connections between the superstructure and the substructure, which are part of the earthquake

resisting system in structures with Seismic Design Requirements (SDR) 3, 4, 5, and 6. These provisions do not apply in Seismic Design Requirements 1 and 2.

5.2.2 Design Criteria

The Engineer shall demonstrate that a clear, straightforward load path from the superstructure to the substructure exists and that all components and connections are capable of resisting the imposed load and displacement effects consistent with the chosen load path.

If the overstrength forces are chosen for use in the design of the superstructure, then the elastic force distribution in the superstructure obtained from an elastic response spectrum analysis is not appropriate for use in the superstructure design. Unless a more refined analysis is made when using the overstrength forces in the superstructure design, the inertial forces expected to act on the superstructure may be assumed to vary linearly along the superstructure, and they shall produce both translational and rotational equilibrium when combined with the plastic mechanism forces from the substructure.

The flow of forces in the assumed load path must be accommodated through all affected components and details including, but not limited to, flanges and webs of main beams or girders, cross-frames, connections, slab-to-girder interfaces, and all components of the bearing assembly from top flange interface through the confinement of anchor bolts or similar devices in the substructure.

The analysis and design of end diaphragms and cross-frames shall consider horizontal supports at an appropriate number of bearings. Slenderness and connection requirements of bracing members that are part of the lateral force resisting system shall comply with applicable provisions specified for main member design.

Members of diaphragms and cross-frames identified by the Designer as part of the load path carrying seismic forces from the superstructure to the bearings shall be designed and detailed to remain elastic, based on the applicable gross area criteria, under all design earthquakes, regardless of the type of bearings used. The applicable provisions for the design of main members shall apply.

However, if elements of the earthquake resisting system are explicitly intended and designed to respond inelastically, then the previous paragraph does not apply to such elements. All other elements of the earthquake resisting system shall either be capacity-protected or designed for the elastic forces.

If elements of the earthquake resisting system are designed to fuse (i.e. breakaway) in the larger earthquake, then the redistribution of force that occurs with such alteration of the seismic load path shall be accounted for in the analysis.

All load-resisting elements shall have sufficient deformation capacity at the displacement of the center of mass of structure as determined from the seismic analysis.

5.2.3 Load Distribution

A viable load path shall be established to transmit seismic loads to the substructure based on the stiffness characteristics of the deck, girders, diaphragms – end, intermediate and pier – (often referred to as cross-frames in steel bridges), lateral bracing, and connections between the superstructure and substructure. Unless a more refined analysis is made, an approximate load path shall be assumed as noted below.

In bridges with:

- A concrete deck that can provide horizontal diaphragm action, or
- A horizontal bracing system in the plane of the deck,

the lateral loads applied to the deck shall be assumed to be transmitted directly to the bearings through end diaphragms and/or pier diaphragms. The development and analysis of the load path through the deck or through the lateral bracing, if present, shall utilize assumed structural actions analogous to those used for the analysis of wind loading.

In bridges that have:

- Decks that cannot provide horizontal diaphragm action and
- No lateral bracing in the plane of the deck,

the lateral loads applied to the deck shall be distributed through the intermediate diaphragms to the bottom lateral bracing or the bottom flange, and then to the bearings, and through the end diaphragms and pier diaphragms in proportion to their relative rigidity and the respective tributary mass of the deck.

If a lateral bracing system is not present, and the bottom flange is not adequate to carry the imposed force effects, the first procedure shall be used, and the deck shall be designed and detailed to provide the necessary horizontal diaphragm action.

5.3 MODELING REQUIREMENTS

5.3.1 General

For the dynamic analysis of structures subjected to earthquakes, the geometric configuration, strength, stiffness, mass, and energy dissipation mechanisms of the structural components and footings shall be included in the mathematical model.

Bridges with multiple frames may be analyzed using models of a partial number of frames. Each model shall represent the geometry, mass, stiffness, and boundary conditions for the frames included in the model.

The seismic analysis shall consider at least the two horizontal ground motion components.

The combination of loads from different horizontal and vertical components is given in Article 3.6.

The effect of the vertical component ground motion on bridges within 10 km of an active fault shall be included according to the requirements in Article 3.4.5.

5.3.2 Distribution of Mass

The modeling of mass shall be made with consideration of the degree of discretization in the model and the anticipated motion due to seismic excitation.

The number of degrees-of-freedom shall be selected to represent the total mass and mass distribution of the structure.

5.3.3 Stiffness and Strength

5.3.3.1 General

The mathematical model shall represent the stiffness of individual structural elements considering the materials, section dimensions, and force transfer between elements. For ductile earthquake resisting elements the stiffness shall be representative of the stiffness near the yield deformation (e.g., cracked section properties shall be used for reinforced concrete elements). For capacity protected elements, including the superstructure, the elastic stiffness shall be represented in the mathematical model.

For Displacement Capacity Verification (nonlinear static analysis), the mathematical model shall include the strength based on nominal material properties. For nonlinear dynamic analysis, the models shall represent the stiffness, strength, and hysteretic behavior of the inelastic seismic resisting elements under cyclic loads.

5.3.3.2 Substructure

The flexural stiffness of columns and pier walls shall consider the effect of axial load. For reinforced concrete columns and pier walls, the stiffness shall represent the effects of cracking. When required by Article 4.10.2, the secant stiffness of columns responding inelastically shall be used.

For Displacement Capacity Verification ("pushover" or inelastic static analysis), the strength of structural steel components in the model shall be based on the nominal plastic capacity. The flexural strength of reinforced and prestressed elements shall be based on nominal material properties of the steel and concrete.

The stiffness of capacity protected elements shall be based on elastic properties, including the effects of concrete cracking.

5.3.3.3 Superstructure

The stiffness of the superstructure shall be consistent with the load path identified in accordance with Article 5.2.3, including composite behavior between girders and decks and effective width of the superstructure that are monolithic with piers.

5.3.4 Foundations

Foundations may be modeled using the Foundation Modeling Method (FMM) defined in Table 5.3.4-1. Articles 8.4.2 and 8.4.3 provide the requirements for estimating foundation springs and the depth to fixity.

The required foundation modeling method depends on the Seismic Design Requirement (SDR) and the Seismic Design and Analysis Procedure (SDAP).

For SDR 3, Foundation Modeling Method I (FMM I) is required for any SDAP.

For SDR 4, 5, and 6, Foundation Modeling Method I may be used for SDAP C. SDAP D and E require the use of Foundation Modeling Method II (FMM II).

For SDAP E, FMM II is required in the Displacement Capacity Verification ("pushover") analysis if it is used in the multi-mode dynamic analysis for displacement demand. The foundation models in the multi-mode dynamic analysis and Displacement Capacity Verification shall be consistent and representative of the footing behavior.

Table 5.3.4-1 Definition of Foundation Modeling Method

Foundation Type	FMM I	FMM II
Spread Footing	Rigid	Rigid for Soil Types 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.

For sites identified as susceptible to liquefaction or lateral spread, the model of the foundations and structures shall consider the nonliquefied and liquefied conditions using the procedures specified in Articles 7.6 and 8.6.

5.3.5 Abutments

The model of the abutment shall reflect the expected behavior of the abutment under seismic loads in each orthogonal direction. Resistance of structural components shall be represented by cracked section properties for multi-mode response spectrum analysis. The resistance from passive pressure shall be represented by a value for the secant stiffness consistent with the maximum displacement – Articles 7.5 and 8.5. For the Displacement Capacity Verification, the strength of each component in the abutment, including soil, shall be included.

5.3.6 Seismic Isolator Units

Seismic isolator units shall be modeled by an effective stiffness based on the properties of the isolator unit.

To simplify the nonlinear behavior of the isolator unit, a bi-linear simplification may be used. The analysis shall be repeated using upper-bound properties in one analysis and lower-bound properties in another if required by Article 15.4. The purpose of the upper- and lower-bound analyses is to determine the maximum forces in the substructure and maximum displacement of the isolation system.

The upper- and lower-bound analyses are not required if the displacements, using equation (5.4.1.1-1), do not vary from the design values by more than 15 percent when the maximum and minimum values of the isolator unit properties are used (Article 15.4). For these simplified calculations, damping ratios greater than 30 percent may be used to establish the 15 percent limit.

5.3.7 Bearing and Joints

Two models shall represent expansion bearings and intermediate joints. The compression model assumes the superstructure at the bearing or joint is closed and can transfer longitudinal forces. The tension model assumes the bearing or joint is open and cannot transfer longitudinal forces. The

stiffness of restraining devices, if any, shall be included in the tension model.

A compression model need not be considered for expansion bearings if it can be demonstrated by calculation that longitudinal forces cannot be transferred through the superstructures at the bearing location.

5.3.8 Damping

Energy dissipation in the structure, including, footings and abutments, may be represented by viscous damping. The selection of the viscous damping ratio depends on the type of dynamic analysis and the configuration of the bridge.

For elastic response spectrum analysis, the viscous damping ratio inherent in the specified ground spectra is 5% damping and this is specified for all structural systems except those with isolation systems. For the Capacity Spectrum method, damping is inherent in the B-Factor.

5.4 ANALYSIS PROCEDURES

The regularity requirements that permit use of the Capacity Spectrum Analysis Method are given in Article 4.4.2. The regularity requirements for using the Uniform Load Method and Multi-mode Methods of Analyses are given in Article 5.4.2.1.

5.4.1 Capacity Spectrum Analysis

The lateral strength of each pier in the longitudinal and transverse directions shall be at least C_c times the tributary weight for the pier.

The lesser of the following equations shall be used to assess C_c for the 50% PE in 75 year and 3% PE in 75-year/1.5 mean-deterministic earthquake loadings:

$$C_c\Delta = \left(\frac{F_v S_1}{2\pi B_L}\right)^2 g \qquad (5.4.1\text{-}1)$$

$$C_c = \frac{F_a S_s}{B_s} \qquad (5.4.1\text{-}2)$$

where B_s and B_L are response reduction factors for short and long period structures, respectively, and are defined in Table 5.4.1-1. The response spectrum values and soil factors, $F_v S_1$ and $F_a S_s$, are defined in Article 3.4. In Equation 5.4.1-1, Δ is the displacement of the pier.

When equation 5.4.1-1 governs for the 3% PE in 75-year/1.5 mean deterministic earthquake, the displacement of the superstructure, Δ, shall satisfy the requirements of Articles 7.3.4 or 8.3.4. When equation 5.4.1-1 governs for the 50% PE in 75-year earthquake, Δ shall be taken as 1.3 times the yield displacement of the pier.

5.4.1.1 Seismic Isolation Systems

The capacity spectrum analysis procedure may be used for structures with seismic isolation systems that meet the regularity requirements for the Uniform Load Method of Article 5.4.2.1 and the effective vibration period is 3 seconds or less, and the effective damping is less than or equal to 30 percent of critical. Article 15.4 specifies other analysis procedures for seismically isolated structures.

The displacement, Δ, (meters) of the superstructure (including the substructure and bearing unit deformation) is given by

$$\Delta = \frac{0.25 F_v S_1 T_{eff}}{B} \text{ (meters)} \qquad (5.4.1.1\text{-}1)$$

$$\Delta = \frac{10 F_v S_1 T_{eff}}{B} \text{ (inches)} \qquad (5.4.1.1\text{-}1(b))$$

where

$$T_{eff} = 2\pi\sqrt{\frac{W}{K_{eff}\, g}} \qquad (5.4.1.1\text{-}2)$$

The damping coefficient, B, is based on the percentage of critical damping according to Table 5.4.1.1-1. The percentage of critical damping depends on the energy dissipation by the isolation system, which shall be determined by test of the isolation systems characteristics, as specified in Article 15.10. The damping coefficient may be determined by linear interpolation of the values in Table 5.4.1.1-1.

Table 5.4.1-1 Capacity Spectrum Response Reduction Factors for Bridges with Ductile Piers

(a) 50% in 75 Year Earthquake Loading

Performance Level	B_S	B_L
Operational	1	1
Life Safety	1	1

(b) 3% in 75 Year Earthquake Loading

Performance Level	B_S	B_L
Operational	1	1
Life Safety	2.3	1.6

Table 5.4.1.1-1 Capacity Spectrum Response Reduction Factors for Bridges with Seismic Isolation Systems

	Damping (as percentage of critical)						
	≤ 2	5	10	20	30	40	50
B	0.8	1.0	1.2	1.5	1.7	1.9	2.0

5.4.2 Elastic Response Spectrum Analysis

5.4.2.1 Selection of Analysis Method

The uniform load method may be used for structures satisfying the requirements in Table 5.4.2.1-1. For structures not satisfying the regularity requirements of Table 5.4.2.1-1, the multi-mode dynamic analysis shall be used.

Table 5.4.2.1-1 Requirements for Uniform Load Method

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	20°	20°	30°	30°	30°
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

5.4.2.2 Uniform Load Method

The uniform load method shall be based on the fundamental mode of vibration in 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 seismic coefficient demand, C_d, specified in Article 3.4 by the response spectra at the appropriate period shall be used to calculate the equivalent uniform seismic load from which seismic force effects are found. However, for periods less than T_s, the seismic coefficient demand shall be equal to S_{DS}.

5.4.2.3 Multi-Mode Dynamic Analysis Method

The elastic multi-mode dynamic analysis method shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. As a minimum, linear dynamic analysis using a three-dimensional model shall be used to represent the structure.

The number of modes included in the analysis shall be at least three times the number of spans in the model for regular bridges, and the total modal mass shall be at least 90%.

The elastic seismic response spectrum as specified in Article 3.4 shall be used for each mode with its inherent 5% damping. The spectrum at the fundamental vibration periods shall be scaled for damping ratios other than 5 percent for an isolated structure. For structures with seismic isolation the scaling shall apply only for periods greater than $0.8T_{eff}$ where T_{eff} is defined in Article 15.4.1. The 5 percent response spectrum shall be used for other modes.

The member forces and displacements due to a single component of ground motion may be estimated by combining the respective response quantities (moment, force, displacement, or relative displacement) from the individual modes by the Complete Quadratic Combination (CQC) method. Combination of forces from orthogonal components of ground motion are specified in Article 3.6.

5.4.3 Seismic Displacement Capacity Verification

The displacement capacity verification analysis shall be applied to individual piers or bents to determine the lateral load-displacement behavior of the pier or bent. The capacity evaluation shall be performed for individual piers or bents in the longitudinal and transverse direction separately.

The capacity evaluation shall identify the component in the pier or bent that first reaches its inelastic deformation capacity as given in Articles 7.7.9, 7.8.6, 8.7.9 or 8.8.6. The displacement at which the first component reaches its maximum permitted deformation capacity defines the maximum displacement capacity, $\Delta_{capacity}$ for the pier or bent and this shall exceed the demand given in Articles 7.3.5 or 8.3.5. The model shall represent all components providing seismic load resistance.

When required by Article 5.3.4, the model for the foundation shall include soil springs or an estimated depth to fixity.

The model for the displacement capacity verification is based on nominal capacities of the inelastic components. Stiffness and strength degradation of inelastic components and effects of loads acting through the lateral displacement shall be considered.

Maximum plastic hinge rotations for structural components are specified in Articles 7.7.9, 7.8.6, 8.7.9 or 8.8.6. The maximum deformation for foundation and abutments are limited by geometric constraints on the structure and given in Article C3.2.

The model of the foundation for the displacement capacity evaluation shall be consistent with the demand analysis, Article 5.3.4.

For the purpose of this Article, the displacement is the displacement at the center of mass for superstructure supported by the pier or bent under consideration.

5.4.4 Nonlinear Dynamic Analysis Procedure

Nonlinear dynamic analysis provides displacements and member actions (forces and deformations) as a function of time for a specified earthquake ground motion. All loads in Extreme Load Case I shall be included in the analysis.

The ground motion time histories shall satisfy the requirements of Article 3.4.4.

A minimum of three ground motions, representing the design event, shall be used in the analysis. Each ground motion shall include two horizontal components and a vertical component. The maximum action for the three ground motions shall be used for design. If more than seven ground motions are used, the design action may be the mean of the actions for the individual ground motions.

Section 6
SEISMIC DESIGN REQUIREMENTS (SDR) 1 AND 2

6.1 GENERAL

Bridges classified as either SDR 1 or 2 in accordance with Table 3.7-2 of Article 3.7 shall conform to all of the requirements of this section. For SDR 2 reinforced concrete columns, pile bents and the top 3 diameters of concrete piles the shear reinforcement shall meet the requirements of Article 6.8.2.

6.2 DESIGN FORCES

For bridges in SDR 1 the horizontal design connection force in the restrained directions shall not be taken less than 0.1 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

For SDR 2, the horizontal design connection force in the restrained directions shall not be taken to be less than 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

If a bridge has all elastomeric bearings or sliders that permit horizontal movement in both the transverse and longitudinal directions, these minimums apply to all bearing connections or the forces shall be determined using Article 6.10.

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.

6.3 DESIGN DISPLACEMENTS

The seat width shall not be less than:

$$N = \left[0.10 + 0.0017L + 0.007H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(2\frac{B}{L}\right)^2} \right] \frac{(1 + 1.25F_v S_1)}{\cos\alpha} \quad (6.3\text{-}1)$$

where,

L is the distance between joints in meters
H is the tallest pier between the joints in meters
B is the width of the superstructure in meters
α is the skew angle

The ratio B/L need not be taken greater than 3/8.

6.4 FOUNDATION DESIGN REQUIREMENTS

Specific design requirements for seismic loads are not required.

6.5 ABUTMENT DESIGN REQUIREMENTS

Specific design requirements for seismic loads are not required.

6.6 LIQUEFACTION DESIGN REQUIREMENTS

Specific design requirements for seismic loads are not required.

6.7 STRUCTURAL STEEL DESIGN REQUIREMENTS

6.7.1 SDR 1

Specific design requirements for seismic loads are not required.

6.7.2 SDR 2

6.7.2.1 Ductile Moment-Resisting Frames and Bents

Ductile moment-resisting frames and bents shall meet the requirements of Article 8.7.4, except as modified in accordance with this article.

6.7.2.1.1 Columns

Columns shall be designed as Ductile Substructure Elements as per Article 8.7.6.1.

The maximum axial compressive load limit of Article 8.7.4.1 shall be replaced by $0.40A_gF_y$.

6.7.2.1.2 Beams, Panel Zones and Connections

Beams, panel zones, moment resisting connections, and column base connections shall be designed as Capacity Protected Elements as defined in Article 8.2.2

The nominal flexural resistance of the column shall be determined from Article 8.7.4.2.

6.7.2.2 Ductile Concentrically Braced Frames

Ductile concentrically braced frames and bents shall meet the requirements of Article 8.7.5.

6.7.2.3 Concentrically Braced Frames and Bents with Nominal Ductility

Concentrically braced frames and bents with nominal ductility shall meet the requirements of Article 8.7.6 except braces in chevron braced frames need not conform to Article 8.7.6.2, but shall meet the requirements of Article 8.7.6.5.

6.8 REINFORCED CONCRETE DESIGN REQUIREMENTS

6.8.1 SDR 1

Specific design requirements for seismic loads are not required.

6.8.2 SDR 2

6.8.2.1 Minimum Shear Steel

For columns, and pile bents or drilled shafts with in-ground hinging, transverse reinforcement shall be provided as specified by the "Implicit Method" for shear in Article 7.8.2.3 with θ = d = 35˚ and Λ = 1.

6.8.2.2 Pile Reinforcement Requirements

For piles the top three-diameters (3D) shall be provided with transverse reinforcement required by the "Implicit Method" in Article 7.8.2.3. The angles shall be set at $\theta = \alpha = 35^\circ$ and $\Lambda = 1$.

6.9 BEARING DESIGN REQUIREMENTS

Bearings and their connections shall be designed for the design forces specified in Article 6.2.

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.

6.10 SEISMIC ISOLATION DESIGN REQUIREMENTS

To reduce the bearing design force requirements of 6.2, bearings may be designed as isolations bearings provided the design displacements and forces of Article 15.7 are used and the quality control tests of Articles 15.12, 15.14, or 15.15 are satisfied.

Section 7
SEISMIC DESIGN REQUIREMENTS (SDR) 3

7.1 GENERAL

Bridges classified as SDR 3 in accordance with Table 3.7-2 of Article 3.7 shall conform to all of the requirements of this section.

7.2 DESIGN FORCES

7.2.1 Ductile Substructures (R>1) – Flexural Capacity

7.2.1.1 SDAP B

No seismic column design forces are specified. The seismic design procedure begins with a column that satisfies all the non-seismic load conditions and meets the minimum longitudinal reinforcement ratio of Article 7.8.2.1 for concrete columns and Article 7.8.3 for Wall Type Piers and the minimum width to thickness ratios of Article 7.7.4 for steel columns.

7.2.1.2 SDAP C

The sum of the capacities of all columns must satisfy Article 5.4.1.

7.2.1.3 SDAP D and E

Column design forces are the maximum of those obtained from an elastic analysis and reduced using the appropriate R-Factor as specified in Steps 2, 3 and 4 of Article 4.5 and combined in accordance with Article 3.6.

7.2.2 Capacity Protected Elements or Actions

The design provisions of Article 4.8 apply to capacity protected elements and actions.

Capacity design principles require that those elements not participating as part of the primary energy dissipating system (flexural hinging in columns), such as column shear, joints and cap beams, spread footings, pile caps and foundations be "capacity protected." This is achieved by ensuring the maximum-overstrength moment and shear from plastic hinges in the columns can be dependably resisted by adjoining elements.

Exception: Elastic design of all substructure elements (Article 4.10), seismic isolation design (Article 7.10) and in the transverse direction of a column when a ductile diaphragm is used (Article 7.7.8.2).

7.2.3 Elastically Designed Elements

There may be instances where a designer chooses to design all of the substructure supports elastically (i.e., R=1.0 for all substructures) or in some cases a limited number of substructure elements are designed elastically. If so, the provisions of Article 4.10 apply.

7.2.4 Abutments and Connections

The seismic design forces for abutments are obtained by SDAP D or E when required and given in Article 7.5. The seismic design forces for connections are the lower of those obtained from Article 7.2.2 or the elastic forces divided by the appropriate R-Factor from Table 4.7-2.

7.2.5 Single Span Bridges

For single-span bridges, regardless of seismic zone and in lieu of a rigorous analysis, the minimum design force at the connections in the restrained direction between the superstructure and the substructure shall not be less than the product of $F_a S_S / 2.5$, and the tributary permanent load.

7.3 DESIGN DISPLACEMENTS

7.3.1 General

For this section, displacement is the displacement at the center of mass for a pier or bent in the transverse or longitudinal direction determined from the seismic analysis except in Article 7.3.2 where the displacement occurs at the bearing seat.

7.3.2 Minimum Seat Width Requirement

The seat width shall not be less than 1.5 times the displacement of the superstructure at the seat according to Equation (7.3.4-2) or:

$$N = \left[0.10 + 0.0017L + 0.007H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(2\frac{B}{L}\right)^2} \right] \frac{(1 + 1.25F_v S_1)}{\cos\alpha} \qquad (7.3.2\text{-}1)$$

where,

L = the distance between joints in meters
H = the tallest pier between the joints in meters
B = the width of the superstructure in meters
α = the skew angle

The ratio B/L need not be taken greater than 3/8.

7.3.3 Displacement Compatibility

All components that are not designed to resist seismic loads must have deformation capacity sufficient to transfer non-seismic loads.

7.3.4 P-Δ Requirements

The displacement of a pier or bent in the longitudinal and transverse direction must satisfy

$$\Delta \le 0.25 C_c H \qquad (7.3.4\text{-}1)$$

where,

$$\Delta = R_d \Delta_e \qquad (7.3.4\text{-}2)$$

$$R_d = \left(1 - \frac{1}{R}\right)\frac{1.25T_s}{T} + \frac{1}{R} \text{ for } T < 1.25T_s \qquad (7.3.4\text{-}3)$$

where T_s is defined in Figure 3.4.1-1, otherwise $R_d = 1$.

Δ_e is the displacement demand from the seismic analysis, R is the ratio between elastic lateral force and the lateral strength of the pier or bent, C_c is the seismic coefficient based on the lateral strength of the pier or bent ($C_c = V/W$ where V is the lateral strength), and H is the height of the pier from the point of fixity for the foundation.

If a nonlinear time history seismic analysis is performed, the displacement demand, Δ, may be obtained directly from the analysis in lieu of Equation 7.3.4-2. However, the displacement Δ shall not be taken less than 0.67 of the displacement determined from an elastic response spectrum analysis.

7.3.5 Minimum Displacement Requirements for Lateral Load Resisting Piers and Bents

For SDAP E the maximum permitted displacement capacity from the Displacement Capacity Verification must be greater than the displacement demand according to the following requirement:

$$1.5\Delta \le \Delta_{capacity} \qquad (7.3.5\text{-}1)$$

where the Δ is defined in Article 7.3.4 and $\Delta_{capacity}$ is the maximum displacement capacity per Article 5.4.3.

When a nonlinear dynamic analysis is performed the displacement demand may not be taken less than 0.67 times the demand from a elastic response spectrum analysis, nor may the displacement capacity be taken greater than the capacity from the Displacement Capacity Verification.

7.4 FOUNDATION DESIGN REQUIREMENTS

7.4.1 Foundation Investigation

7.4.1.1 General

A subsurface investigation, including borings and laboratory soil tests, shall be conducted in accordance with the provisions of Appendix B to provide pertinent and sufficient information for the determination of the Site Class of Article 3.4.2.1. The type and cost of foundations should be considered in the economic, environmental, and aesthetic studies for location and bridge type selection.

7.4.1.2 Subsurface Investigation

Subsurface explorations shall be made at pier and abutment locations, sufficient in number and depth, to establish a reliable longitudinal and transverse substrata profile. Samples of material encountered shall be taken and preserved for future reference and/or testing. Boring logs shall be prepared in detail sufficient to locate material strata, results of penetration tests, groundwater, any artesian action, and where samples were taken. Special attention shall be paid to the detection of narrow, soft seams that may be located at stratum boundaries.

7.4.1.3 Laboratory Testing

Laboratory tests shall be performed to determine the strength, deformation, and flow characteristics of soils and/or rocks and their suitability for the foundation selected. In areas of higher seismicity (e.g., SDR 3, 4, 5, and 6), 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, if unusual soils exist or if the foundation is supporting a critical bridge.

7.4.2 Spread Footings

Spread footing foundations for SDR 3 shall be designed using column loads developed by capacity design principles or elastic seismic loads, in accordance with Strength Limit State requirements given in Article 10.6.3 of the AASHTO LRFD provisions. It will not normally be necessary to define spring constants for displacement evaluations or moment-rotation and horizontal force-displacement behavior of the footing-soil system (Article 5.3.4). Checks shall also be made to confirm that flow slides and loss of bearing support from liquefaction do not occur (Article 7.6).

7.4.2.1 Moment and Shear Capacity

The overturning capacity of the spread footings shall be evaluated using 1.0 times the nominal moment capacity of the column (Article 4.8) or the elastic seismic design force within the column, whichever is less. Procedures for Strength Limit State Design given in Article 10.6.3 of the AASHTO LRFD provisions shall be used when performing this evaluation.

A triangular elastic stress distribution within the soil shall be used. The peak bearing soil pressure for the triangular distribution shall not exceed the ultimate bearing capacity of the soil at the toe of the footing. The width of maximum liftoff shall be no greater than 1/2 of the footing width for moment loading in each of the two directions treated separately.

If a non-triangular stress distribution occurs or if the liftoff is greater 1/2 of the footing, either the footing shall be re-sized to meet the above criteria or special studies shall be conducted to demonstrate that non-triangular stress pressure distribution or larger amounts of liftoff will not result in excessive permanent settlement during seismic loading. The special studies shall include push-over analyses with nonlinear foundation springs for SDAP E conditions.

No shear capacity evaluation of the footing will normally be required for SDR 3**.**

7.4.2.2 Liquefaction Check

An evaluation of the potential for liquefaction within near-surface soil shall be made in accordance with requirements given in Article 7.6 and Appendix D of these Specifications. If liquefaction is predicted to occur for the design earthquake, the following additional requirements shall be satisfied:

<u>Liquefaction without Lateral Flow or Spreading</u>

For sites that liquefy but do not undergo lateral flow or spreading, the bottom of the spread footing shall be located either below the liquefiable layer or at least twice the minimum width above the liquefiable layer. If liquefaction occurs below the footing, settlements resulting from the dissipation of excess porewater pressures shall be established in accordance with procedures given in Appendix D.

If the depth of the liquefiable layer is less than twice the minimum foundation width, spread footing foundations shall not be used, unless

- ground improvement is performed to mitigate the occurrence of liquefaction, or
- special studies are conducted to demonstrate that the occurrence of liquefaction will not be detrimental to the performance of the bridge support system.

Before initiating any evaluations of ground improvement alternatives or before conducting special studies, the potential applicability of deep foundations as an alternative to spread footings shall be discussed with the Owner.

Liquefaction with Lateral Flow or Spreading

If lateral flow or lateral spreading is predicted to occur, the amount of displacement associated with lateral flow or lateral spreading shall be established in accordance with procedures given in Appendix D. Once the deformation has been quantified, the following design approach shall be used.

- Determine whether the spread footings can be designed to resist the forces generated by the lateral spreading without unusual size or design requirements.
- If the footing cannot resist forces from lateral spreading or flow, assess whether the structure is able to tolerate the anticipated movements and meet the geometric and structural constraints of Table C3.2-1. The maximum plastic rotation shall be as defined in Article 7.7.9 and 7.8.6.
- If the structure cannot meet the performance requirements of Table 3.2-1, assess the costs and benefits of various mitigation measures to minimize the movements to a level that will meet the desired performance objective. If a higher performance is desired so that the spread footings will not have to be replaced, the allowable plastic rotations for concrete columns given in Article 7.7.9 and 7.8.6 shall be met.

The Owner shall be apprised of and concur with the approach used for the design of spread footing foundations for lateral flow or lateral spreading conditions.

7.4.3 Driven Piles

7.4.3.1 General

Resistance factors for pile capacities shall be as specified in Table 10.5.4-2 of the AASHTO LRFD provisions, with the exception that resistance factors of 1.0 shall be used for seismic loads.

For the Effect of Settling Ground and Downdrag Loads, unfactored load and resistance factors (γ = 1.0; ϕ = 1.0) shall be used, unless required otherwise by the Owner.

Batter piles shall not be used where downdrag loads are expected unless special studies are performed.

For seismic loading the groundwater table location shall be the average groundwater location, unless the Owner approves otherwise.

7.4.3.2 Design Requirements

Driven pile foundations subject to SDR 3 shall be designed for column moments and shears developed in accordance with the principles of capacity design (Article 4.8) or the elastic design forces, whichever is smaller. The Strength Limit State requirements given in Article 10.7.3 of the AASHTO LRFD provisions shall apply for design.

With the exception of pile bents, it will not normally be necessary to define spring constants for displacement evaluations or moment-rotation and horizontal force-displacement analyses for SDR 3 (Article 5.3.4). For pile bents, the estimated depth of fixity shall be used in evaluating response.

If liquefaction is predicted at the site, the potential effects of liquefaction on the capacity of the driven pile foundation system shall be evaluated in accordance with procedures given in Article 7.4.3.4.

7.4.3.3 Moment and Shear Design

The capacity of the geotechnical elements of driven pile foundations shall be designed using 1.0 times the nominal moment capacity of the column or the elastic design force within the column (Article 4.8), whichever is smaller. Unfactored resistance (ϕ = 1.0) shall be used in performing the geotechnical capacity check. The loads on the

leading pile row during overturning shall not exceed the plunging capacity of the piles. Separation between the pile tip and the soil (i.e. gapping) shall be allowed only in the most distant row of piles in the direction of loading. Forces on all other rows of piles shall either be compressive or not exceed the nominal tension capacity of the piles.

If the plunging capacity of the leading pile is exceeded or if uplift of other than the trailing rows of piles occurs, special studies shall be conducted to show that performance of the pile system is acceptable. These studies shall be performed only with the prior consent of the Owner and SDAP E is required.

Structural elements of pile foundations shall be designed using the overstrength moment capacity of the column or the elastic design force within the column (Article 4.8), whichever is smaller.

The maximum shear force on the pile(s) shall be less than the structural shear capacity of the piles.

7.4.3.4 Liquefaction Check

An evaluation of the potential for liquefaction shall be made in accordance with requirements given in Article 7.6 and Appendix D of these Specifications. If liquefaction is predicted to occur for the design earthquake, the following additional requirements shall be satisfied:

Liquefaction without Lateral Flow or Spreading

- The pile shall penetrate beyond the bottom of the liquefied layer by at least 3 pile diameters or to a depth that axial and lateral pile capacity are not affected by liquefaction of the overlying layer, whichever is deeper.
- The shear reinforcement in a concrete or pre-stressed concrete pile shall meet the requirements of Sec 7.8.2.3 from the pile or bent cap to a depth of 3 diameters below the lowest liquefiable layer.
- Effects of downdrag on the pile settlements shall be determined in accordance with procedures given in Appendix D.

Liquefaction with Lateral Flow or Lateral Spreading

- Design the piles to resist the forces generated by the lateral spreading.
- If the forces cannot be resisted, assess whether the structure is able to tolerate the anticipated movements and meet the geometric and structural constraints of Table C3.2-1. The maximum plastic rotation of the piles shall be as defined in Article 7.7.9 and Article 7.8.6.
- If the structure cannot meet the performance requirements of Table 3.2-1, assess the costs and benefits of various mitigation measures to reduce the movements to a tolerable level to meet the desired performance objective. If a higher performance is desired so that the piles will not have to be replaced, the allowable plastic rotations of Articles 7.7.9.2 and 7.8.6.2 shall be met.

7.4.4 Drilled Shafts

Procedures identified in Article 7.4.3.2, including those for liquefaction and dynamic settlement, shall be applied with the exception that the ultimate capacity in compression or uplift loading for single shaft foundations in SDR 3 shall not be exceeded during maximum seismic loading without special design studies and the Owner's approval. The flexibility of the drilled shaft shall also be represented in the design using either the estimated depth of fixity or soil springs in a lateral pile analysis.

Diameter adjustments shall be considered during lateral load analyses of shafts with a diameter greater than 600 mm if the shaft is free to rotate, as in the case of a column extension (i.e., no pile cap). Contributions from base shear shall also be considered.

7.5 ABUTMENT DESIGN REQUIREMENTS

7.5.1 General

The effect of earthquakes shall be investigated using the extreme event limit state of Table 3.2-1 with resistance factors $\varphi = 1.0$. Requirements for static design should first be met, as detailed in Articles 11.6.1 through 11.6.4 of the AASHTO LRFD provisions. Selection of abutment types prior to static design shall recognize type selection criteria for seismic conditions, as described in Articles 3.3, 3.3.1, and Section 4, Table 3.3.1-1 and Figure C3.3.1-4.

7.5.1.1 Abutments and 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 shall be considered in the seismic design of bridges, as outlined in these provisions. Damage to walls that is allowed to occur during earthquakes shall be consistent with the performance criteria. Abutment participation in the overall dynamic response of the bridge systems 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 (i.e., 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 based on an applicable passive earth-pressure theory.

7.5.2 Longitudinal Direction

Under earthquake loading, the earth pressure action on abutment walls changes from a static condition to one of generally two possible conditions, depending on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge/abutment configuration. For seat-type abutments where 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 would 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 passive pressure condition behind the backwall.

For stub or integral abutments, the abutment stiffness and capacity under passive pressure loading, are primary design concerns, as discussed in Articles 7.5.2.1 and 7.5.2.2. However, for partial depth or full depth seat abutment walls, earthquake-induced active earth pressures will continue to act below the backwall following separation of a knock-off backwall. These active pressures need to be considered in evaluating wall stability.

7.5.2.1 SDAP B and C

Abutments designed for service load conditions in these categories should resist earthquake loads with minimal damage with the exception of bridges in Seismic Hazard Level IV using SDAP C. For seat-type abutments, minimal abutment movement could be expected under dynamic active pressure conditions. However, bridge superstructure displacement demands could be 100 mm or more and potentially impact the abutment backwall. Where expected displacement demands are greater than a normal expansion gap of 25 to 50 mm, a knock-off backwall detail is recommended to minimize foundation damage, or alternatively, a cantilever deck slab to extend the seat gap should be provided, with a knock-off backwall tip.

In the case of integral abutments, sufficient reinforcing should be provided in the diaphragm to accommodate higher lateral pressures. For spread footing foundations, knock-off tabs or other fuse elements should be provided to minimize foundation damage. For pile-supported foundations, fuse elements should be used or connection detailing should ensure increased moment ductility in the piles.

7.5.2.2 SDAP D and E

For these design categories passive pressure resistance in soils behind integral abutment walls and knock-off walls for seat abutments will usually be mobilized due to the large longitudinal superstructure displacements associated with the inertial loads. For design purposes static passive pressures may be used without potential reductions associated with inertial loading in abutment backfill. Inclusion of abutment stiffness and capacity in bridge response analyses will reduce ductility demands on bridge columns as discussed in Article C3.3.

Case 1: To ensure that the columns are always able to resist the lateral loads, designers may choose to assume zero stiffness and capacity of abutments. In this case designers should check abutment damage potential and performance due to abutment displacement demand. Knock-off backwall details for seat abutments should be utilized to protect abutment foundations and increased reinforcing used in diaphragms or integral abutments to accommodate passive pressures.

Case 2: Where abutment stiffness and capacity is included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally adapted for static service load design, as illustrated schematically in Figure 7.5.2.2-1. Whether presumptive or computed passive pressures are used for design as described in the commentary paragraphs, backfill in this zone should be controlled by specifications unless the passive pressure that is used in less than 70% of the presumptive value.

Abutment stiffness and passive pressure capacity for either (1) SDAP D or (2) SDAP E two-step analysis methods should be characterized by a bi-linear relationship as shown in Figure 7.5.2.2-2. For seat type abutments, knock-off backwall details should be utilized with superstructure diaphragms designed to accommodate passive pressures, as illustrated in Figure C3.3.1-4. For integral abutments the end diaphragm should be designed for passive pressures, and utilize a stub pile footing or normal footing for support, with a sliding seat. Passive pressures may be assumed uniformly distributed over the height (H) of the backwall or diaphragm. Thus the total passive force is:

$$P_p = p_p * H \qquad (7.5.2.1\text{-}1)$$

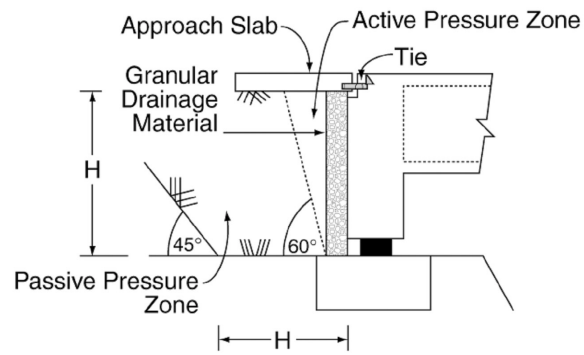


Figure 7.5.2.2-1 Design Passive Pressure Zone

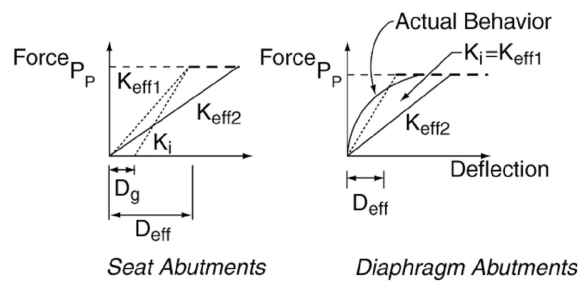


Figure 7.5.2.2-2 Characterization of Abutment Capacity and Stiffness

where:

H = wall height in meters

p_p = passive pressure behind backwall

Calculation of Best-Estimate Passive Force 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 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 7.5.2.2-1. Therefore the properties of backfill that is only placed adjacent to the wall in the active pressure zone may not be appropriate.

If presumptive passive pressures are to be used for design, then the following criteria should apply:

(1) Soil in the "passive pressure zone" should be compacted to a dry density greater than 95 percent of the maximum per ASTM Standard Method D1557 or equivalent.

(2) For cohesionless, non-plastic backfill (fines content less than 30 percent), the passive pressure p_p may be assumed equal to H/10 MPa per meter of length of wall (2H/3 ksf per foot length of wall).

(3) For cohesive backfill (clay fraction > 15 percent), the passive pressure p_p may be assumed equal to 0.25 MPa (5 ksf) provided the estimated unconfined compressive strength is greater than 0.20 MPa (4 ksf).

The presumptive values given above apply for use in the " *Permissible with Owner's Approva*l" category, as defined in Article 3.3.1. If the design is based upon presumptive resistances that are no larger than 70 percent of the values listed above, then the structure may be classified in the "*Permissible"* category.

In all cases granular drainage material must be placed behind the abutment wall to ensure adequate mobilization of wall friction.

Calculation of Stiffness

For SDAP D one-step analyses and for the demand calculation of SDAP E analyses, an equivalent linear secant stiffness, K_{effn}, is required for analyses. For integral or diaphragm abutments, an initial secant stiffness (Figure 7.5.2.2-2) may be calculated as follows:

$$K_{effl} = P_p/0.02H \qquad (7.5.2.2\text{-}2)$$

If computed abutment forces exceed the capacity, the stiffness should be softened iteratively (K_{eff2} to K_{effn}) until abutment displacements are consistent (within 30 percent) with the assumed stiffness. For seat abutments the expansion gap should be included in the initial estimate of the secant stiffness. Thus:

$$K_{effl} = P_p/(0.02H + D_g) \qquad (7.5.2.2\text{-}3)$$

where:

D_g = gap width

For SDAP E two-step analyses, where push-over analyses are conducted, values of P_p and the initial estimate of K_{effl} should be used to define a bilinear load-displacement behavior of the abutment for the capacity assessment.

For partial depth or full-depth seat abutment walls, where knock-off backwalls are activated, the remaining lower wall design and stability check under the action of continuing earthquake-induced active earth pressures should be evaluated. For a no-collapse performance criteria, and assuming conventional cantilever retaining wall construction, horizontal wall translation under dynamic active pressure loading is acceptable. However, rotational instability may lead to collapse and thus must be prevented.

The design approach is similar to that of a free-standing retaining wall, except that lateral force from the bridge superstructure needs to be included in equilibrium evaluations, as the superstructure moves outwards from the wall. Earthquake-induced active earth pressures should be computed using horizontal accelerations at least equal to 50 percent of the site peak ground acceleration (i.e., F_aS_s / 5.0). Using less than the expected site acceleration implies that limited sliding of the wall may occur during the earthquake. A limiting equilibrium condition should be checked in the horizontal direction. To ensure safety against potential overturning about the toe, a restoring moment of at least 50 percent more than the driving overturning moment should exist. If necessary, wall design (initially based on a static loading condition) should be modified to meet the above condition.

7.5.3 Transverse Direction

In general, abutments shall be designed to resist earthquake forces in the transverse direction elastically for the 50% PE in 75-year earthquake. For the 3% PE in 75-year/1.5 mean deterministic event, the abutment may either be designed to

resist transverse forces elastically or a fuse shall be provided to limit the transverse force transfer at the abutment. 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 for the various Seismic Design and Analysis Procedures are listed below.

In the context of these provisions, elastic resistance includes the use of elastomeric, sliding, or isolation bearings designed to accommodate the design displacements, soil frictional resistance acting against the base of a spread footing-supported abutment, 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 in the rare earthquake; or piles that develop a complete plastic mechanism. Article 3.3.1 outlines those mechanisms that are permissible with the Owner's approval.

The stiffness of abutments under transverse loading may be calculated based on the procedures given in Article 8.4 for foundation stiffnesses. Where fusing elements are used, allowance shall be made for the reduced stiffness of the abutment after fusing occurs.

7.5.3.1 SDAP B and C

Connection design forces also apply to shear restraint elements such as shear keys.

7.5.3.2 SDAP D and E

For structures in these categories, 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.

For short, continuous superstructure bridges (Length/Width < 4) with low skew angles (<20 degrees), low plan curvature (subtended angle < 30 degrees), and which also are designed for sustained soil mobilization in the transverse direction, the elastic forces and displacements for the transverse earthquake design may be reduced by 1.4 to account for increased damping provided by the soil at the abutments. Herein transverse earthquake is defined as acting perpendicular to a chord extending between the two abutments. Sustained soil mobilization requires resistance to be present throughout the range of cyclic motion. Where combined mechanisms provide resistance, at least 50 percent of the total resistance must be provided by a sustained mechanism for the system to qualify for the 1.4 reduction.

The design of concrete shear keys should consider the unequal forces that may develop in a skewed abutment, particularly if the intermediate piers are also skewed. (This effect is amplified if intermediate piers also have unequal stiffness, such as wall piers.) The shear key design should also consider unequal loading if multiple shear keys are used. The use of recessed or hidden shear keys should be avoided if possible, since these are difficult to inspect and repair.

7.6 LIQUEFACTION DESIGN REQUIREMENTS

7.6.1 General

An evaluation of the potential for and consequences of liquefaction within near-surface soil shall be made in accordance with the following requirements. A liquefaction assessment is required unless one of the following conditions is met or as directed otherwise by the Owner.

- Mean magnitude for the 3% PE in 75-year event is less than 6.0 (Figures 7.6.1-1 to 7.6.1-4);
- Mean magnitude of the 3% PE in 75-year event is less than 6.4 and equal to or greater than 6.0, and the normalized Standard Penetration Test (SPT) blow count [$(N_1)_{60}$] is greater than 20;
- Mean magnitude for the 3% PE in 75-year event is less than 6.4 and equal to or greater than 6.0, $(N_1)_{60}$ is greater than 15, and F_aS_s is between 0.25 and 0.375.

If the mean magnitude shown in Figures 7.6.1-1 to 7.6.1-4 is greater than or equal to 6.4, or if the above requirements are not met for magnitudes between 6.0 and 6.4, or if for the 50% PE in 75 year event F_aS_s is greater than 0.375, evaluations of liquefaction and associated phenomena such as lateral flow, lateral spreading, and dynamic settlement shall be evaluated in accordance with these Specifications.

7.6.2 Evaluation of Liquefaction Potential

Procedures given in Appendix D shall be used to evaluate the potential for liquefaction.

7.6.3 Evaluation of the Effects of Liquefaction and Lateral Ground Movement

Procedures given in Appendix D shall be used to evaluate the potential for and effects of liquefaction and liquefaction-related permanent ground movement (i.e., lateral spreading, lateral flow, and dynamic settlement). If both liquefaction and ground movement occur, they shall be treated as separate and independent load cases, unless agreed to or directed otherwise by the Owner.

7.6.4 Design Requirements if Liquefaction and Ground Movement Occurs

If it is determined from Appendix D that liquefaction can occur at a bridge site, then one or more of the following approaches shall be implemented in the design.

If liquefaction and no lateral flow occurs, then the bridge shall be designed by conventional procedures including the following requirements:

1. Piled Foundations, Drilled Shafts and Pile Bents: The pile or shaft shall penetrate beyond the bottom of the liquefied layer by at least 3 pile diameters or to a depth that is not affected by liquefaction of the overlying layer or by partial build-up in pore-water pressure, whichever is deeper. In addition the shear reinforcement in a concrete or pre-stressed concrete pile shall meet the requirements of Sec 7.8.2.3 from the pile or bent cap to a depth of 3 diameters below the lowest liquefiable layer.
2. Spread Footings: The bottom of the spread footing shall either be below the liquefiable layer or it shall be at least twice the minimum width of the footing above the liquefiable layer. If liquefaction occurs beneath the base of the footing, the magnitude of settlement caused by liquefaction shall be estimated, and its effects on bridge performance assessed.

If lateral flow or lateral spreading is predicted to occur, the following options shall be considered as detailed in Appendix D.

1. Design the piles or spread footings to resist the forces generated by the lateral spreading.
2. If the structure cannot be designed to resist the forces, assess whether the structure is able to tolerate the anticipated movements and meet the geometric and structural constraints of Table C3.2-1. The maximum plastic rotation of the piles shall be as defined in Article 7.7.9 and 7.8.6.
3. If the structure cannot meet the performance requirements of Table 3.2-1, assess the costs and benefits of various mitigation measures to minimize the movements to a tolerable level to meet the desired performance objective. If a higher performance is desired so that the spread footings or piles will not have to be replaced, the allowable plastic rotations of Articles 7.7.9.2 and 7.8.6.2 shall be met.

7.6.5 Detailed Foundation Design Requirements

Article 7.4 contains detailed design requirements for each of the different foundation types.

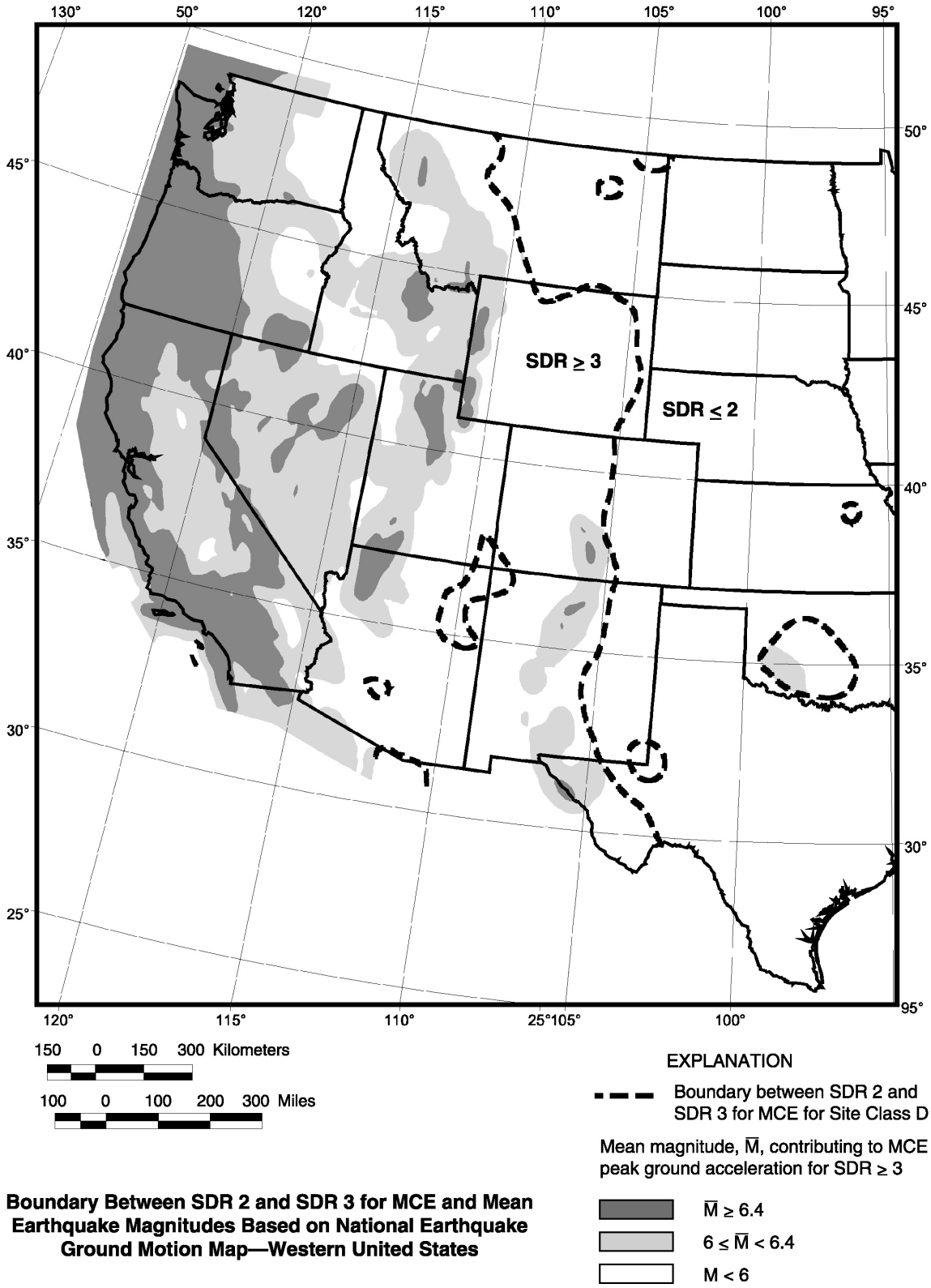


Figure 7.6.1-1 Mean Earthquake Magnitude Map for Western United States

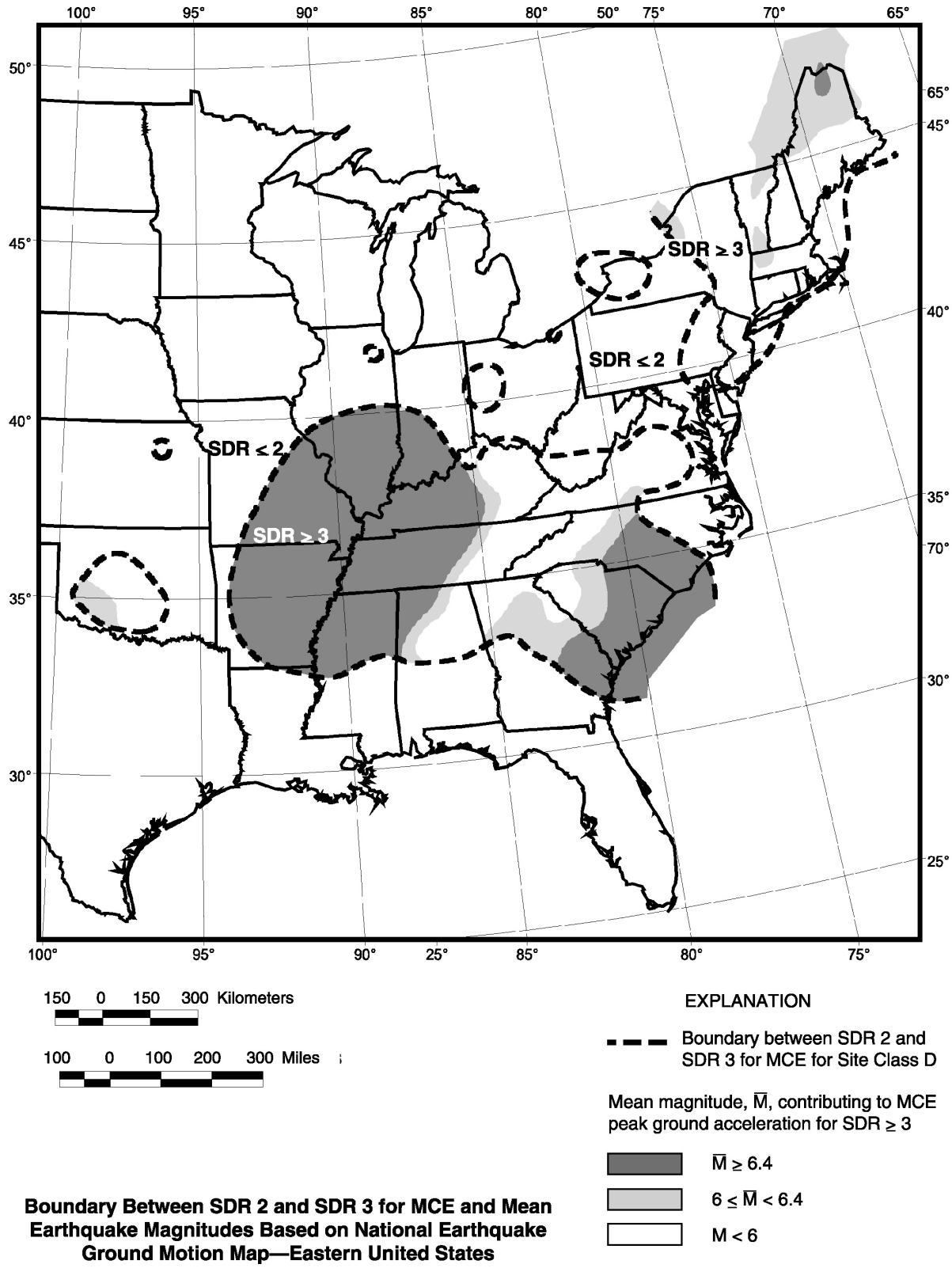


Figure 7.6.1-2 Mean Earthquake Magnitude Map for Central and Eastern United States

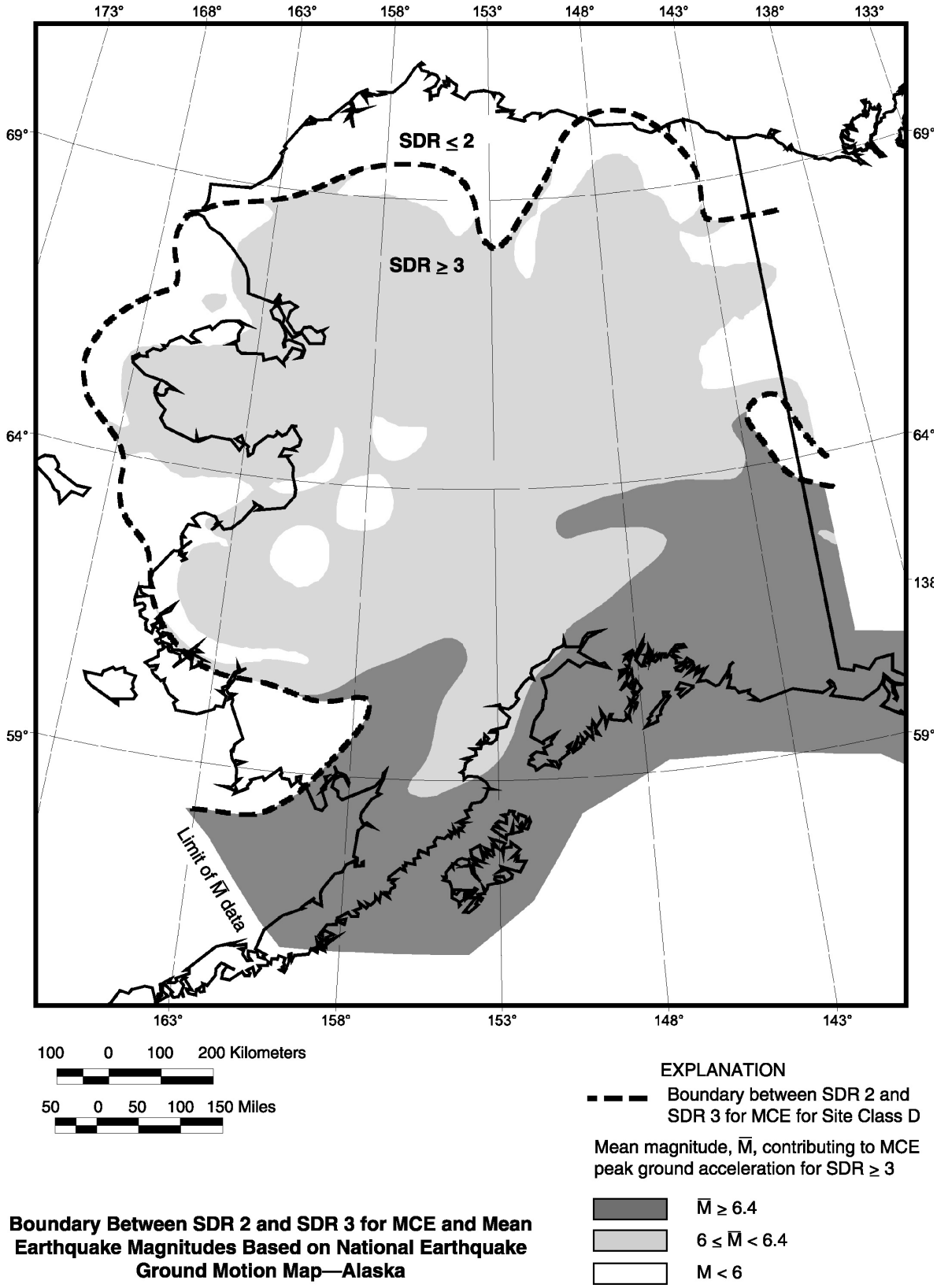
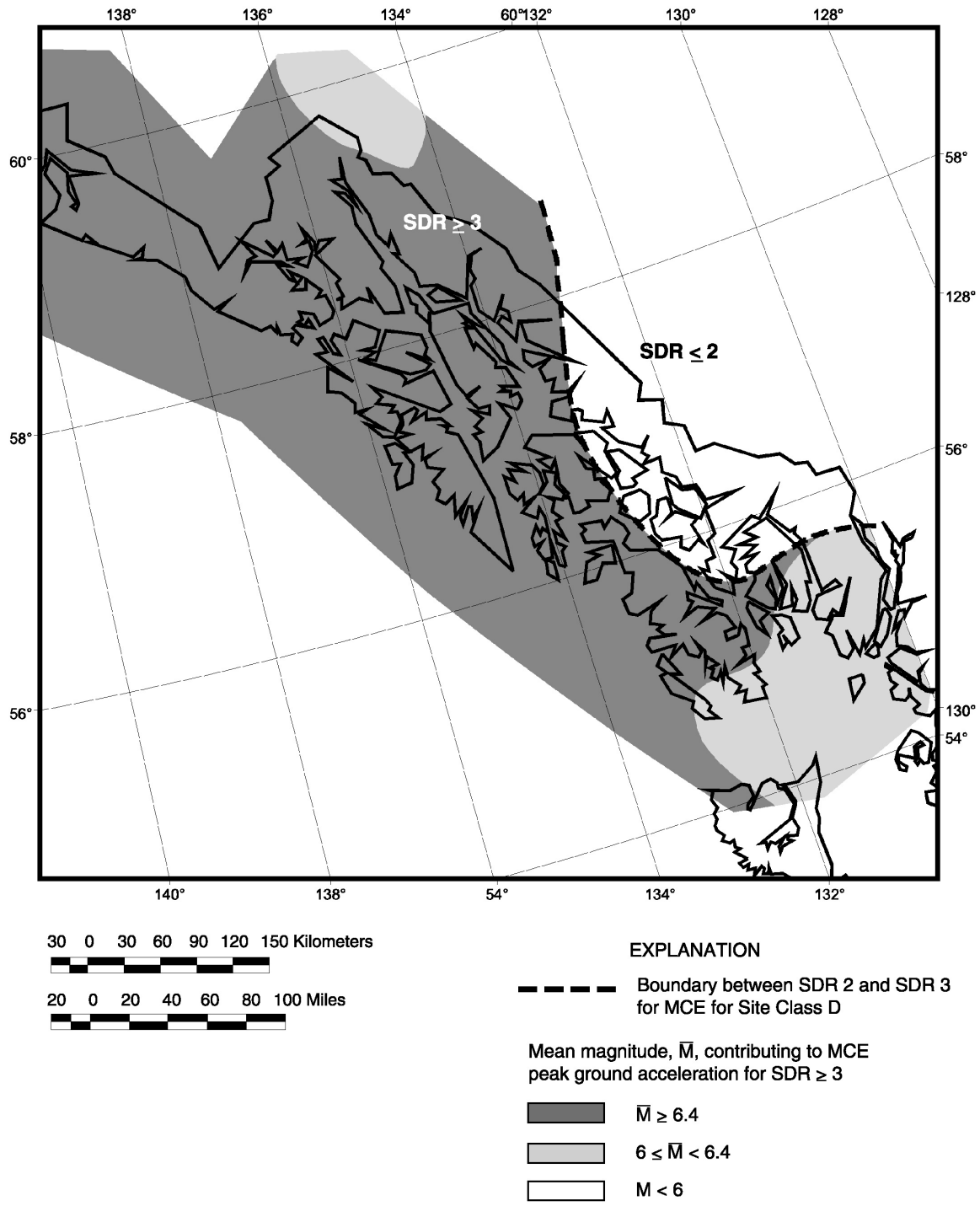


Figure 7.6.1-3 Mean Earthquake Magnitude Map for Northwest Alaska



Boundary Between SDR 2 and SDR 3 for MCE and Mean Earthquake Magnitudes Based on National Earthquake Ground Motion Map—Southeast Alaska

Figure 7.6.1-4 Mean Earthquake Magnitude Map for Southeast Alaska

7.6.6 Other Collateral Hazards

The potential occurrence of collateral hazards resulting from fault rupture, landsliding, differential ground compaction, and flooding and inundation shall be evaluated. Procedures for making these evaluations are summarized in Appendix D.

7.7 STRUCTURAL STEEL DESIGN REQUIREMENTS

7.7.1 General

The provisions of this article shall apply only to a limited number of specially detailed steel components designed to dissipate hysteretic energy during earthquakes. This article does not apply to steel members that are designed to remain elastic during earthquakes.

For the few specially designed steel members that are within the scope of this article, the other requirements of Section 6 of the AASHTO LRFD provisions are also applicable (unless superseded by more stringent requirements of this article).

Continuous and clear load path or load paths shall be assured. Proper load transfer shall be considered in designing foundations, substructures, superstructures and connections.

Welds shall be designed as capacity protected elements. Partial penetration groove welds shall not be used in ductile substructures.

Abrupt changes in cross sections of members in ductile substructures are not permitted within the plastic hinge zones defined in Article 4.9 unless demonstrated acceptable by analysis and supported by research results.

7.7.2 Materials

Ductile substructure elements and ductile end-diaphragms, as defined in Articles 7.7.4 through 7.7.8, shall be made of either:

(a) M270 (ASTM 709M) Grade 345 and Grade 345W steels

(b) ASTM A992 steel, or

(c) A500 Grade B or A501 steels (if structural tubing or pipe).

Other steels may be used provided that they are comparable to the approved Grade 345 steels.

In Article 7.7, nominal resistance is defined as the resistance of a member, connection or structure based on the expected yield strength (F_{ye}), 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.

Overstrength capacity is defined as the resistance of a member, connection or structure based on the nominal dimensions and details of the final section(s) chosen, calculated accounting for the expected development of large strains and associated stresses larger than the minimum specified yield values.

The expected yield strength shall be used in the calculation of nominal resistances, where expected yield strength is defined as $F_{ye} = R_y F_y$ where R_y shall be taken as 1.1 for the permitted steels listed above.

Welding requirements shall be compatible with AWS/AASHTO D1.5-96 Structural Bridge Welding Code. However, under-matched 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 A709/A709M Supplementary Requirement S84 (Fracture Critical). Welds metal connecting these members shall meet the toughness requirements specified in the AWS D1.5 Bridge Specification for Zone III.

7.7.3 Sway Stability Effects

The sway effects produced by the vertical loads acting on the structure in its displaced configuration shall be determined from a second-order analysis. Alternatively, recognized approximate methods for P-Δ analysis, or the provisions in Article 7.3.4, can be used.

7.7.4 Ductile Moment Resisting Frames and Single Column Structures

This article applies to ductile moment-resisting frames and bents, constructed with I-shape beams and columns connected with their webs in a common plane. Except as noted in Article 7.7.4-1, columns shall be designed as ductile structural elements, while the beams, the

panel zone at column-beam intersections and the connections shall be designed as Capacity Protected Elements.

7.7.4.1 Columns

Width-to-thickness ratios of compression elements of columns shall be in compliance with Table 7.7.4-1. Full penetration flange and web welds are required 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 provisions. 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 6.10.7 of the AASHTO LRFD provisions.

The potential plastic hinge zones (Article 4.9), near the top and base of each column, shall be laterally supported and the unsupported distance from these locations shall not exceed $17250r_y/F_y$. These lateral supports shall be provided either directly to the flanges or indirectly through a column web stiffener or a continuity plate. Each column flange lateral support shall resist a force of not less than 2% of the nominal column flange strength (btF_y) at the support location. The possibility of complete load reversal shall be considered.

When no lateral support can be provided, the column maximum slenderness 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 away from the plastic hinge zones as defined in Article 4.9 at a minimum distance equal to the greater of:

(a) one-fourth the clear height of column;
(b) twice the column depth; and
(c) one meter (39 inches).

7.7.4.2 Beams

The factored resistance of the beams shall be determined in accordance with Article 6.10.2 of the AASHTO LRFD provisions. 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. The probable flexural resistance of columns shall be taken as the product of the overstrength factor (defined in Article 4.8) times the columns nominal flexural resistance determined either in accordance to Article 6.9.2.2 of the AASHTO LRFD provisions, or

$$M_{nx} = 1.18M_{px}\left[1-\frac{P_u}{AF_{ye}}\right] \le M_{px} \qquad (7.7.4\text{-}1)$$

unless demonstrated otherwise by rational analysis, and where M_{px} is the column plastic moment under pure bending calculated using F_{ye}.

7.7.4.3 Panel Zones and Connections

Column-beam intersection panel zones, moment resisting connections and column base connections shall be designed as Capacity Protected Elements.

Panel zones shall be designed such that the vertical shearing resistance is determined in accordance with Article 6.10.7.2 of the AASHTO LRFD provisions.

Beam-to-column connections shall have resistance not less than the resistance of the beam stipulated in Article 7.7.4.2.

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. Their b/t shall meet the limits for projecting elements of Article 6.9.4.2 of the AASHTO LRFD provisions. These continuity plates shall be proportioned to meet the stiffener requirements stipulated in Article 6.10.8.2 of the AASHTO LRFD provisions and shall be connected to both flanges and the web.

Flanges and connection plates in bolted connections shall have a factored net section ultimate resistance calculated by Equation 6.8.2.1-2, at least equal to the factored gross area yield resistance given by Equation 6.8.2.1-1, with A_g and A_n in Article 6.8.2.1 taken here as the area of the flanges and connection plates in tension. These referenced equations and article are from the AASHTO LRFD provisions

Table 7.7.4-1 Limiting Width-to-Thickness Ratios

Description of element	Width-to-thickness ratio (b/t)[1]	Limiting width-to-thickness ratio λ_p[2]	Limiting width-to-thickness ratio k[3]
Flanges of I-shaped sections and channels in compression	$\frac{b_f}{2t_f}$	$\frac{135}{\sqrt{F_y}}$	0.30
Webs in combined flexural and axial compression	$\frac{h_c}{t_w}$	**For** $\frac{P_u}{\Phi_b P_y} \le 0.125$ $\frac{1365}{\sqrt{F_y}}\left(1-\frac{1.54P_u}{\Phi_b P_y}\right)$ **For** $\frac{P_u}{\Phi_b P_y} > 0.125$ $\frac{500}{\sqrt{F_y}}\left(2.33-\frac{P_u}{\Phi_b P_y}\right) \ge \frac{665}{\sqrt{F_y}}$	**For** $\frac{P_u}{\Phi_b P_y} \le 0.125$ $3.05\left(1-\frac{1.54P_u}{\Phi_b P_y}\right)$ **For** $\frac{P_u}{\Phi_b P_y} > 0.125$ $1.12\left(2.33-\frac{P_u}{\Phi_b P_y}\right) \ge 1.48$
Hollow circular sections (pipes)	$\frac{D}{t}$	$\frac{8950}{F_y}$	$\frac{200}{\sqrt{F_y}}$
Unstiffened rectangular tubes	$\frac{b}{t}$	$\frac{300}{\sqrt{F_y}}$	0.67
Legs of angles	$\frac{b}{t}$	$\frac{145}{\sqrt{F_y}}$	0.32

1. Width-to-thickness ratios of compression elements – Note that these are more stringent for members designed to dissipate hysteretic energy during earthquake than for other members (Article 6.9.4.2).

2. Limits expressed in format to satisfy the requirement $\frac{b}{t} \le \lambda_p$

3. Limits expressed in format to satisfy the requirement $\frac{b}{t} \le k\sqrt{\frac{E}{F_y}}$

4. Note: In the above, b_f and t_f are respectively the width and thickness of an I-shaped section, h_c is the depth of that section and t_w is the thickness of its web.

7.7.4.4 Multi-tier Frame Bents

For multi-tier frame bents, capacity design principles as well as the requirements of Article 7.7.4.1 may be modified by the engineer to achieve column plastic hinging only at the top and base of the column, and plastic hinging at the ends of all intermediate beams. Column plastic hinging shall not be forced at all joints at every tier.

7.7.5 Ductile Concentrically Braced Frames

Braces are the Ductile Substructure Elements in ductile concentrically braced frames.

7.7.5.1 Bracing Systems

Diagonal braces shall be oriented such that a nearly identical ultimate strength is achieved in both sway directions, when considering only the strength contribution of braces in tension. To achieve this, it is required that, at any level in any planar frame, the sum of the horizontal components of the strength of the braces in tension when the frame sway in one direction, shall be within 30% of the same value for sway in the other direction.

Article 7.7.5 is only applicable to braced frames for which all braces' action lines meet at beam-to-column intersection points (such as X-braces).

7.7.5.2 Design Requirements for Ductile Bracing Members

Bracing members shall have a slenderness ratio, KL/r, less than $2600/\sqrt{F_y}$ or Article 6.9.3 of the AASHTO LRFD Provisions.

The width-to-thickness ratios of bracing members should be limited as indicated in Table 7.7.4-1. For back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio shall not exceed $200/\sqrt{F_y}$, rather the limit of Table 7.7.4-1.

In built-up bracing members, the slenderness ratio of the individual parts between stitches shall be not greater than 0.4 times the slenderness ratio of the member as a whole. When it can be shown that braces will buckle without causing shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio of the individual parts does not exceed 0.75 times the slenderness ratio of the built-up member.

7.7.5.3 Brace Connections

The controlling overstrength capacity shall be taken as the axial tensile yield strength of the brace (A_gF_{ye}). Brace connections shall be designed as Capacity Protected Elements.

Connections must be designed to ensure that the bracing member is capable of yielding the gross section. Consequently, brace strength calculated based on tension rupture on the effective net section and block shear rupture, shall be greater that the design tensile strength of brace given by gross section yielding.

Eccentricities in bracing connections shall be minimized.

Brace connections including gusset plates shall be detailed to avoid brittle failures due to rotation of the brace when it buckles. This ductile rotational behavior shall be allowed for, either in the plane of the frame or out of it, depending on the slenderness ratios.

The design of gusset plates shall also include consideration of buckling.

Stitches that connect the separate elements of built-up bracing members shall, if the overall buckling mode induces shear in the stitches, have a strength at least equal to the design tensile strength of each element. The spacing of stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

7.7.5.4 Columns, Beams and Other Connections

Columns, beams, beam-to-column connections and column splices that participate in the lateral-load-resisting system shall be designed as Capacity Protected Elements with the following additional requirements:

(a) Columns, beams and connections shall resist forces arising from load redistribution following brace buckling or yielding. The brace

compressive resistance shall be taken as 0.3 $\phi_c P_n$ if this creates a more critical condition.

(b) Column splices made with partial penetration groove welds and subject to net tension forces due to overturning effects shall have Factored Resistances not less than 50% of the flange yield load of the smaller member at the splice.

7.7.6 Concentrically Braced Frames with Nominal Ductility

Braces are the Ductile Substructure Elements in nominally ductile concentrically braced frames.

7.7.6.1 Bracing Systems

Diagonal braces shall be oriented such that a nearly identical ultimate strength is achieved in both sway directions, when considering only the strength contribution of braces in tension. To achieve this, it is required that, at any level in any planar frame, the sum of the horizontal components of the strength of the braces in tension when the frame sway in one direction, shall be within 30% of the same value for sway in the other direction.

The categories of bracing systems permitted by this Article includes:

(a) tension-only diagonal bracing,
(b) chevron bracing (or V-bracing) and,
(c) direct tension-compression diagonal bracing systems of the geometry permitted in Article 8.7.5.1, but that do not satisfy all the requirements for ductile concentrically braced frames.

Tension-only bracing systems in which braces are connected at beam-to-column intersections are permitted in bents for which every column is fully continuous over the entire bent height, and where no more than 4 vertical levels of bracing are used along the bent height.

7.7.6.2 Design Requirements for Nominally Ductile Bracing Members

Bracing members shall have a slenderness ratio, KL/r, less than $3750/\sqrt{F_y}$ or Article 6.9.3 of the AASHTO LRFD Provisions. This limit is waived for members designed as tension-only bracing.

In built-up bracing members, the slenderness ratio of the individual parts shall be not greater than 0.5 times the slenderness ratio of the member as a whole.

For bracing members having KL/r less than $2600/\sqrt{F_y}$., the width-to-thickness ratios of bracing members should be limited as indicated in Table 7.7.4-1. For bracing members that exceed that value, the width-to-thickness ratio limits can be obtained by linear interpolation between the values in Table 7.7.4-1 when KL/r is equal to $2600/\sqrt{F_y}$. and 1.3 times the values in Table 7.7.4-1 when KL/r is equal to $3750/\sqrt{F_y}$..

For back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio limit can be taken as $200/\sqrt{F_y}$.

No width-to-thickness ratio limit is imposed for braces designed as tension-only members and having KL/r greater than $3750/\sqrt{F_y}$.

7.7.6.3 Brace Connections

Brace connections shall be designed as Capacity Protected Elements. The controlling overstrength capacity shall be taken as the axial tensile yield strength of the brace ($A_g F_{ye}$).

For tension-only bracing the controlling probable resistance shall be multiplied by an additional factor of 1.10.

Connections must be designed to ensure that the bracing member is capable of yielding the gross section. Consequently, brace strength calculated based on tension rupture on the effective net section and block shear rupture, shall be less that the design tensile strength of brace given by gross section yielding.

Stitches that connect the separate elements of built-up bracing members shall, if the overall buckling mode induces shear in the stitches, have a strength at least equal to one-half of the design tensile strength of each element. The spacing of stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

7.7.6.4 Columns, Beams and Other Connections

Columns, beams, and connections shall be designed as Capacity Protected Elements.

7.7.6.5 Chevron Braced and V-Braced Systems

Braces in chevron braced frames shall conform to the requirements of Article 7.7.6.2, except that bracing members shall have a slenderness ratio, KL/r, less than $2600/\sqrt{F_y}$. Tension-only designs are not permitted.

The beam attached to chevron braces or V-braces shall be continuous between columns and its top and bottom flanges shall be designed to resist a lateral load of 2% of the flange yield force ($F_y b_f t_{bf}$) at the point of intersection with the brace.

Columns, beams and connections shall be designed to resist forces arising from load redistribution following brace buckling or yielding, including the maximum unbalanced vertical load effect applied to the beam by the braces. The brace compressive resistance shall be 0.3 $\phi_c P_n$ if this creates a more critical condition.

A beam that is intersected by chevron braces shall be able to support its permanent dead and live loads without the support provided by the braces.

7.7.7 Concrete Filled Steel Pipes

Concrete-filled steel pipes use 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, 6.12.3.2.2 of the AASHTO LRFD provisions, as well as the requirements in this Article.

7.7.7.1 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 so that:

$$\frac{P_u}{P_r} + \frac{BM_u}{M_{rc}} \le 1.0 \qquad (7.7.7.1\text{-}1)$$

and

$$\frac{M_u}{M_{rc}} \le 1.0 \qquad (7.7.7.1\text{-}2)$$

where P_r is defined in Articles 6.9.2.1 and 6.9.5.1 of the AASHTO LRFD Provisions, and M_{rc} is defined in Article 7.7.7.2

$$B = \frac{P_{ro} - P_{rc}}{P_{rc}} \qquad (7.7.7.1\text{-}3)$$

P_{ro} = factored compressive resistance (Articles 6.9.2.1 and 6.9.5.1 of the AASHTO LRFD Provisions) with $\lambda = 0$

$$P_{rc} = \phi_c A_c f'_c \qquad (7.7.7.1\text{-}4)$$

M_u is the maximum resultant moment applied to the member in any direction, calculated as specified in Article 4.5.3.2.2 of the AASHTO LRFD provisions.

7.7.7.2 Flexural Strength

The factored moment resistance of a concrete filled steel pipe for Article 7.7.7.1 shall be calculated using either of the following two methods:

(a) Method 1 – Using Exact Geometry

$$M_{rc} = \phi_f \ [C_r e + C'_r e'] \qquad (7.7.7.2\text{-}1)$$

where

$$C_r = F_y \beta \frac{Dt}{2} \qquad (7.7.7.2\text{-}2)$$

$$C'_r = f'_c \left[\frac{\beta D^2}{8} - \frac{b_c}{2}\left(\frac{D}{2} - a\right)\right] \qquad (7.7.7.2\text{-}3)$$

$$e = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{1}{\beta} \right] \qquad (7.7.7.2\text{-}4)$$

$$e' = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{b_c^2}{1.5\beta D^2 - 6b_c(0.5D - a)} \right] \qquad (7.7.7.2\text{-}5)$$

$$a = \frac{b_c}{2}\tan\left(\frac{\beta}{4}\right) \qquad (7.7.7.2\text{-}6)$$

$$b_c = D\sin\left(\frac{\beta}{2}\right) \qquad (7.7.7.2\text{-}7)$$

where β is in radians and found by the recursive equation:

$$\beta = \frac{A_s F_y + 0.25 D^2 f'_c\left[\sin(\beta/2) - \sin^2(\beta/2)\tan(\beta/4)\right]}{\left(0.125\ D^2 f'_c + DtF_y\right)} \qquad (7.7.7.2\text{-}8)$$

(b) Method 2 – Using Approximate Geometry

A conservative value of M_{rc} is given by

$$M_{rc} = \phi_f\left[(Z - 2th_n^2)Fy + \left[\frac{2}{3}(0.5D - t)^3 - (0.5D - t)h_n^2\right]f'_c\right] \qquad (7.7.7.2\text{-}9)$$

where

$$h_n = \frac{A_c f'_c}{2Df'_c + 4t(2F_y - f'_c)} \qquad (7.7.7.2\text{-}10)$$

and Z is the plastic modulus of the steel section alone.

For capacity design purposes, in determining the force to consider for the design of capacity protected elements, the moment calculated by this approximate method shall be increased by 10%.

7.7.7.3 Beams and Connections

Capacity-protected members must be designed to resist the forces resulting from hinging in the concrete-filled pipes calculated from Article 7.7.7.2.

7.7.8 Other Systems

This Article provides minimum considerations that must be addressed for the design of special systems.

7.7.8.1 Ductile Eccentrically Braced Frames

Ductile eccentrically braced frames for bents and towers may be used provided that the system, and in particular the eccentric link and link beam, can be demonstrated to remain stable up to the expected level of inelastic response. This demonstration of performance shall be preferably achieved through full-scale cyclic tests of specimens of size greater or equal to that of the prototype.

Seismic design practice for eccentrically braced frames used in buildings can be used to select width-to-thickness ratios, stiffeners spacing and size, and strength of the links, as well as to design diagonal braces and beams outside of the links, columns, brace connections, and beam-to-column connections.

Only the eccentric brace configuration in which the eccentric link is located in the middle of a beam is permitted.

7.7.8.2 Ductile End-Diaphragm in Slab-on-Girder Bridge

Ductile end-diaphragms in slab-on-girder bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

(a) Specially detailed diaphragms capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;
(b) Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;
(c) Design considers the combined and relative stiffness and strength of end-diaphragms and girders (together with their bearing stiffeners) in establishing the diaphragms strength and design forces to consider for the capacity protected elements;
(d) The response modification factor to be considered in design of the ductile diaphragm is given by:

$$R = \left(\frac{\mu + \frac{K_{DED}}{K_{SUB}}}{1 + \frac{K_{DED}}{K_{SUB}}} \right) \qquad (7.7.8.2\text{-}1)$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrated otherwise, μ should not be taken greater than 4;

(e) All details/connections of the ductile end-diaphragms are welded.
(f) 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.
(g) An effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the slab to the diaphragm.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be based on available experimental research results.

7.7.8.3 Ductile End Diaphragms in Deck Truss Bridges

Ductile end-diaphragms in deck-truss bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

(a) Specially detailed diaphragms capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;
(b) Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;
(c) The last lower horizontal cross-frame before each support is also designed as a ductile panel equivalent and in parallel to its adjacent vertical end-diaphragm;
(d) Horizontal and vertical energy dissipating ductile panels are calibrated to have a ratio of stiffness approximately equal to their strength ratio;
(e) The concrete deck is made continuous between supports (and end-diaphragms), and an effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the deck to the diaphragms.;
(h) The response modification factor to be considered in design of the ductile diaphragm is given by:

$$R = \left(\frac{\mu + \frac{K_{DED}}{K_{SUB}}}{1 + \frac{K_{DED}}{K_{SUB}}} \right) \qquad (7.7.8.3\text{-}1)$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrated otherwise, μ should not be taken greater than 4;
(i) All capacity-protected members are demonstrated able to resist without damage or instability the maximum calculated seismic displacements.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be based on available experimental research results.

7.7.8.4 Other Systems

Other framing systems and frames that incorporate special bracing, active control, or other energy- absorbing devices, or other types of special ductile superstructure elements shall be designed on the basis of published research results, observed performance in past earthquakes, or special investigation, and provide a level of safety comparable to those in the AASHTO LRFD Specifications.

7.7.9 Plastic Rotational Capacities

The plastic rotational capacity shall be based on the appropriate performance limit state for the bridge. In lieu of the prescriptive values given below, the designer may determine the plastic rotational capacity from tests and/or a rational analysis.

7.7.9.1 Life Safety Performance

A conservative values of θ_p=0.035 radians may be assumed.

7.7.9.2 Immediate Use Limit State

To ensure the immediate use of the bridge structure following a design ground motion, the maximum rotational capacity should be limited to θ_p=0.005 radians.

7.7.9.3 In Ground Hinges

The maximum rotational capacity for in-ground hinges should be restricted to θ_p=0.01 radians.

7.8 REINFORCED CONCRETE DESIGN REQUIREMENTS

7.8.1 General

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 Article 9.2 of the AASHTO *LRFD Bridge Construction Specifications*.

High strength high alloy bars, with an ultimate tensile strength of up to 1600 MPa, may be used for longitudinal column reinforcement for seismic loading providing it can be demonstrated through tests that the low cycle fatigue properties is not inferior to normal reinforcing steels with yield strengths of 520 MPa or less.

Wire rope or strand may be used for spirals in columns if it can be shown through tests that the modulus of toughness exceeds 100MPa.

In compression members, all longitudinal bars shall be enclosed by perimeter hoops. Ties shall be used to provide lateral restraint to intermediate longitudinal bars within the reinforced concrete cross section.

Transverse hoops and ties that shall be equivalent to:

- No. 10 bars for No. 29 or smaller bars,
- No. 16 bars for No. 36 or larger bars, and
- No. 16 bars for bundled bars.

The spacing of transverse hoops and ties shall not exceed the least dimension of the compression member or 300 mm. Where two or more bars larger than No. 36 are bundled together, the spacing shall not exceed half the least dimension of the member or 150 mm.

Deformed wire**,** wire rope or welded wire fabric of equivalent area may be used instead of bars.

Hoops and ties shall be arranged so that every corner and alternate longitudinal bar has lateral support provided by the corner of a tie having an included angle of not more than 135°. Except as specified herein, no bar shall be farther than 150 mm center-to-center on each side along the tie from such a laterally supported bar.

Where the column design is based on plastic hinging capability, no longitudinal bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where the bars are located around the periphery of a circle, a complete circular tie may be used if the splices in the ties are staggered.

Ties shall be located vertically not more than half a tie spacing above the footing or other support and not more than half a tie spacing below the lowest horizontal reinforcement in the supported member.

7.8.2 Column Pier Requirements

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 not less 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 Article 7.8.3 shall apply.

A pier may be designed as a pier in its strong direction and a column in its weak direction.

The piles of pile bents as well as drilled shaft 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 either "structurally isolated" in such a way that they do not add to the flexural strength capacity of the columns or the column and adjacent structural elements shall be designed to resist the forces generated by increased flexural strength capacity.

The size of the gap required for structural separation is 0.05 times the distance from the center of the column to the extreme edge of the flare, or 1.5 times the calculated plastic rotation from the pushover analysis times the distance from the center of the column to the extreme edge of the flare. Equation 7.8.6.1-4 provides an estimate of the reduced plastic hinge length at this location.

For oversized or architectural portions of piers or columns, minimum longitudinal and transverse reinforcement that complies with temperature and shrinkage requirements elsewhere in these specifications shall be provided.

7.8.2.1 Longitudinal Reinforcement

The area of longitudinal reinforcement shall not be less than 0.008 or more than 0.04 times the gross cross-section area A_g.

7.8.2.2 Flexural Resistance

The biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.6. The column shall be investigated for both extreme load cases, 50% PE in 75 years and 3% PE in 75 years/1.5 mean deterministic as per Articles 4.4, 4.5 and 4.6. The resistance factors of Article 5.5.4.2 of the AASHTO LRFD provisions shall be replaced for both spirally and tied reinforcement columns by the value ϕ = 1.0, providing other member actions have been designed in accordance with the principles of capacity design.

7.8.2.3 Column Shear and Transverse Reinforcement

Provision of transverse reinforcement for shear shall be determined by one of the following two methods: implicit approach or an explicit approach. The implicit approach may be used for all Seismic Hazard Levels. However, for Seismic Hazard Level IV with a two-step design (SDAP E), the shear strength shall be checked using the explicit approach.

Method 1: Implicit Shear Detailing Approach

(a) In potential plastic hinge zones (Article 4.9)

- For circular sections
- For rectangular sections

$$\rho_v = K_{shape}\,\Lambda\,\frac{\rho_t}{\phi}\,\frac{f_{su}}{f_{yh}}\,\frac{A_g}{A_{cc}}\tan\alpha\,\tan\theta \qquad (7.8.2.3\text{-}1)$$

in which

ρ_v = ratio of transverse reinforcement given by either equation 7.8.2.3-2 or 7.8.2.3-3.

- for rectangular sections

$$\rho_V = \frac{A_{sh}}{b_W s} \qquad (7.8.2.3\text{-}2)$$

and

- for circular columns

$$\rho_V = \frac{\rho_s}{2} = \frac{2A_{bh}}{sD''} \qquad (7.8.2.3\text{-}3)$$

where

A_{sh} = the area of the transverse hoops and cross-ties transverse to the axis of bending

A_{bh} = the area of one spiral bar or hoop in a circular section

S = the center-to-center spacing of hoopsets or the pitch the spiral steel

b_w = the web width resisting shear in a rectangular section D'' = spiral diameter in a circular section

The terms in equation (7.8.2.3-1) are defined below:

K_{shape} = factor that depends on the shape of the section and shall be taken as

- for circular sections $K_{shape} = 0.32$
- for square sections with 25 percent of the longitudinal reinforcement placed in each face $K_{shape} = 0.375$
- for walls with strong axis bending $K_{shape} = 0.25$
- for walls with weak axis bending $K_{shape} = 0.5$

$\Lambda =$ fixity factor,
$\Lambda = 1$ fixed-free (pinned one end)
$\Lambda = 2$ fixed-fixed

f_{su} = the ultimate tensile stress of the longitudinal reinforcement. If f_{su} is not available from coupon tests, then it shall be assumed that $f_{su} = 1.5. f_y$. For SDR 2 f_{su} may be taken as f_y.

θ = angle of the principal crack plane given by

$$\tan\theta = \left(\frac{1.6}{\Lambda} \frac{\rho_v}{\rho_t} \frac{A_v}{A_g} \right)^{0.25} \qquad (7.8.2.3\text{-}4)$$

with $\theta \geq 25^\circ$ and $\theta \geq \alpha$

$\alpha =$ geometric aspect ratio angle given by

$$\tan\alpha = \frac{D'}{L}$$

where D' = pitch circle diameter of the longitudinal reinforcement in a circular section, or the distance between the outer layers of the longitudinal steel in other section shapes.

A_v = shear area of concrete which may be taken as $0.8A_g$ for a circular section, or $A_v = b_w d$ for a rectangular section.

The spacing of the spirals or hoopsets shall not exceed 250mm or one-half the member width.

(b) Outside the Potential Plastic Hinge Zone

Outside the potential plastic hinge zone (Article 4.9) the transverse reinforcement may be reduced to account for some contribution of the concrete in shear resistance. The required amount of transverse reinforcement, outside the potential plastic hinge zone ρ_v^*, shall be given by

$$\rho_v^* = \rho_v - 0.17 \frac{\sqrt{f_c'}}{f_{yh}} \qquad (7.8.2.3\text{-}5)$$

where ρ_v = the steel provided in the potential plastic hinge zone.

ρ_v^* shall not be less than the minimum amount of transverse reinforcement required elsewhere in these specifications based on non-seismic requirements.

Method 2: Explicit Approach

The design shear force, V_u, on each principal axis of each column and pile bent shall be determined from considerations of the flexural overstrength being developed at the most probable locations of critical sections within the member, with a rational combination of the most adverse end moments.

In the end regions, the shear resisting mechanism shall be assumed to be provided by a combination of truss (V_s) and arch (strut) action (V_p) such that

$$\phi V_s \geq V_u - \phi(V_p + V_c) \qquad (7.8.2.3\text{-}6)$$

where V_p = the contribution due to arch action given by

$$V_p = \frac{\Lambda}{2} P_e \tan\alpha \qquad (7.8.2.3\text{-}7)$$

where

$$\tan\alpha = \frac{D'}{L} \qquad (7.8.2.3\text{-}8)$$

$P_e =$ compressive axial force including seismic effects

D' = pitch circle diameter of the longitudinal reinforcement in a circular column, or the distance between the outermost layers of bars in a rectangular column

L = column length

Λ = fixity factor defined above

V_c = the tensile contribution of the concrete towards shear resistance. At large displacement ductilities only a minimal contribution can be assigned as follows

$$V_c = 0.05\sqrt{f_c^{'}}\ b_w d \qquad (7.8.2.3\text{-}9)$$

Outside the plastic hinge zone

$$V_c = 0.17\sqrt{f_c^{'}}\ b_w d \qquad (7.8.2.3\text{-}10)$$

where

$f_c^{'}$ = concrete strength in MPa,

b_w = web width of the section, and

d = effective depth

V_s = the contribution of shear resistance provided by transverse reinforcement given by:

(i) for circular columns:

$$V_s = \frac{\pi}{2}\frac{A_{bh}}{s} f_{yh} D^{''} \cot\theta \qquad (7.8.2.3\text{-}11)$$

(ii) for rectangular sections

$$V_s = \frac{A_v}{s} f_{yh} D^{''} \cot\theta \qquad (7.8.2.3\text{-}12)$$

where

A_{bh} = area of one circular hoop/spiral reinforcing bar

A_{sh} = total area of transverse reinforcement in one layer in the direction of the shear force

f_{yh} = transverse reinforcement yield stress

$D^{''}$ = centerline section diameter/width of the perimeter spiral/hoops

θ = principal crack angle/plane calculated as follows:

$$\tan\theta = \left(\frac{1.6\rho_v A_v}{\Lambda \rho_t A_g}\right)^{0.25} \geq \tan\alpha \qquad (7.8.2.3\text{-}13)$$

where

ρ_v = volumetric ratio of shear reinforcement given by

$\rho_v = \dfrac{A_{sh}}{b_w s}$ for rectangular section

$\rho_v = \dfrac{\rho_s}{2} = \dfrac{2A_{bh}}{sD^{''}}$ for circular columns.

and A_v = shear area of concrete which may be taken as $0.8A_v$ for a circular section, or $A_v = b_w d$ for a rectangular section.

Extent of Shear Steel

Shear steel shall be provided in all potential plastic hinge zones as defined in Article 4.9.

7.8.2.4 Transverse Reinforcement for Confinement at Plastic Hinges

The core concrete of columns and pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions. The spacing shall be taken as specified in Article 7.8.2.6.

For a circular column, the volumetric ratio of spiral reinforcement, ρ_s, shall not be less than:

a) for circular sections

$$\rho_s = 0.008\frac{f_c^{'}}{U_{sf}}\left[12\left(\frac{P_e}{f_c^{'}A_g} + \rho_t\frac{f_y}{f_c^{'}}\right)^2\left(\frac{A_g}{A_{cc}}\right)^2 - 1\right] \qquad (7.8.2.4\text{-}1)$$

b) for rectangular sections

$$\frac{A_{sh}}{sB^{''}} + \frac{A_{sh}^{'}}{sD^{''}} = 0.008\frac{f_c^{'}}{U_{sf}}\left[15\left(\frac{P_e}{f_c^{'}A_g} + \rho_t\frac{f_y}{f_c^{'}}\right)^2\left(\frac{A_g}{A_{cc}}\right)^2 - 1\right] \qquad (7.8.2.4\text{-}2)$$

where:

$f_c^{'}$ = specified compressive strength of concrete at 28 days, unless another age is specified (MPa)

f_y = yield strength of reinforcing bars (MPa)

P_e = factored axial load (N) including seismic effects

U_{sf} = strain energy capacity (modulus of toughness) of the transverse reinforcement = 110 MPa.

$\rho_s = \frac{4A_b}{D's}$ = ratio of transverse reinforcement

where

A_b = area of longitudinal reinforcing bars being restrained by rectilinear hoops and/or cross ties.

D' = center-to-center diameter of perimeter hoop for spiral. Within plastic hinge zones, splices in spiral reinforcement shall be made by full-welded splices or by full-mechanical connections.

s = vertical spacing of hoops, not exceeding 100 mm (mm)

A_{cc} = area of column core concrete, measured to the centerline of the perimeter hoop or spiral (mm^2)

A_g = gross area of column (mm^2)

A_{sh} = total area of transverse reinforcement in the direction of the applied shear

A'_{sh} = total area of transverse reinforcement perpendicular to direction of the applied shear

B'' & D'' = core dimension of tied column in the direction under consideration (mm)

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 as specified in Article 5.10.2.2 of the AASHTO LRFD provisions.

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 75 mm at one end and a hook of not less than 90° with an extension not less than six diameters at the other end.
- Hooks shall engage all peripheral longitudinal bars.

Transverse reinforcement meeting the following requirements shall be 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 75 mm 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 75 mm extension that engages the longitudinal reinforcement.

7.8.2.5 Transverse Reinforcement for Longitudinal Bar Restraint in Plastic Hinges

The longitudinal reinforcement in the potential plastic hinge zone shall be restrained by antibuckling steel as follows:

(i) $s \le 6d_b$ (7.8.2.5-1)

(ii) For circular sections confined by spirals or circular hoops

$$\rho_s = 0.016\left(\frac{D}{s}\right)\left(\frac{s}{d_b}\right)\rho_t \frac{f_y}{f_{yh}} \quad (7.8.2.5\text{-}2)$$

(iii) for rectangular sections confined by transverse hoops and/or cross ties the area of the cross tie or hoop legs (A_{bh}) shall be:

$$A_{bh} = 0.09A_b \frac{f_y}{f_{yh}} \quad (7.8.2.5\text{-}3)$$

where

ρ_s = ratio of transverse reinforcement

$\left(\rho_s = \frac{4A_{bh}}{sD},\right)$

D = diameter of circular column

d_b = diameter of longitudinal reinforcing bars being restrained by circular hoop or spiral

A_b = area of longitudinal reinforcing bars being restrained by rectilinear hoops and/or cross ties

A_{bh} = bar area of the transverse hoops or ties restraining

The longitudinal steel

ρ_t = volumetric ratio of longitudinal reinforcement

f_y = yield stress of the longitudinal reinforcement

f_{yh} = yield stress of the transverse reinforcing bars

7.8.2.6 Spacing for Transverse Reinforcement for Confinement and Longitudinal Bar Restraint

Transverse reinforcement for confinement and longitudinal bar restraint (Articles 7.8.2.4 and 7.8.2.5) shall be provided at all plastic hinge zones as defined in Article 4.9 except that the requirements of Article 7.8.2.5 need not apply to the pile length from 3D to 10D below the pile cap.

The spacing of transverse reinforcement shall not be less than:

$$\frac{M}{V}\left(1-\frac{M_y}{M_{po}}\right) \qquad (7.8.2.6\text{-}1)$$

The spacing of transverse reinforcement shall not exceed one-quarter of the minimum member dimension or 150 mm center-to-center.

7.8.2.7 Splices

The provisions of Article 5.11.5 of the AASHTO LRFD provisions shall apply for the design of splices.

Lap splices in longitudinal reinforcement shall be used only within the center half of column height, and the splice length shall not be less than 400mm or 60-bar diameters.

The spacing of the transverse reinforcement over the length of the splice shall not exceed one-quarter of the minimum member dimension.

Full-welded or full-mechanical connection splices conforming to Article 5.11.5 may be used, provided that not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is greater than 450mm measured along the longitudinal axis of the column.

7.8.2.8 Flexural Overstrength

Article 4.8 provides several alternate methods for calculating the flexural moment overstrength capacity (M_{po}) for columns/ piles/ drilled shafts that are part of the ERS. The plastic moment-axial load interaction formula of Equation C8.8.2.8-1 may be used to calculate the overstrength moment of a column or drilled shaft:

7.8.3 Limited Ductility Requirements for Wall Type Piers

These limited ductility provisions, herein specified, shall apply to the design for the strong direction of a pier. Providing ductile detailing is used, either direction of a pier may be designed as a column conforming to the provisions of Article 7.8.2, with the response modification factor for columns used to determine the design forces. If the pier is not designed as a column in either direction, then the limitations for factored shear resistance herein specified shall apply.

The minimum reinforcement ratio, both horizontally, ρ_h, and vertically, ρ_v, in any pier shall not be less than 0.0025. The vertical reinforcement ratio shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 450 mm. The reinforcement required for shear shall be continuous and shall be distributed uniformly.

The factored shear resistance, V_r, in the pier shall be taken as the lesser of:

$$V_r = 0.253\sqrt{f_c'}bd \qquad (7.8.3\text{-}1)$$

$$V_r = \phi V_n \qquad (7.8.3\text{-}2)$$

for which:

$$V_n = \left[0.063\sqrt{f_c'} + \rho_h y_y\right] bd \qquad (7.8.3\text{-}3)$$

Horizontal and vertical layers of reinforcement should be provided on each face of a pier. Splices in horizontal pier reinforcement shall be staggered and splices in the two layers shall not occur at the same location.

7.8.4 Moment Resisting Connection Between Members (Column/Beam and Column/Footing Joints)

7.8.4.1 Implicit Approach: Direct Design

Flexural reinforcement in continuous, restrained, or cantilever members or in any member of a rigid frame shall be detailed to provide continuity of reinforcement at intersections with other members to develop the nominal moment resistance of the joint.

In SDR 3 and above, joints shall be detailed to resist shears resulting from horizontal loads through the joint.

Transverse reinforcement in cap beam-to-column or pile cap-to-column joints should consist of the greater of:

(a) Confinement reinforcement given in Article 7.8.2.4;

(b) Longitudinal bar restraint reinforcement given in Article 7.8.2.5; this article can be waived if the longitudinal bars framing into the joint are surrounded by sufficient concrete to inhibit bar buckling. For the purpose of waiving this article cover to the longitudinal steel shall be taken as the greater of 150 mm or 6 longitudinal bar diameters.

(c) Shear reinforcement given by Article 7.8.2.3 where the principal crack angle θ is given by the aspect ratio of the member and is defined by the joint dimensions as follows

$$\tan\theta = \tan\alpha = \frac{D}{H_c}$$

where

$D =$ width or diameter of the column framing into the joint

$H_c =$ the height of the cap beam/joint. Thus the joint shear horizontal (transverse) reinforcement is given by:

For circular columns with spirals or circular hoops

$$\rho_s \geq 0.76 \frac{\rho_t}{\phi} \frac{f_{su}}{f_{yh}} \frac{A_g}{A_{cc}} \tan^2 \alpha. \qquad (7.8.4.1\text{-}1)$$

for rectangular sections with rectilinear hoops and/or ties

$$\frac{A_{sh}}{sB''} \geq 1.2 \frac{B'/D' + 0.5}{2B'/D' + 2} \frac{\rho_t}{\phi} \frac{f_{su}}{f_{yh}} \frac{A_g}{A_{cc}} \tan^2 \alpha \qquad (7.8.4.1\text{-}2)$$

If the above equations lead to congested steel placement details, then alternative details may be adopted through the use of rational strut and tie models as given in Article 7.8.4.2.

where

$\rho_s =$ ratio of transverse hoops/spirals $\left(\rho_s = \frac{4A_{bh}}{sD'}\right)$

$\rho_t =$ ratio of longitudinal reinforcement area to gross area of section

$A_{sh} =$ area of transverse reinforcement in the direction of the applied shear

$f_{su} =$ yield strength of transverse reinforcement

$A_g =$ gross area of section

$A_{cc} =$ confined core area (take as $0.8A_g$ for a circular section

$\phi =$ resistance factor for seismic shear (0.85)

7.8.4.2 Explicit Approach: Detailed Design

7.8.4.2.1 Design Forces and Applied Stresses

Moment-resisting connections between members shall be designed to transmit the maximum forces applied by the connected members. Connection forces shall be based on the assumption of maximum plastic moment.

Forces acting on the boundaries of connections shall be considered to be transmitted by mechanisms involving appropriate contributions by concrete and reinforcement actions. Mechanisms shall be based on an analysis of force-transfer within the connection, and shall be supported by relevant test results.

Principal stresses is any vertical plane within a connection shall be calculated in accordance with Eq. (7.8.4.2-1) and (7.8.4.2-2)

Principal tension stress is given by:

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \quad (7.8.4.2\text{-}1)$$

Principal compression stress is given by:

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \quad (7.8.4.2\text{-}2)$$

where

f_h and f_v = the average axial stresses in the horizontal and vertical directions within the plane of the connection under consideration (compression stress positive) and

v_{hv} = the average shear stress within the plane of the connection.

7.8.4.2.2 Minimum Required Horizontal Reinforcement

When the principal tension stress is less than $P_t = 0.29\sqrt{f_c^{'}}$ MPa, the minimum amount of horizontal joint shear reinforcement to be provided shall be capable of transferring 50 percent of the cracking stress resolved to the horizontal direction.

For circular columns, or columns with intersecting spirals, the volumetric ratio of transverse reinforcement in the form of spirals or circular hoops to be continued into the cap or footing shall not be less than

$$\rho_s = \frac{0.29\sqrt{f_c^{'}}}{f_{yh}} \quad (7.8.4.2\text{-}3)$$

where

f_{yh} = yield stress of horizontal hoop/tie reinforcement in the joint.

7.8.4.2.3 Maximum Allowable Compression Stresses

Principal compression stress in a connection, calculated in accordance with Equation 7.8.4.2-2 shall not exceed $p_c = 0.25 f_c^{'}$.

7.8.4.3 Reinforcement for Joint Force Transfer

7.8.4.3.1 Acceptable Reinforcement Details

Where the magnitude of principal tension stress values (calculated in accordance with Equation 7.8.4.2-1), exceed $\rho_t = 0.29\sqrt{f_c^{'}}$ MPa, vertical and horizontal joint rein-forcement, placed in accordance with Articles 7.8.4.3.2, 7.8.4.3.3 and 7.8.4.3.4.is required.

7.8.4.3.2 Vertical Reinforcement

Stirrups

On each side of the column or pier wall, the beam member that is subject to bending forces shall have vertical stirrups, with a total area $A_{jv} = 0.16 A_{st}$ located within a distance $0.5D$ or $0.5h$ from the column or pier wall face. These vertical stirrups shall be distributed over a width not exceeding $2D$.

where

A_{st} = total area of longitudinal steel

D = diameter of circular column

h = depth of rectangular column

Clamping Reinforcement

Longitudinal reinforcement contributing to cap beam or footing flexural strength (i.e., superstructure top reinforcement, cap top reinforcement, footing bottom reinforcement) shall be clamped into the joint by vertical bars providing a total area of $0.08A_{ST}$. These bars shall be hooked around the restrained longitudinal reinforcement and extend into the joint a distance not less than two-thirds of the joint depth. If more than 50 percent of the superstructure moment capacity and/or cap-beam moment capacity is provided by prestress, this reinforcement may be omitted, unless needed for the orthogonal direction of response.

7.8.4.3.3 Horizontal Reinforcement

Additional longitudinal reinforcement in the cap beam, superstructure, and footing of total amount $0.08A_{ST}$ over and above the required for flexural strength, shall be placed in the face adjacent to the column (i.e., bottom of cap beam or superstructure; top of footing), extending through the joint and for a sufficient distance to develop its yield strength at a distance of $0.5D$ from the column face, as shown in Figure 7.8.4.2-1

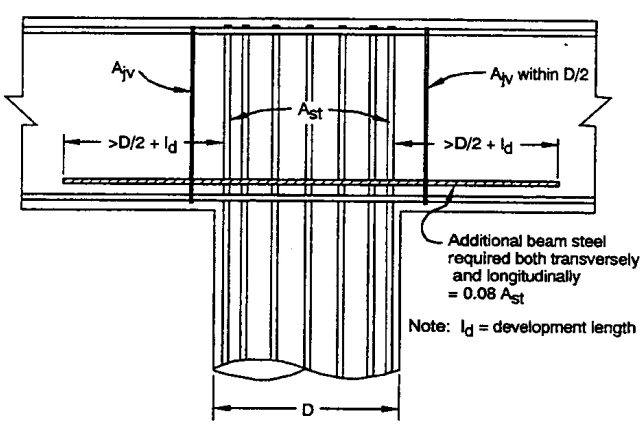


Figure 7.8.4.2-1 Additional cap beam bottom reinforcement for joint force transfer.

7.8.4.3.4 Hoop or Spiral Reinforcement

The required volumetric ratio of column joint hoop or spiral reinforcement to be carried into the cap or footing shall not be less than

$$\rho_s \geq \frac{0.4A_{ST}}{\ell_{ac}^2} \qquad (7.8.4.3\text{-}1)$$

7.8.4.4 Footing Strength

In determining the flexural strength of footings resisting gravity plus seismic overloads, with monolithic column/footing connections, the effective width of the footing shall not be taken to be greater than the width of the column plus a tributary footing width, equal to the effective depth of the footing, on either side of the column.

The effective width for determining the shear strength of footings for gravity plus seismic overloads shall be as for flexural overstrength

When the nominal shear strength in footings arising from the maximum flexural overstrength, vertical stirrups or ties shall be provided to carry the deficit in shear strength. These stirrups shall be placed within the effective width as defined above.

7.8.5 Concrete Piles

7.8.5.1 Transverse Reinforcement Requirements

The upper end of every pile shall be reinforced and confined as a potential plastic hinge region as specified in Article 4.9, except where it can be established that there is no possibility of any significant lateral deflection in the pile. If an analysis of the bridge and pile system indicates that a plastic hinge can form at a lower level, the plastic hinge zone shall extend 3D below the point of maximum moment. The transverse reinforcement in the top 3D of the pile shall be detailed for the maximum of shear, confinement, and longitudinal bar restraint as for concrete columns described in Article 7.8.2. The top 10D of the pile shall be detailed for the maximum of shear and confinement as for concrete columns and described in Articles 7.8.2.3 and 7.8.2.4.

7.8.5.2 Volumetric Ratio of Transverse Reinforcement

In lieu of a precise soil structure interaction analysis to ascertain the shear demand, a value of α = 25 degrees may be assumed for use in the implicit shear design equations.

7.8.5.3 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, provided by not less than four bars, shall not be less than 0.008**.**

7.8.6 Plastic Rotation Capacities

The plastic rotational capacity shall be based on the appropriate performance limit state for the bridge. In lieu of the prescriptive values given below, the designer may determine the plastic rotational capacity from tests and/or a rational analysis.

7.8.6.1 Life Safety Performance

The plastic rotational capacity of hinges shall be based on

$$\theta_p = 0.11\frac{L_p}{D'}\left(N_f\right)^{-0.5} \qquad rad \quad (7.8.6.1\text{-}1)$$

in which

N_f = number of cycles of loading expected at the maximum displacement amplitude which may be estimated from

$$N_f = 3.5\left(T_n\right)^{-1/3} \qquad (7.8.6.1\text{-}2)$$
$$2 \le N_f \le 10$$

where T_n = natural period of vibration of the structure.

For liquifiabile soils and piled foundation assessment, use $N_f = 2$

L_p = effective plastic hinge length give by

$$L_p = 0.08\frac{M}{V} + 4400\varepsilon_y d_b \qquad (7.8.6.1\text{-}3)$$

where
M/V = shear span of the member (M = end moment V = shear force)
ε_y = yield strain of the longitudinal reinforcement;

When an isolation gap of length L_g is provided between a structurally separated flare and an adjacent structural element, the plastic hinge length is given by

$$L_p = L_g + 8800\varepsilon_y d_b \qquad (7.8.6.1\text{-}4)$$

where L_g is the gap between the flare and the adjacent element.

D' = the distance between the outer layers of the longitudinal reinforcement on opposite faces of the member, equal to the pitch circle diameter for a circular section.

d_b = diameter of the main longitudinal reinforcing bars.

In lieu of the precise analysis given above, a conservative value of $\theta_p = 0.035 \quad rad$ shall be assumed.

For life-safety assessment of pile foundations that are potentially liquifiable, then $\theta_p = 0.05 rad$

7.8.6.2 Immediate Use Performance

To ensure the immediate use of the bridge structure following a design ground motion, the maximum rotational capacity should be limited to $\theta_p = 0.01 \quad rad$.

7.8.6.3 In-Ground Hinges

The maximum rotational capacity for in-ground hinges shall be restricted to $\theta_p = 0.02$ rad.

7.9 BEARING DESIGN REQUIREMENTS

There are three design or testing alternates for bearings that are not designed and tested as seismic isolation bearings as per Article 7.10. Alternate 1 requires both prototype and quality

control testing of bearings as per Article 7.9.1. If testing of bearings is not performed for the required forces and displacements, then Alternate 2 provides a design option to provide a positive restraint system for the bearing. The restraint shall be capable of resisting the forces generated in the 3% PE in 75 year/1.5 mean deterministic event utilizing an analytical model that assumes that all bearings so designed are restrained. Alternate 3 provides a design option that permits a bearing to fail, provided there is a flat surface on which the girders can slide. The bearing or masonry plinth cannot impede the movement. The bridge must be analyzed in this condition and allowance for 150% of the calculated movement shall be provided.

If Alternate 3 is selected then a non-linear time history analysis is required using an appropriate coefficient of friction for the sliding surface to determine the amount of displacement that will result. The bearings shall be assumed to have failed early in the time history so a conservative value of the displacement is obtained.

7.9.1 Prototype and Quality Control Tests

Prototype Tests – each manufacturer shall perform a set of prototype tests on two full size bearings to qualify that particular bearing type and size for the rated forces or displacements of it's application. The sequence of tests shall be those given in Article 15.10.2 for the displacement or force for which it is to be qualified. For fixed bearings, the sequence of tests shall be performed for 110% of the lateral force capacity of the bearing where 110% of the force capacity replaces the total design displacement in Article 15.10.2. For bearings that permit movement, the total design displacement shall be 110% of the displacement for which they are to be qualified.

Quality Control Tests – a set of quality control tests shall be performed on 1 out of every 10 bearings of a given type and size. The tests shall be similar to those required for isolation bearings as specified in Articles 15.12.2, 15.14.2 and 15.15.6. For fixed bearings, the total design displacement shall be replaced by the lateral force capacity for which they are qualified.

7.10 SEISMIC ISOLATION DESIGN REQUIREMENTS

The design and testing requirements for the isolators are given in Articles 15.12 through 15.15

The analysis requirements for a seismically isolated bridge are given in Article 5.3.6 and Article 5.4.1.1 for the capacity spectrum method and Article 5.4.2.3 for a multi-mode analysis and Article 5.4.4 for a nonlinear time-history analysis. Other analysis and modeling issues are given in Article 15.4 and design properties of the isolators are given in Article 15.5. If an upper and lower bound analysis is performed as per Article 15.4, then the design forces and displacement shall be the maximum of those obtained from the upper and lower bound analyses respectively.

The supporting substructures may be all designed elastically using the provisions of Article 4.10. If an R of 1.5 as per Table 4.7-1 is used to design the substructure, all other elements connected to the column shall be designed using the Capacity Design procedures of Article 4.8. The design and testing of the isolator units is given in Article 15.10 and other design issues related to the isolators are given in Section 15.

Section 8
SEISMIC DESIGN REQUIREMENTS (SDR) 4, 5 and 6

8.1 GENERAL

Bridges classified as SDR 4, 5, and 6 in accordance with Table 3.7-2 of Article 3.7 shall conform to all of the requirements of this section. SDR 5 and 6 bridges are not permitted to use ERE or ERS (Article 3.3.1) that require owners approval. SDR 6 bridges also require approach slabs and, although not mandated for SDR 5, the use of approach slabs is encouraged.

8.2 DESIGN FORCES

8.2.1 Ductile Substructures (R>1) — Flexural Capacity

8.2.1.1 SDAP C

The sum of the capacities of all columns must satisfy Article 5.4.1.

8.2.1.2 SDAP D and E

Column design forces are the maximum of those obtained from an elastic analysis and reduced using the appropriate R-Factor as specified in Steps 2, 3 and 4 of Article 4.5 and combined in accordance with Article 3.6.

8.2.2 Capacity Protected Elements or Actions

The design provisions of Article 4.8 apply to capacity protected elements and actions.

Capacity design principles require that those elements not participating as part of the primary energy dissipating system (flexural hinging in columns), such as column shear, joints and cap beams, spread footings, pile caps and foundations be "capacity protected". This is achieved by ensuring the maximum overstrength moment and shear from plastic hinges in the columns can be dependably resisted by adjoining elements.

Exception: Elastic design of all substructure elements (Article 4.10), seismic isolation design (Article 8.10) and in the transverse direction of a column when a ductile diaphragm is used (Article 8.7.8.2).

8.2.3 Elastically Designed Elements

There may be instances where a designer chooses to design all of the substructure supports elastically (i.e., R=1.0 for all substructures) or in some cases a limited number of substructure elements are designed elastically. If so, the provisions of Article 4.10 apply.

8.2.4 Abutments and Connections

The seismic design forces for abutments are obtained by SDAP D or E when required and given in Article 8.5. The seismic design forces for connections are the lower of those obtained from Article 8.2.2 or the elastic forces divided by the appropriate R-Factor from Table 4.7-2.

8.2.5 Single Span Bridges

For single-span bridges, regardless of seismic zone and in lieu of a rigorous analysis, the minimum design force at the connections in the restrained direction between the superstructure and the substructure shall not be less than the product of $F_a S_S / 2.5$, and the tributary permanent load.

8.3 DESIGN DISPLACEMENTS

8.3.1 General

For this section, displacement is the displacement at the center of mass for a pier or bent in the transverse or longitudinal direction determined from the seismic analysis except in Article 8.3.2 where the displacement occurs at the bearing seat.

8.3.2 Minimum Seat Width Requirement

The seat width shall not be less than 1.5 times the displacement of the superstructure at the seat according to Equation (8.3.4-2) or:

$$N = \left[0.10 + 0.0017L + 0.007H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(2\frac{B}{L}\right)^2} \right] \frac{(1 + 1.25F_v S_1)}{\cos\alpha} \quad (8.3.2\text{-}1)$$

where,

L = distance between joints in meters
H = tallest pier between the joints in meters
B = width of the superstructure in meters
α = skew angle

The ratio B/L need not be taken greater than 3/8.

8.3.3 Displacement Compatibility

All components that are not designed to resist seismic loads must have deformation capacity sufficient to transfer non-seismic loads.

8.3.4 P-Δ Requirements

The displacement of a pier or bent in the longitudinal and transverse direction must satisfy

$$\Delta \leq 0.25 C_c H \quad (8.3.4\text{-}1)$$

where,

$$\Delta = R_d \Delta_e \quad (8.3.4\text{-}2)$$

$$R_d = \left(1 - \frac{1}{R}\right)\frac{1.25T_s}{T} + \frac{1}{R} \text{ for } T < 1.25T_s \quad (8.3.4\text{-}3)$$

where T_s is defined in Figure 3.4.1-1, otherwise $R_d = 1$,

Δ_e is the displacement demand from the seismic analysis, R is the ratio between elastic lateral force and the lateral strength of the pier or bent, C_c is the seismic coefficient based on the lateral strength of the pier or bent ($C_c = V/W$ where V is the lateral strength), and H is the height of the pier from the point of fixity for the foundation.

If a nonlinear time history seismic analysis is performed, the displacement demand, Δ, may be obtained directly from the analysis in lieu of Equation 8.3.4-2. However, the displacement Δ shall not be taken less than 0.67 of the displacement determined from an elastic response spectrum analysis.

8.3.5 Minimum Displacement Requirements for Lateral Load Resisting Piers and Bents

For SDAP E the maximum permitted displacement capacity from the Displacement Capacity Verification must be greater than the displacement demand according to the following requirement:

$$1.5\Delta \leq \Delta_{capacity} \quad (8.3.5\text{-}1)$$

where the Δ is defined in Article 8.3.4 and $\Delta_{capacity}$ is the maximum displacement capacity per Article 5.4.3.

When a nonlinear dynamic analysis is performed the displacement demand may not be taken less than 0.67 times the demand from a elastic response spectrum analysis, nor may the displacement capacity be taken greater than the capacity from the Displacement Capacity Verification.

8.4 FOUNDATION DESIGN REQUIREMENTS

8.4.1 Foundation Investigation

8.4.1.1 General

A subsurface investigation, including borings and laboratory soil tests, shall be conducted in accordance with the provisions of Appendix B to provide pertinent and sufficient information for the determination of the Site Class of Article 3.4.2.1. The type and cost of foundations should be considered in the economic, environmental, and aesthetic studies for location and bridge type selection.

8.4.1.2 Subsurface Investigation

Subsurface explorations shall be made at pier and abutment locations, sufficient in number and depth, to establish a reliable longitudinal and transverse substrata profile. Samples of material encountered shall be taken and preserved for future reference and/or testing. Boring logs shall be prepared in detail sufficient to locate material strata, results of penetration tests, groundwater, any artesian action, and where samples were taken. Special attention shall be paid to the detection of narrow, soft seams that may be located at stratum boundaries.

8.4.1.3 Laboratory Testing

Laboratory tests shall be performed to determine the strength, deformation, and flow characteristics of soils and/or rocks and their suitability for the foundation selected. In areas of higher seismicity (e.g., SDR 3, 4, 5, and 6), 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, if unusual soils exist or if the foundation is supporting a critical bridge.

8.4.2 Spread Footings

The design of spread footing foundations located in SDR 4, 5, and 6 shall be based on column moments and shears developed using capacity design principles as described in Section 4.8.

Foundation flexibility (Article 5.3.4) shall be modeled for Soil Types C, D, and E if foundation flexibility results in more than a 20 percent change in response (Article C5.3.4). For Soil Types A and B, soil flexibility does not need to be considered because of the stiffness of the soil or rock. The potential for and effects of liquefaction and dynamic settlement shall also be determined for spread footing foundations subject to SDR 4 and above. Normally, spread footings shall not be located at sites within SDR 4, 5, and 6 where liquefaction is predicted to occur, unless:

- The foundation is located below the liquefiable layer.
- It can be demonstrated by special studies that liquefaction and its effects are very limited, or
- The ground will be improved such that liquefaction will not occur.

Owner approval shall be obtained before proceeding with a spread footing design at a site where liquefaction is predicted to occur.

8.4.2.1 Spring Constants for Footing (Nonliquefiable Sites)

When required to represent foundation flexibility, spring constants shall be developed for spread footing using equations given in Tables 8.4.2.1-1 and 8.4.2.1-2. Alternate procedures given in FEMA 273 (1997) are also suitable for estimating spring constants. These computational methods are appropriate for sites that do not liquefy or lose strength during earthquake loading. See Article 8.4.2.3 for sites that are predicted to liquefy.

The shear modulus (G) used to compute the stiffness values in Table 8.4.2.1-1 shall be determined by adjusting the low-strain shear modulus (G_{max}) for the level of shearing strain using the following strain adjustment factors, unless other methods are approved by the Owner.

$\underline{F_v S_1 \leq 0.40}$

- $G/G_{max} = 0.50$ for 50% in 75-year event
- $G/G_{max} = 0.25$ for 3% in 75-year event

$\underline{F_v S_1 > 0.40}$

- $G/G_{max} = 0.25$ for 50% in 75-year event
- $G/G_{max} = 0.10$ for 3% in 75-year event

Uplift shall be allowed for footings subject to SDR 4, 5, and 6. The following area adjustment factors (R_a) shall be applied to the equivalent area to account for geometric nonlinearity introduced by uplift, unless the Owner approves otherwise.

$\underline{F_v S_1 \leq 0.40}$

- $R_a = 1.0$ for the 50% in 75-year event
- $R_a = 0.75$ for the 3% in 75-year event

Table 8.4.2.1-1. Surface Stiffnesses for a Rigid Plate on a Semi-Infinite Homogeneous Elastic Half-Space (adapted from Gazetas, 1991)[1]

Stiffness Parameter	*Rigid Plate Stiffness at Surface, Ki'*
Vertical Translation, K_z'	$\frac{GL}{1-\nu}\left[0.73+1.54\left(\frac{B}{L}\right)^{0.75}\right]$
Horizontal Translation, K_y' (toward long side)	$\frac{GL}{2-\nu}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right]$
Horizontal Translation, K_x' (toward short side)	$\frac{GL}{2-\nu}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right]-\frac{GL}{0.75-\nu}\left[0.1\left(1-\frac{B}{L}\right)\right]$
Rotation, $K_{\theta x}$' (about x axis)	$\frac{G}{1-\nu}\ I_X^{0.75}\left(\frac{L}{B}\right)^{0.25}\left(2.4+0.5\frac{B}{L}\right)$
Rotation, $K_{\theta y}$' (about y axis)	$\frac{G}{1-\nu}\ I_Y^{0.75}\left[3\left(\frac{L}{B}\right)^{0.15}\right]$

1. See Figure 8.4.2.1-1** for definitions of terms

Table 8.4.2.1-2. Stiffness Embedment Factors for a Rigid Plate on a Semi-Infinite Homogeneous Elastic Half-Space (adapted from Gazetas, 1991)[1]

Stiffness Parameter	*Embedment Factors, ei*
Vertical Translation, e_z	$\left[1+0.095\frac{D}{B}\left(1+1.3\frac{B}{L}\right)\right]\left[1+0.2\left(\frac{(2L+2B)}{LB}d\right)^{0.67}\right]$
Horizontal Translation, e_y (toward long side)	$\left[1+0.15\left(\frac{2D}{B}\right)^{0.5}\right]\left\{1+0.52\left[\frac{\left(D-\frac{d}{2}\right)16\,(L+B)\,d}{BL^2}\right]^{0.4}\right\}$
Horizontal Translation, e_x (toward short side)	$\left[1+0.15\left(\frac{2D}{L}\right)^{0.5}\right]\left\{1+0.52\left[\frac{\left(D-\frac{d}{2}\right)16\,(L+B)\,d}{LB^2}\right]^{0.4}\right\}$
Rotation, $e_{\theta x}$ (about x axis)	$1+2.52\frac{d}{B}\left(1+\frac{2d}{B}\left(\frac{d}{D}\right)^{-0.20}\left(\frac{B}{L}\right)^{0.50}\right)$
Rotation, $e_{\theta y}$ (about y axis)	$1+0.92\left(\frac{2d}{L}\right)^{0.60}\left(1.5+\left(\frac{2d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.60}\right)$

Note. Embedment factors multiplied by spring

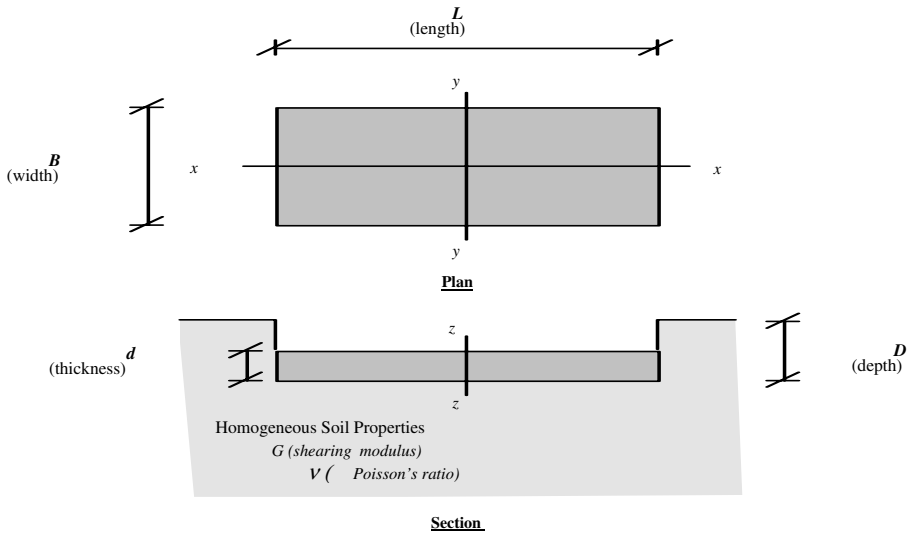


Figure 8.4.2.1-1. Properties of a Rigid Plate on a Semi-infinite Homogeneous Elastic Half-Space for Stiffness Calculations

$F_vS_1 > 0.40$

- $R_a = 0.75$ for the 50% in 75-year event
- $R_a = 0.5$ for the 3% in 75-year event

Values of G_{max} shall be determined by seismic 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). The uncertainty in determination of G_{max} shall be considered when establishing strain adjustment factors.

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.

8.4.2.2 Moment-Rotation and Shear-Displacement Relationships for Footing (Nonliquefiable Sites)

The moment and shear capacity of the foundation shall be confirmed for design loads given in Article 4.8. Moment-rotation and shear force-displacement relationships shall be developed as required by Article 5.3.4. Unless approved otherwise by the Owner, the moment-rotation curve for SDAP E shall be represented by a bilinear, moment-rotation curve. The initial slope of the bi-linear curve shall be defined by the rotational spring constant given in Article 8.4.2.1.

The maximum resisting force (i.e., plastic capacity) on the force-deformation curve shall be defined for the best-estimate case. The footing liftoff shall be no more that 50 percent at peak displacement during the push-over analysis, unless special studies are performed and approved by the Owner. A bilinear force displacement relationship shall also be developed for the shear component of resistance.

This approach shall not be used at sites that will liquefy during seismic loading. See Article 8.4.2.3 for sites that liquefy.

8.4.2.3 Liquefaction and Dynamic Settlement

An evaluation of the potential for liquefaction within near-surface soil shall be made in accordance with requirements given in Article 8.6 and Appendix D of these Specifications. If liquefaction is predicted to occur under the design ground motion, spread footings foundations shall not be used unless

- the footing is located below the liquefiable layer
- ground improvement is performed to mitigate the occurrence of liquefaction, or
- special studies are conducted to demonstrate that the occurrence of liquefaction will not be

detrimental to the performance of the bridge support system.

The Owner's approval shall be obtained before initiating ground improvement or special studies.

8.4.3 Driven Piles

8.4.3.1 General

Resistance factors for pile capacities shall be as specified in Table 10.5.4-2 of the AASHTO LRFD provisions, with the exception that resistance factors of 1.0 shall be used for seismic loads.

For the effect of settling ground and downdrag loads, unfactored load and resistance factors (γ = 1.0; ϕ = 1.0) shall be used, unless required otherwise by the Owner.

Batter piles shall not be used where downdrag loads are expected unless special studies are performed.

For seismic loading the groundwater table location shall be the average groundwater location, unless the Owner approves otherwise.

8.4.3.2 Design Requirements

The design of driven pile foundations shall be based column loads determined by capacity design principles (Article 4.8) or elastic seismic forces, whichever is smaller. Both the structural and geotechnical elements of the foundation shall be designed for the capacity design forces of Article 4.8.

Foundation flexibility (Article 5.3.4) shall be incorporated into design for Soil Profile Types C, D, and E, if the effects of foundation flexibility contribute more than 20 percent to the displacement of the system. For SDAP E foundations flexibility shall be included in the push-over analysis whenever it is included in the dynamic analysis.

Liquefaction shall be considered when applicable during the development of spring constants and capacity values for these seismic design and analysis procedures.

8.4.3.3 Axial and Rocking Stiffness for Driven Pile/Pile Cap Foundations (Nonliquefiable Sites)

The axial stiffness of the driven pile foundations shall be determined for design cases in which foundation flexibility is included. For many applications, the axial stiffness of a group of piles can be estimated within sufficient accuracy using the following equation:

$$K_{sv} = \Sigma\ 1.25AE/L \qquad (8.4.3.2\text{-}1)$$

where A = cross-sectional area of the pile

E = modulus of elasticity of the piles

L = length of the piles

N = number of piles in group and is represented by the summation symbol in the above equations.

The rocking spring stiffness values about each horizontal pile cap axis can be computed assuming each axial pile spring acts as a discrete Winkler spring. The rotational spring constant (i.e., moment per unit rotation) is then given by

$$K_{srv} = \Sigma\ k_{vn}\ S_n^2 \qquad (8.4.3.2\text{-}2)$$

where k_{vn} = axial stiffness of the nth pile

S_n = distance between the nth pile and the axis of rotation

The effects of group action on the determination of stiffness shall be considered if the center-to-center spacing of piles for the group in the direction of loading is closer than 3 pile diameters.

8.4.3.4 Lateral Stiffness Parameters for Driven Pile/Pile Cap Foundations (Nonliquefiable Sites)

The lateral stiffness parameters of driven pile foundations shall be estimated for design cases in which foundation flexibility is included. Lateral response of a pile foundation system depends on the stiffness of the piles and, very often, the stiffness of the pile cap. Procedures for defining

the stiffness of the pile component of the foundation system are covered in this article. Methods for introducing the pile cap stiffness are addressed in Article 8.4.3.5.

For preliminary analyses involving an estimate of the elastic displacements of the bridge, pile stiffness values can be obtained by using a series of charts prepared by Lam and Martin (1986). These charts are reproduced in Figures 8.4.3.4-1 through 8.4.3.4-6. The charts are applicable for mildly nonlinear response, where the elastic response of the pile dominates the nonlinear soil stiffness.

For push-over analyses the lateral load displacement relationship must be extended into the nonlinear range of response. It is usually necessary to use computer methods to develop the load-displacement relationship in this range, as both the nonlinearity of the pile and the soil must be considered. Programs such as LPILE (Reese et al., 1997), COM 624 (Wang and Reese, 1991), and FLPIER (Hoit and McVay, 1996) are used for this purpose. These programs use nonlinear "p-y" curves to represent the load-displacement response of the soil; they also can accommodate different types of pile-head fixity. Procedures for determining the "p-y" curves are discussed by Lam and Martin (1986) and more recently by Reese et al. (1997).

The effects of group action on lateral stiffness shall be considered if the center-to-center spacing of the piles is closer than 3 pile diameters.

8.4.3.5 Pile Cap Stiffness and Capacity

The stiffness and capacity of the pile cap shall be considered in the design of the pile foundation. The pile cap provides horizontal resistance to the shear loading in the column. Procedures for evaluating the stiffness and the capacity of the footing in shear shall follow procedures given in Article C8.4.2.2 for spread footings, except that the base shear resistance of the cap shall be neglected.

When considering a system comprised of a pile and pile cap, the stiffness of each shall be considered as two springs in parallel. The composite spring shall be developed by adding the reaction for each spring at equal displacements.

8.4.3.6 Moment and Shear Design (Nonliquefiable Sites)

The capacity of the structural elements of driven pile foundations shall be designed to resist the capacity design forces of Article 4.8 or the elastic design force within the column, whichever is smaller. Unfactored resistance ($\phi = 1.0$) shall be used in performing the geotechnical capacity check. The leading row piles during overturning shall not exceed the plunging capacity of the piles. Separation between the pile tip and the soil (i.e. gapping) shall be allowed only in the most distant row of trailing piles. Forces on all other rows of piles shall either be compressive or not exceed the nominal tension capacity of the piles. The maximum shear force on the pile(s) shall be less than the structural shear capacity of the piles.

If the plunging capacity is exceeded or gapping of other than the trailing row of piles occurs, special studies shall be conducted to show that performance of the pile system is acceptable. Special studies shall be performed only with the prior consent of the Owner and require SDAP E.

8.4.3.7 Liquefaction and Dynamic Settlement Evaluations

If liquefaction is predicted to occur at the site, effects of liquefaction on the bridge foundation shall be evaluated. This evaluation shall consider the potential for loss in lateral bearing support, flow and lateral spreading of the soil, settlement below the toe of the pile, and settlement from drag loads on the pile as excess porewater pressures in liquefied soil dissipate. Procedures given in Appendix D shall be followed when making these evaluations.

If liquefaction causes unacceptable bridge performance, consideration should be given to the use of ground improvement methods to meet design requirements. In light of the potential costs of ground improvement, the Owner shall be consulted before proceeding with a design for ground improvement to review the risks associated with liquefaction relative to the costs for remediating the liquefaction potential.

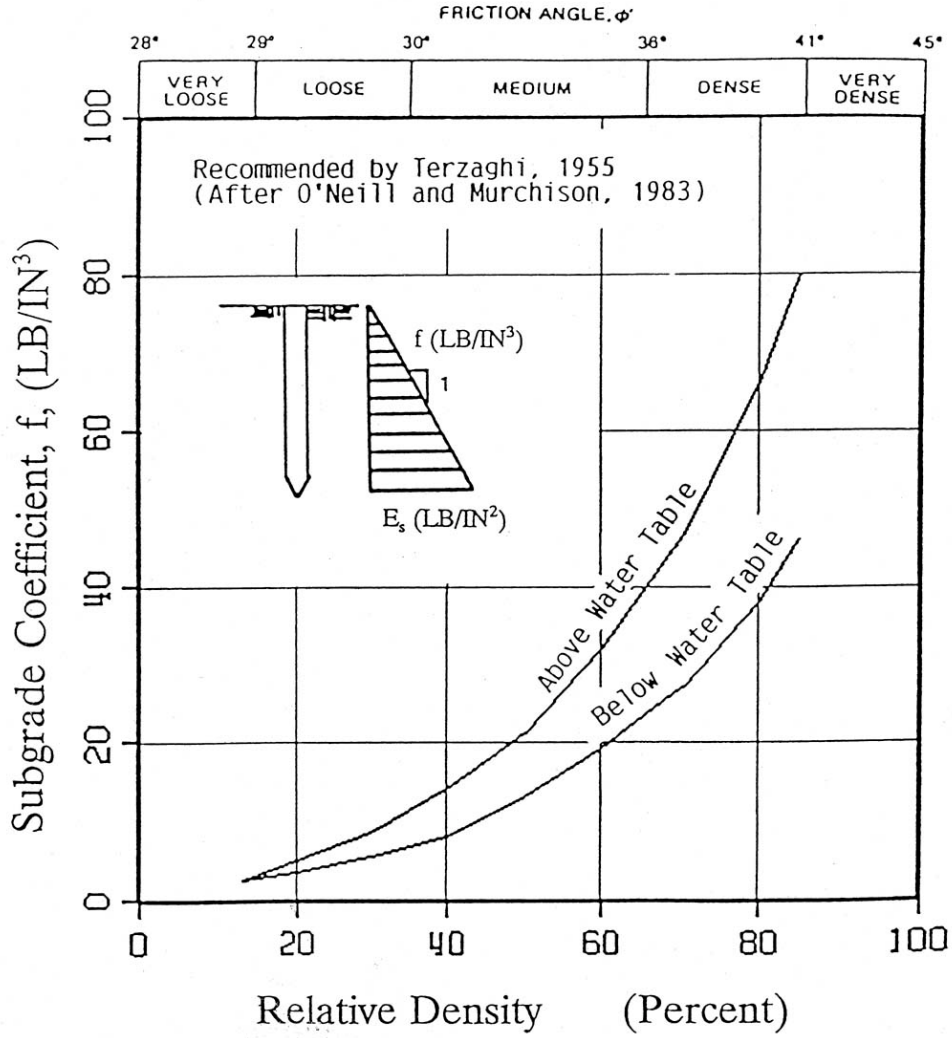


Figure 8.4.3.4-1. Recommendations for Coefficient of Variation in Subgrade Modulus with Depth for Sand (ATC, 1996)

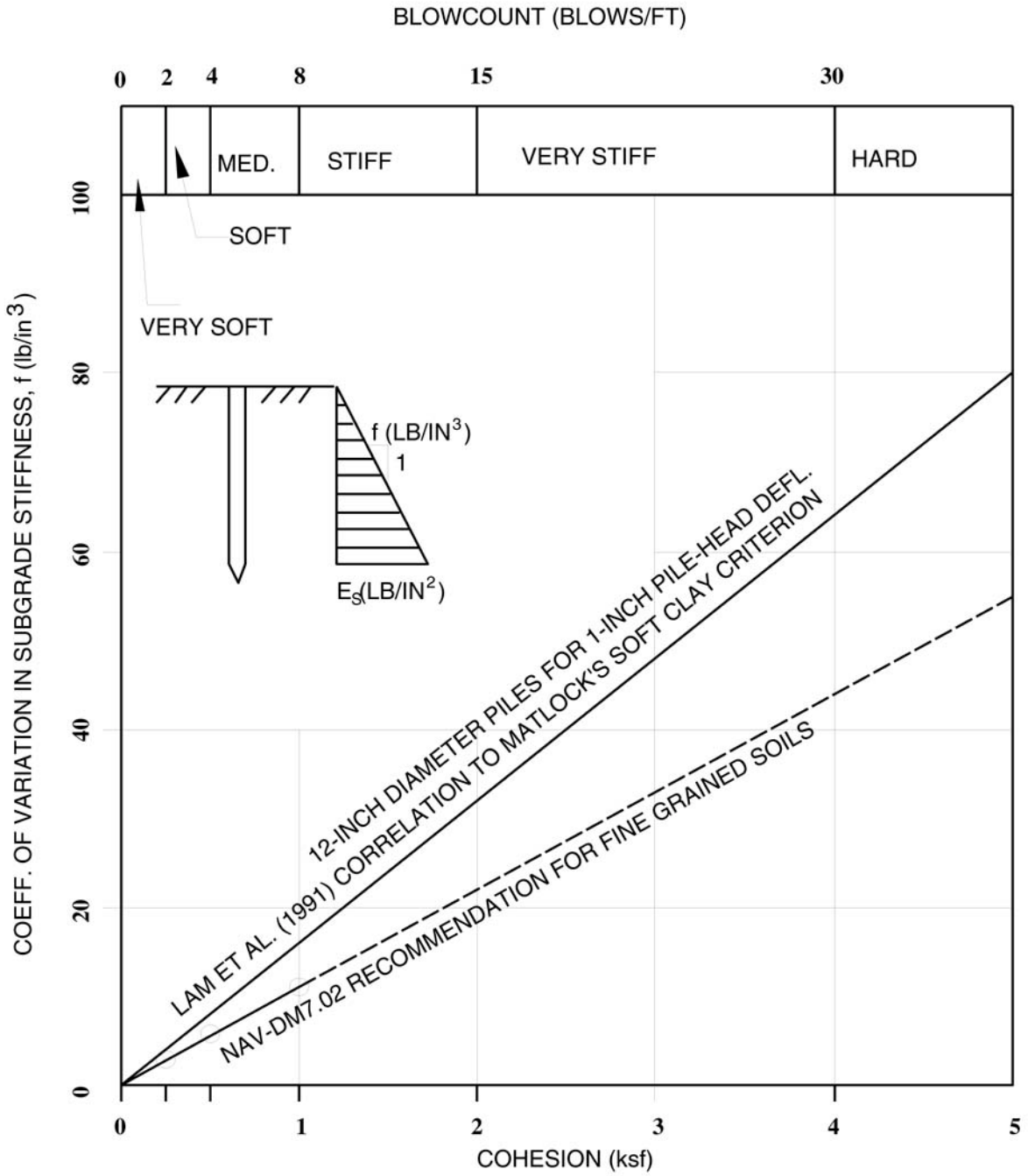


Figure 8.4.3.4-2. Recommendations for Coefficient of Variation in Subgrade Modulus with Depth for Clay (ATC, 1996)

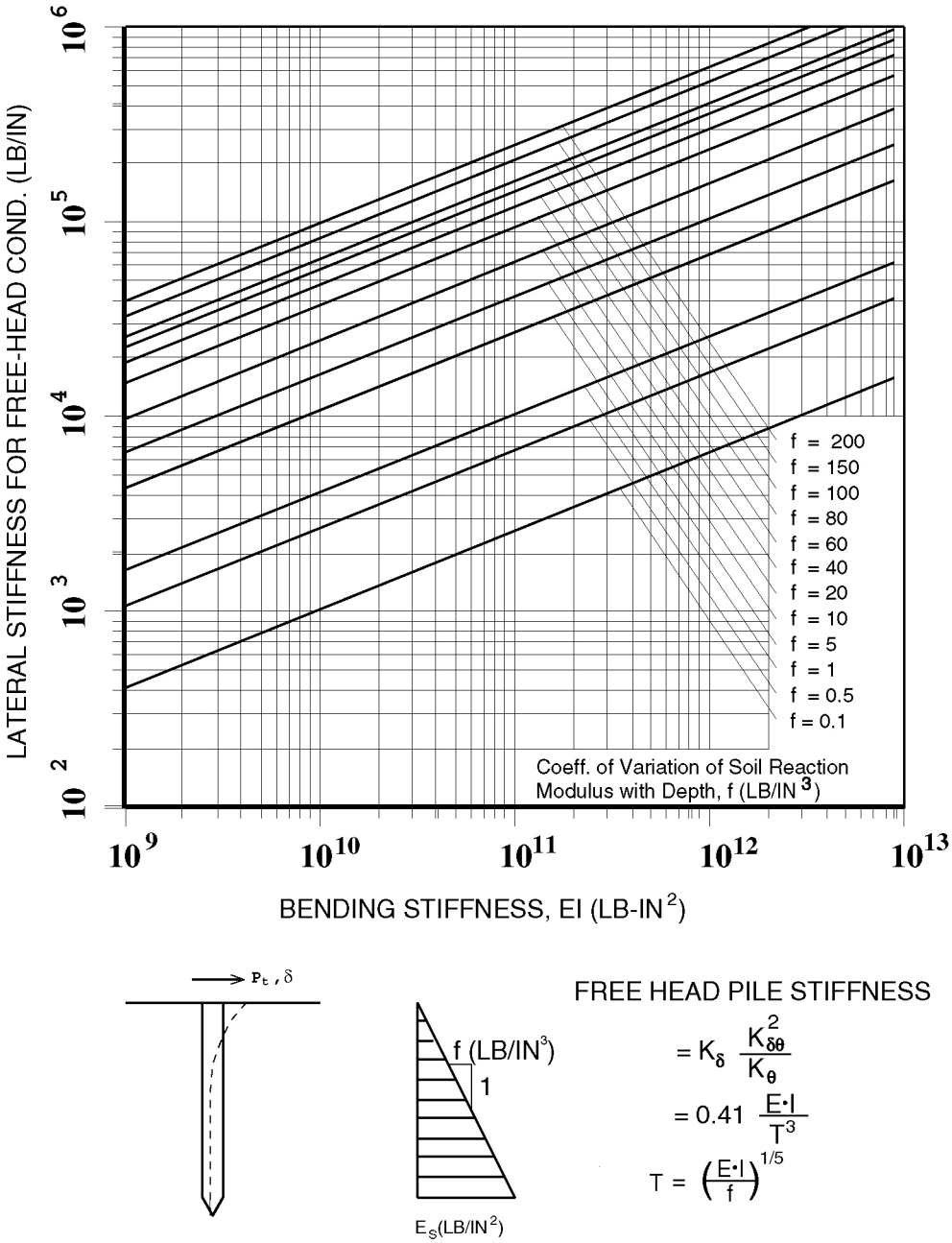


Figure 8.4.3.4-3. Coefficient of Lateral Pile Head Stiffness for Free-Head Pile Lateral Stiffness (ATC, 1996)

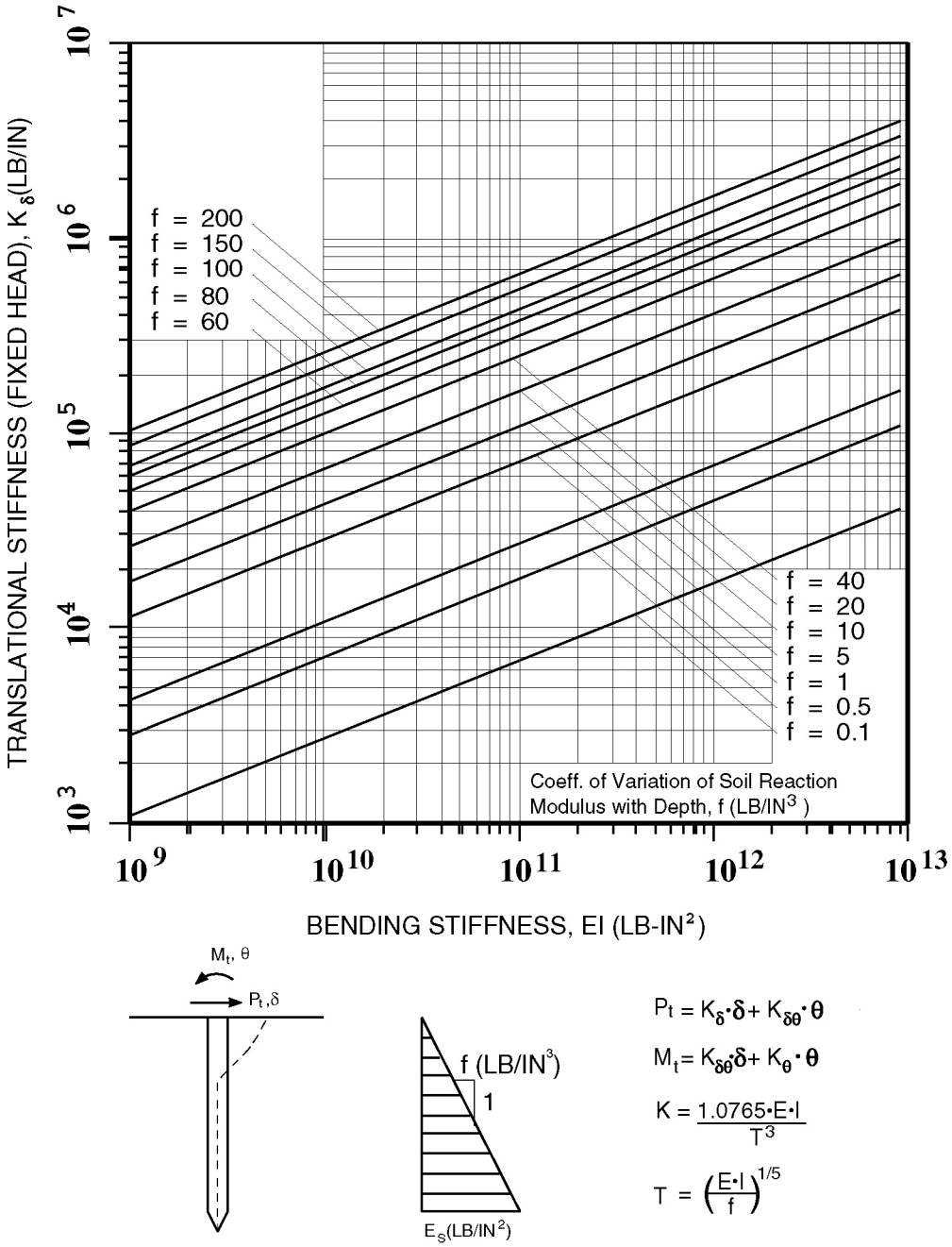


Figure 8.4.3.4-4 Coefficient for Lateral Pile-Head Stiffness for Fixed-Head Pile Lateral Stiffness (ATC, 1996)

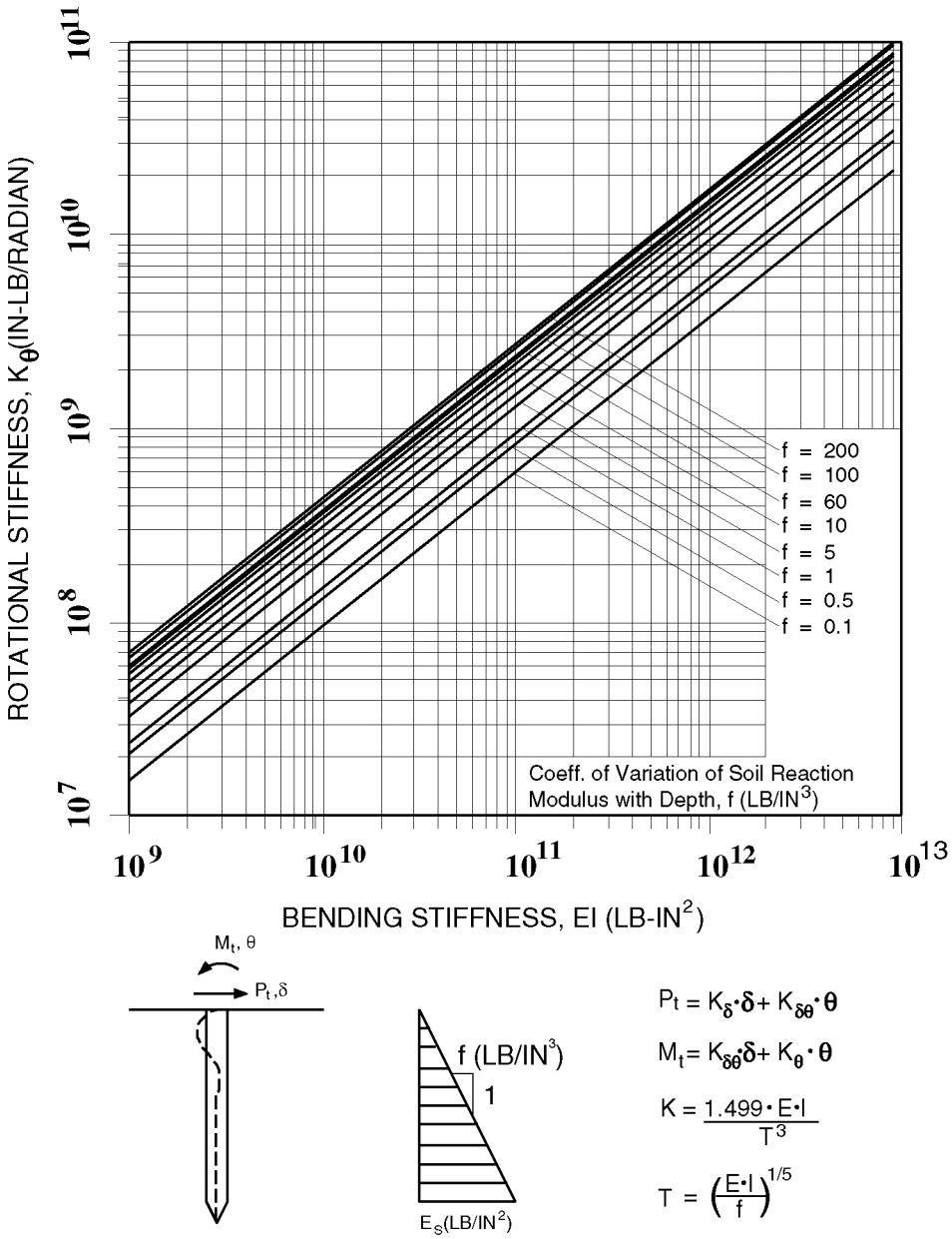


Figure 8.4.3.4-5 Coefficient for Pile Head Rotation (ATC, 1996)

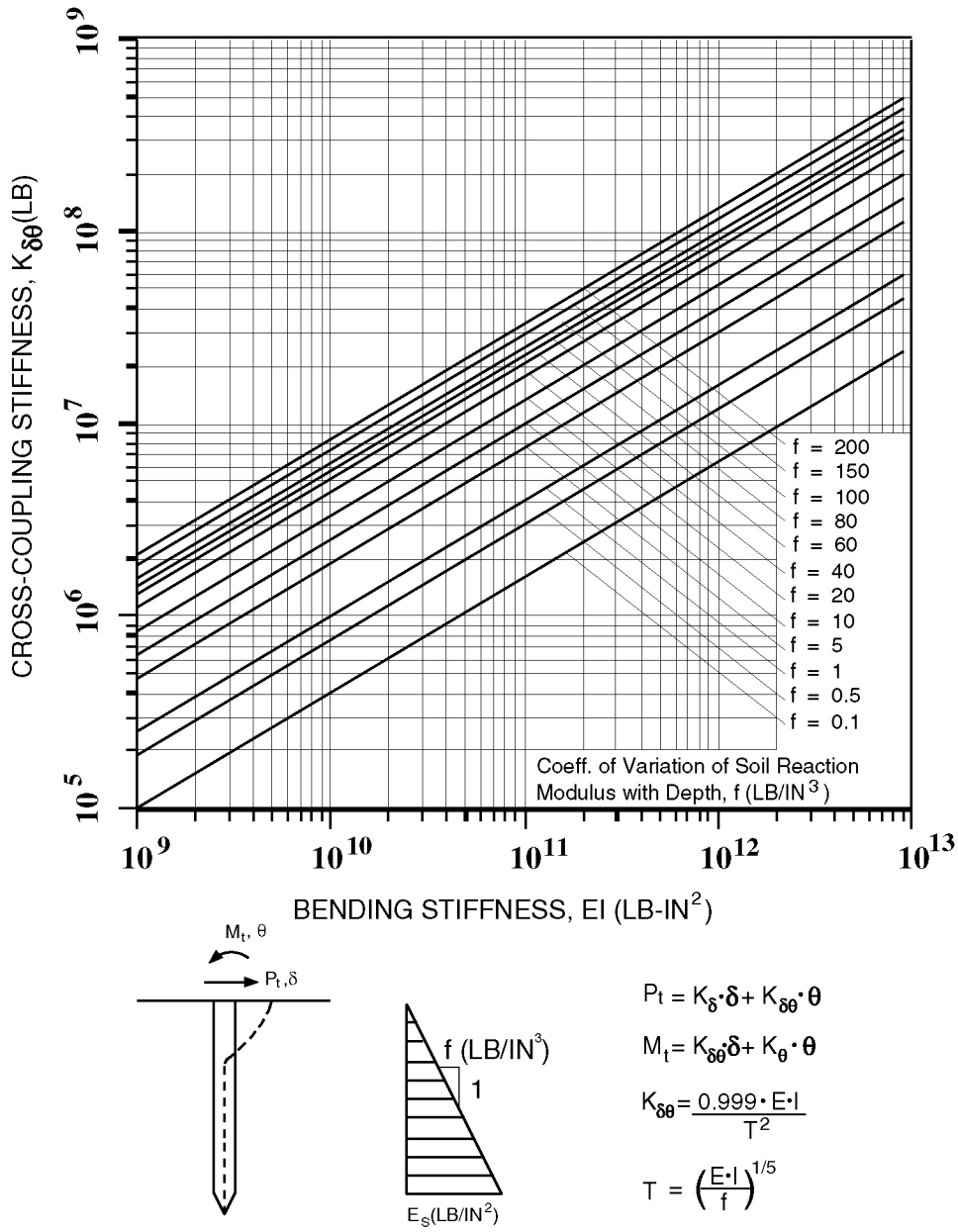


Figure 8.4.3.4-6. Coefficient for Cross-Coupling Stiffness Term (ATC, 1996)

8.4.4 Drilled Shafts

Procedures identified in Article 8.4.3, including liquefaction and dynamic settlement, generally apply with the exceptions that, (1) the ultimate capacity of single shaft foundations in compression and uplift shall not be exceeded under maximum seismic loads and (2) the flexibility of the drilled shaft shall be represented using either the estimated depth of fixity or soil springs in a lateral pile analysis.

Checks shall be conducted to confirm that minimum shaft lengths occur. The stable length can be determined by conducting nonlinear computer modeling or by using a length (L) > $\pi\lambda$ where $\lambda = [EI_p/E_s]^{0.25}$ and EI_p and E_s are the bending stiffness of the pile and the subgrade modulus of the soil, respectively.

The nonlinear properties of the shaft shall be considered in evaluating the lateral response of the pile to lateral loads during a seismic event. Diameter adjustments shall be considered during lateral analyses of shafts with a diameter greater than 600 mm if the shaft is free to rotate, as in the case of a column extension (i.e., no pile cap). Contributions from base shear shall also be considered

8.5 ABUTMENT DESIGN REQUIREMENTS

8.5.1 General

The effect of earthquakes shall be investigated using the extreme event limit state of Table 3.2-1 with resistance factors $\varphi = 1.0$. Requirements for static design should first be met, as detailed in Articles 11.6.1 through 11.6.4 of the AASHTO LRFD provisions. Selection of abutment types prior to static design shall recognize type selection criteria for seismic conditions, as described in Articles 3.3, 3.3.1, Section 4, Table 3.3.1-1 and Figure C3.3.1-4.

8.5.1.1 Abutments and 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 shall be considered in the seismic design of bridges, as outlined in these provisions. Damage to walls that is allowed to occur during earthquakes shall be consistent with the performance criteria. Abutment participation in the overall dynamic response of the bridge systems 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 (i.e., 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 based on an applicable passive earth-pressure theory.

8.5.2 Longitudinal Direction

Under earthquake loading, the earth pressure action on abutment walls changes from a static condition to one of generally two possible conditions, depending on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge/abutment configuration. For seat-type abutments where 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 would 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 passive pressure condition behind the backwall.

For stub or integral abutments, the abutment stiffness and capacity under passive pressure loading, are primary design concerns, as discussed in Articles 8.5.2.1 and 8.5.2.2. However, for partial depth or full depth seat abutment walls, earthquake-induced active earth pressures will continue to act below the backwall following separation of a knock-off backwall. These active pressures need to be considered in evaluating wall stability.

8.5.2.1 SDAP B and C

Abutments designed for service load conditions in these categories should resist earthquake loads with minimal damage with the exception of bridges in Seismic Hazard Level IV using SDAP C. For seat-type abutments, minimal abutment movement could be expected under dynamic active pressure conditions. However, bridge superstructure displacement demands could be 100 mm or more and potentially impact the abutment backwall. Where expected displacement demands are greater than a normal expansion gap of 25 to 50 mm, a knock-off backwall detail is recommended to minimize foundation damage, or alternatively, a cantilever deck slab to extend the seat gap should be provided, with a knock-off backwall tip.

In the case of integral abutments, sufficient reinforcing should be provided in the diaphragm to accommodate higher lateral pressures. For spread footing foundations, knock-off tabs or other fuse elements should be provided to minimize foundation damage. For pile-supported foundations, fuse elements should be used or connection detailing should ensure increased moment ductility in the piles.

8.5.2.2 SDAP D and E

For these design categories passive pressure resistance in soils behind integral abutment walls and knock-off walls for seat abutments will usually be mobilized due to the large longitudinal superstructure displacements associated with the inertial loads. For design purposes static passive pressures may be used without potential reductions associated with inertial loading in abutment backfill. Inclusion of abutment stiffness and capacity in bridge response analyses will reduce ductility demands on bridge columns as discussed in Article C3.3.

Case 1: To ensure that the columns are always able to resist the lateral loads, designers may choose to assume zero stiffness and capacity of abutments. In this case designers should check abutment damage potential and performance due to abutment displacement demand. Knock-off backwall details for seat abutments should be utilized to protect abutment foundations and increased reinforcing used in diaphragms or integral abutments to accommodate passive pressures.

Case 2: Where abutment stiffness and capacity is included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally adapted for static service load design, as illustrated schematically in Figure 8.5.2.2-1. Whether presumptive or computed passive pressures are used for design as described in the commentary paragraphs, backfill in this zone should be controlled by specifications unless the passive pressure that is used in less than 70% of the presumptive value.

Abutment stiffness and passive pressure capacity for either (1) SDAP D or (2) SDAP E two-step analysis methods should be characterized by a bi-linear relationship as shown in Figure 8.5.2.2-2. For seat type abutments, knock-off backwall details should be utilized with superstructure diaphragms designed to accommodate passive pressures, as illustrated in Figure C3.3.1-4. For integral abutments the end diaphragm should be designed for passive pressures, and utilize a stub pile footing or normal footing for support, with a sliding seat. Passive pressures may be assumed uniformly distributed over the height (H) of the backwall or diaphragm. Thus the total passive force is:

$$P_p = p_p^* H \qquad (8.5.2.2\text{-}1)$$

where:

H = wall height in meters

p_p = passive pressure behind backwall

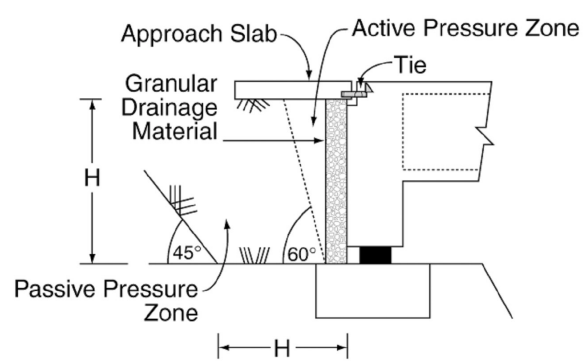


Figure 8.5.2.2-1 Design Passive Pressure Zone

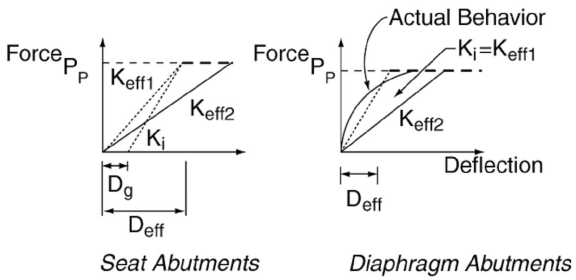


Figure 8.5.2.2-2 Characterization of Abutment Capacity and Stiffness

Calculation of Best-Estimate Passive Force 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 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 8.5.2.2-1. Therefore the properties of backfill that is only placed adjacent to the wall in the active pressure zone may not be appropriate.

If presumptive passive pressures are to be used for design, then the following criteria should apply:

(1) Soil in the "passive pressure zone" should be compacted to a dry density greater than 95 percent of the maximum per ASTM Standard Method D1557 or equivalent.

(2) For cohesionless, non-plastic backfill (fines content less than 30 percent), the passive pressure p_p may be assumed equal to H/10 MPa per meter of length of wall (2H/3 ksf per foot length of wall).

(3) For cohesive backfill (clay fraction > 15 percent), the passive pressure p_p may be assumed equal to 0.25 MPa (5 ksf) provided the estimated unconfined compressive strength is greater than 0.20 MPa (4 ksf).

The presumptive values given above apply for use in the " *Permissible with Owner's Approva*l" category, as defined in Article 3.3.1. If the design is based upon presumptive resistances that are no larger than 70 percent of the values listed above, then the structure may be classified in the "*Permissible"* category.

In all cases granular drainage material must be placed behind the abutment wall to ensure adequate mobilization of wall friction.

Calculation of Stiffness

For SDAP D one-step analyses and for the demand calculation of SDAP E analyses, an equivalent linear secant stiffness, K_{effn}, is required for analyses. For integral or diaphragm abutments, an initial secant stiffness (Figure 8.5.2.2-2) may be calculated as follows:

$$K_{effI} = P_p/0.02H \qquad (8.5.2.2\text{-}2)$$

If computed abutment forces exceed the capacity, the stiffness should be softened iteratively (K_{eff2} to K_{effn}) until abutment displacements are consistent (within 30 percent) with the assumed stiffness. For seat abutments the expansion gap should be included in the initial estimate of the secant stiffness. Thus:

$$K_{effI} = P_p/(0.02H + D_g) \qquad (8.5.2.2\text{-}3)$$

where:

D_g = gap width

For SDAP E two-step analyses, where push-over analyses are conducted, values of P_p and the initial estimate of K_{effI} should be used to define a bilinear load-displacement behavior of the abutment for the capacity assessment.

For partial depth or full-depth seat abutment walls, where knock-off backwalls are activated, the remaining lower wall design and stability check under the action of continuing earthquake-induced active earth pressures should be evaluated. For a no-collapse performance criteria, and assuming conventional cantilever retaining wall construction, horizontal wall translation under dynamic active pressure loading is acceptable. However, rotational instability may lead to collapse and thus must be prevented.

The design approach is similar to that of a free-standing retaining wall, except that lateral force from the bridge superstructure needs to be included in equilibrium evaluations, as the superstructure moves outwards from the wall. Earthquake-induced active earth pressures should be computed using horizontal accelerations at least equal to 50 percent of the site peak ground acceleration (i.e., F_aS_s / 5.0). Using less than the expected site acceleration implies that limited sliding of the wall may occur during the earthquake. A limiting equilibrium condition should be checked in the horizontal direction. To ensure safety against potential overturning about the toe, a restoring moment of at least 50 percent more than the driving overturning moment should exist. If necessary, wall design (initially based on a static loading condition) should be modified to meet the above condition.

8.5.3 Transverse Direction

In general, abutments shall be designed to resist earthquake forces in the transverse direction elastically for the 50% PE in 75-year earthquake. For the 3% PE in 75-year/1.5 mean deterministic event, the abutment may either be designed to resist transverse forces elastically or a fuse shall be provided to limit the transverse force transfer at the abutment. 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 for the various Seismic Design and Analysis Procedures are listed below.

In the context of these provisions, elastic resistance includes the use of elastomeric, sliding, or isolation bearings designed to accommodate the design displacements, soil frictional resistance acting against the base of a spread footing-supported abutment, 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 in the rare earthquake; or piles that develop a complete plastic mechanism. Article 3.3.1 outlines those mechanisms that are permissible with the Owner's approval.

The stiffness of abutments under transverse loading may be calculated based on the procedures given in Article 8.4 for foundation stiffnesses. Where fusing elements are used, allowance shall be made for the reduced stiffness of the abutment after fusing occurs.

8.5.3.1 SDAP B and C

Connection design forces also apply to shear restraint elements such as shear keys.

8.5.3.2 SDAP D and E

For structures in these categories, 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.

For short, continuous superstructure bridges (Length/Width < 4) with low skew angles (<20 degrees), low plan curvature (subtended angle < 30 degrees), and which also are designed for sustained soil mobilization in the transverse direction, the elastic forces and displacements for the transverse earthquake design may be reduced by 1.4 to account for increased damping provided by the soil at the abutments. Herein transverse earthquake is defined as acting perpendicular to a chord extending between the two abutments. Sustained soil mobilization requires resistance to be present throughout the range of cyclic motion. Where combined mechanisms provide resistance, at least 50 percent of the total resistance must be provided by a sustained mechanism for the system to qualify for the 1.4 reduction.

The design of concrete shear keys should consider the unequal forces that may develop in a skewed abutment, particularly if the intermediate piers are also skewed. (This effect is amplified if intermediate piers also have unequal stiffness, such as wall piers.) The shear key design should also consider unequal loading if multiple shear keys are used. The use of recessed or hidden shear

keys should be avoided if possible, since these are difficult to inspect and repair.

8.6 LIQUEFACTION DESIGN REQUIREMENTS

8.6.1 General

An evaluation of the potential for and consequences of liquefaction within near-surface soil shall be made in accordance with the following requirements: A liquefaction assessment is required unless one of the following conditions is met or as directed otherwise by the Owner.

- Mean magnitude for the 3% PE in 75-year event is less than 6.0 (Figures 8.6.1-1 to 8.6.1-4);
- Mean magnitude of the 3% PE in 75-year event is less than 6.4 and equal to or greater than 6.0, and the normalized Standard Penetration Test (SPT) blow count [$(N_1)_{60}$] is greater than 20;
- Mean magnitude for the 3% PE in 75-year event is less than 6.4 and equal to or greater than 6.0, $(N_1)_{60}$ is greater than 15, and F_aS_s is between 0.25 and 0.375; or

If the mean magnitude shown in Figures 8.6.1-1 to 8.6.1-4 is greater than or equal to 6.4, or if the above requirements are not met for magnitudes between 6.0 and 6.4 or if for the 50% PE in 75 year event, F_aS_s is greater than 0.375, evaluations of liquefaction and associated phenomena such as lateral flow, lateral spreading, and dynamic settlement shall be evaluated in accordance with these Specifications.

8.6.2 Evaluation of Liquefaction Potential

Procedures given in Appendix D shall be used to evaluate the potential for liquefaction.

8.6.3 Evaluation of the Effects of Liquefaction and Lateral Ground Movement

Procedures given in Appendix D shall be used to evaluate the potential for and effects of liquefaction and liquefaction-related permanent ground movement (i.e., lateral spreading, lateral flow, and dynamic settlement). If both liquefaction and ground movement occur, they shall be treated as separate and independent load cases, unless agreed to or directed otherwise by the Owner.

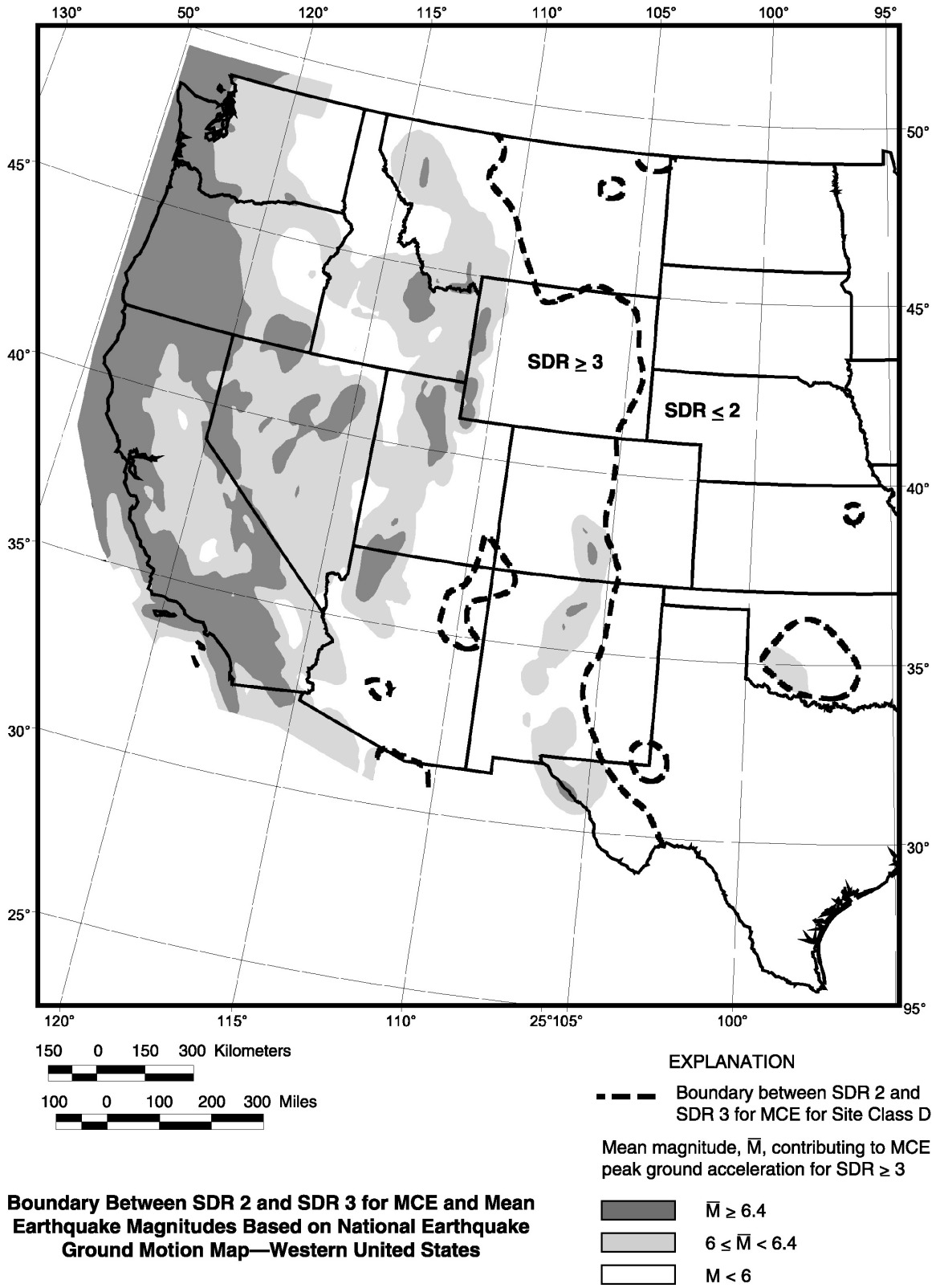
8.6.4 Design Requirements if Liquefaction and Ground Movement Occurs

If it is determined from Appendix D that liquefaction can occur at a bridge site, then one or more of the following approaches shall be implemented in the design.

Bridges shall be supported on deep foundations unless (1) the footing is located below the liquefiable layer, (2) special design studies are conducted to demonstrate that the footing will tolerate liquefaction, or (3) the ground is improved so that liquefaction does not occur. If spread footings are being considered for use at a liquefiable site, Owner approval shall be obtained before beginning the design process.

If liquefaction occurs, then the bridge shall be designed and analyzed in two configurations as follows:

1. Nonliquefied Configuration: The structure shall be analyzed and designed, assuming no liquefaction occurs using the ground response spectrum appropriate for the site soil conditions.
2. Liquefied Configuration: The structure as designed in Nonliquefied Configuration above shall be reanalyzed and redesigned, if necessary, assuming that the layer has liquefied and the liquefied soil provides whatever residual resistance is appropriate (i.e., “p-y curves” or modulus of subgrade reaction values for lateral pile response analyses consistent with liquefied soil conditions). The design spectra shall be the same as that used in Nonliquefied Configuration unless a site-specific response spectra has been developed using nonlinear, effective stress methods (e.g., computer program DESRA or equivalent) that properly account for the buildup in pore-water pressure and stiffness degradation in liquefiable layers. The reduced response spectra resulting from the site-specific nonlinear, effective stress analyses shall not



Boundary Between SDR 2 and SDR 3 for MCE and Mean Earthquake Magnitudes Based on National Earthquake Ground Motion Map—Western United States

Figure 8.6.1-1 Mean Earthquake Magnitude Map for Western United States

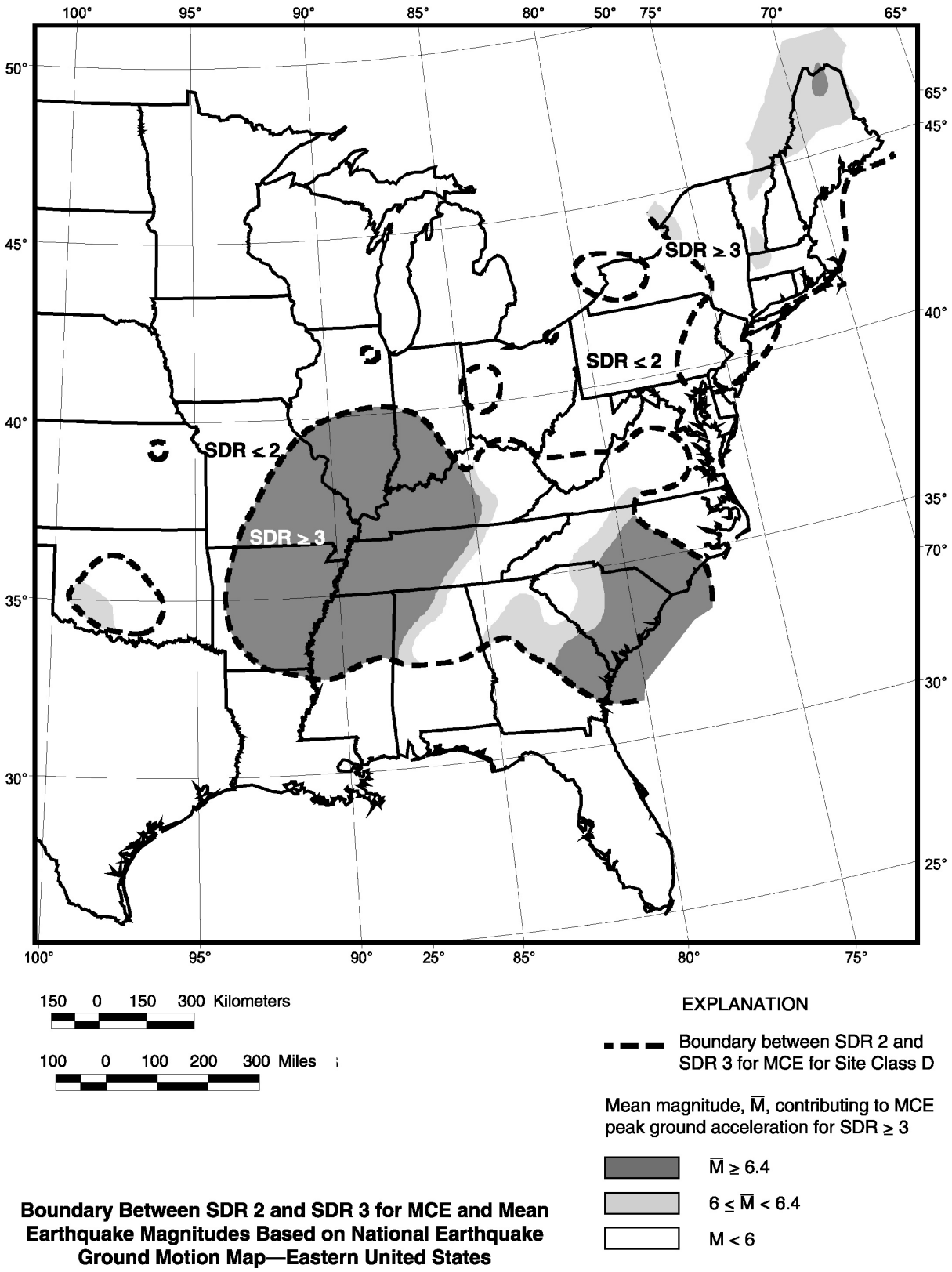


Figure 8.6.1-2 Mean Earthquake Magnitude Map for Eastern United States

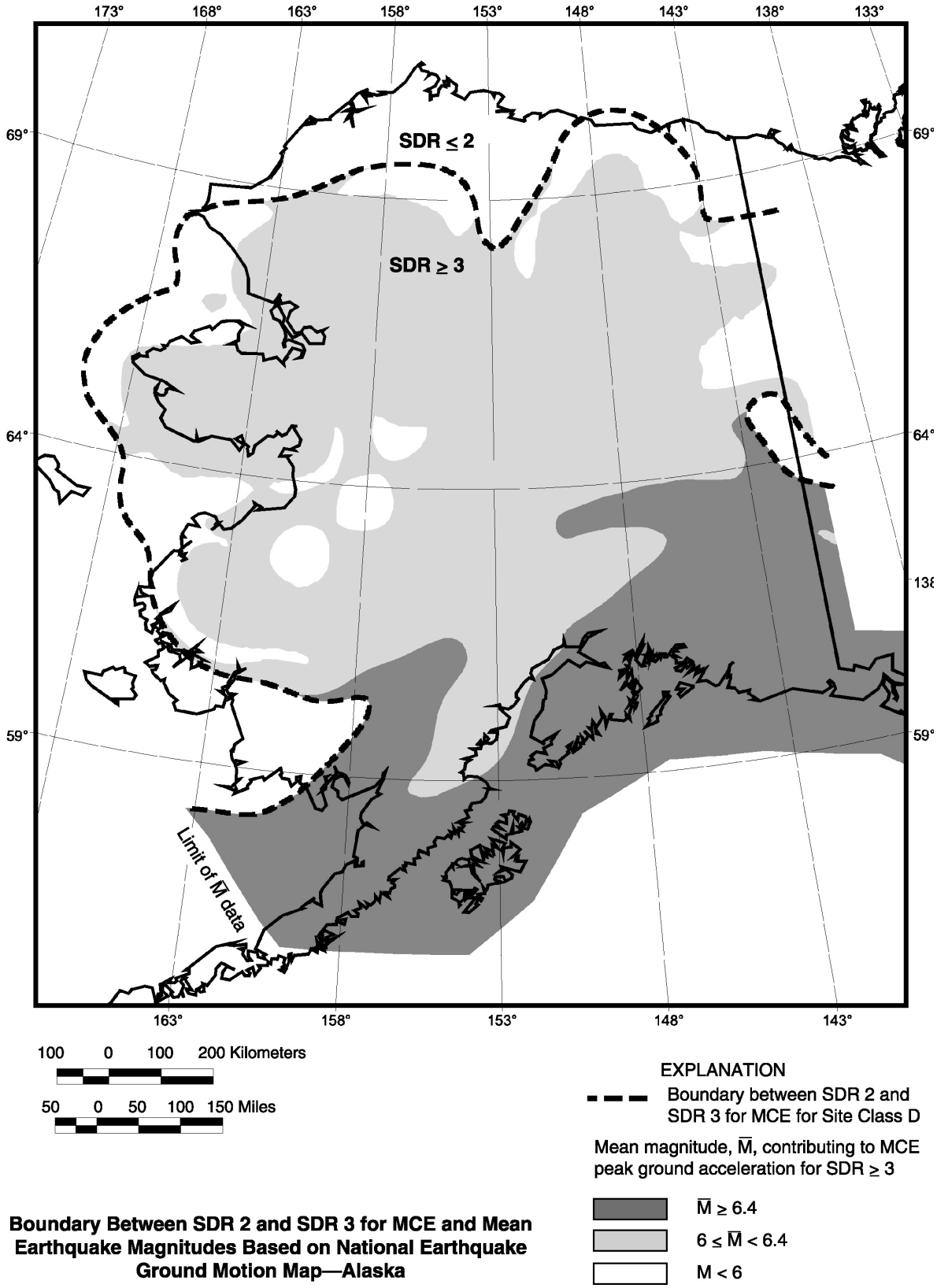
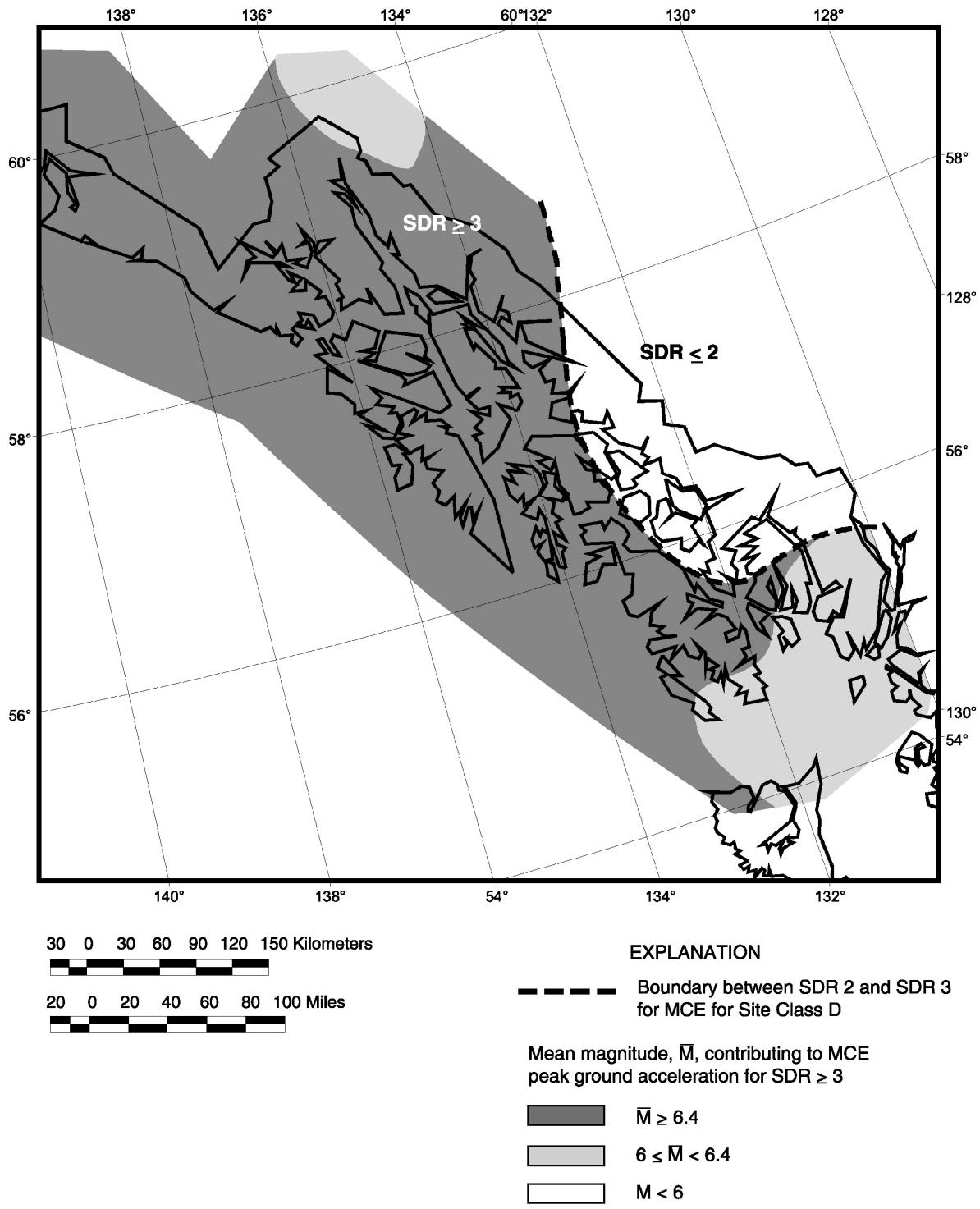


Figure 8.6.1-3 Mean Earthquake Magnitude Map for Alaska



Boundary Between SDR 2 and SDR 3 for MCE and Mean Earthquake Magnitudes Based on National Earthquake Ground Motion Map—Southeast Alaska

Figure 8.6.1-4 Mean Earthquake Magnitude Map for Southeast Alaska

be less than 2/3's of that used in Nonliquefied Configuration. The Designer shall provide a drawing of the load path and energy dissipation mechanisms in this condition as required by Article 3.3 since it is likely that plastic hinges will occur in different locations than for the non-liquefied case. Shear reinforcement given in Article 8.8.2.3 shall be used in all concrete and prestressed concrete piles to a depth of 3 pile diameters below the liquefied layer.

If lateral flow or lateral spreading occurs, the following options shall be considered.

1. Design the piles to resist the forces generated by the lateral spreading.
2. If the structure cannot be designed to resist the forces, assess whether the structure is able to tolerate the anticipated movements and meet the geometric and structural constraints of Table C3.2-1. The maximum plastic rotation of the piles is 0.05 radians as per Article 8.7.9 and 8.8.6.
3. If the structure cannot meet the performance requirements of Table 3.2-1, assess the costs and benefits of various mitigation measures to minimize the movements to a tolerable level to meet the desired performance objective. If a higher performance is desired so that the piles will not have to be replaced, the allowable plastic rotations in-ground hinges of Article 8.7.9.2 and 8.8.6.2 shall be met.

8.6.5 Detailed Foundation Design Requirements

Article 8.4 contains detailed design requirements for each of the different foundation types.

8.6.6 Other Collateral Hazards

The potential occurrence of collateral hazards resulting from fault rupture, landsliding, differential ground compaction, and flooding and inundation shall be evaluated. Procedures for making these evaluations are summarized in Appendix D.

8.7 STRUCTURAL STEEL DESIGN REQUIREMENTS

8.7.1 General

The provisions of this article shall apply only to a limited number of specially detailed steel components designed to dissipate hysteretic energy during earthquakes. This article does not apply to steel members that are designed to remain elastic during earthquakes.

For the few specially designed steel members that are within the scope of this article, the other requirements of Section 6 of the LRFD provisions are also applicable (unless superseded by more stringent requirements in this article).

Continuous and clear load path or load paths shall be assured. Proper load transfer shall be considered in designing foundations, substructures, superstructures and connections.

Welds shall be designed as capacity protected elements. Partial penetration groove welds shall not be used in ductile substructures.

Abrupt changes in cross sections of members in ductile substructures are not permitted within the plastic hinge zones defined in Article 4.9 unless demonstrated acceptable by analysis and supported by research results.

8.7.2 Materials

Ductile substructure elements and ductile end-diaphragms, as defined in Articles 8.7.4 through 8.7.8, shall be made of either:

(a) M270 (ASTM 709M) Grade 345 and Grade 345W steels

(b) ASTM A992 steel, or

(c) A500 Grade B or A501 steels (if structural tubing or pipe).

Other steels may be used provided that they are comparable to the approved Grade 345 steels.

Nominal resistance is defined as the resistance of a member, connection or structure based on the expected yield strength (F_{ye}), 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.

Overstrength capacity is defined as the resistance of a member, connection or structure based on the nominal dimensions and details of the

final section(s) chosen, calculated accounting for the expected development of large strains and associated stresses larger than the minimum specified yield values.

The expected yield strength shall be used in the calculation of nominal resistances, where expected yield strength is defined as $F_{ye} = R_y F_y$ where R_y shall be taken as 1.1 for the permitted steels listed above.

Welding requirements shall be compatible with AWS/AASHTO D1.5-96 Structural Bridge Welding Code. However, under-matched 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 A709/A709M Supplementary Requirement S84 (Fracture Critical). Welds metal connecting these members shall meet the toughness requirements specified in the AWS D1.5 Bridge Specification for Zone III.

8.7.3 Sway Stability Effects

The sway effects produced by the vertical loads acting on the structure in its displaced configuration shall be determined from a second-order analysis. Alternatively, recognized approximate methods for P-Δ analysis, or the provisions in Article 8.3.4, can be used.

8.7.4 Ductile Moment Resisting Frames and Single Column Structures

This article applies to ductile moment-resisting frames and bents, constructed with I-shape beams and columns connected with their webs in a common plane. Except as noted in Article 8.7.4-1, columns shall be designed as ductile structural elements, while the beams, the panel zone at column-beam intersections and the connections shall be designed as Capacity Protected Elements.

8.7.4.1 Columns

Width-to-thickness ratios of compression elements of columns shall be in compliance with Table 8.7.4-2. Full penetration flange and web welds are required 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 provisions. 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 6.10.7 of the AASHTO LRFD provisions.

The potential plastic hinge zones (Article 4.9), near the top and base of each column, shall be laterally supported and the unsupported distance from these locations shall not exceed $17250 r_y / F_y$. These lateral supports shall be provided either directly to the flanges or indirectly through a column web stiffener or a continuity plate. Each column flange lateral support shall resist a force of not less than 2% of the nominal column flange strength (btF_y) at the support location. The possibility of complete load reversal shall be considered.

When no lateral support can be provided, the column maximum slenderness 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 away from the plastic hinge zones as defined in Article 4.9 at a minimum distance equal to the greater of:

(a) one-fourth the clear height of column;
(b) twice the column depth; and
(c) one meter (39 inches).

8.7.4.2 Beams

The factored resistance of the beams shall be determined in accordance with Article 6.10.2 of the LRFD provisions. 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. The probable flexural resistance of columns shall be taken as the product of the overstrength factor (defined in Article 4.8) times the columns nominal flexural resistance determined either in accordance to Article 6.9.2.2 of the AASHTO LRFD provisions, or by

Table 8.7.4-1 Limiting Width-to-Thickness Ratios

Description of element	Width-to-thickness ratio (b/t)[1]	Limiting width-to-thickness ratio λ_p[2]	Limiting width-to-thickness ratio k[3]
Flanges of I-shaped sections and channels in compression	$\frac{b_f}{2t_f}$	$\frac{135}{\sqrt{F_y}}$	0.30
Webs in combined flexural and axial compression	$\frac{h_c}{t_w}$	**For** $\frac{P_u}{\Phi_b P_y} \le 0.125$ $\frac{1365}{\sqrt{F_y}}\left(1-\frac{1.54P_u}{\Phi_b P_y}\right)$ **For** $\frac{P_u}{\Phi_b P_y} > 0.125$ $\frac{500}{\sqrt{F_y}}\left(2.33-\frac{P_u}{\Phi_b P_y}\right) \ge \frac{665}{\sqrt{F_y}}$	**For** $\frac{P_u}{\Phi_b P_y} \le 0.125$ $3.05\left(1-\frac{1.54P_u}{\Phi_b P_y}\right)$ **For** $\frac{P_u}{\Phi_b P_y} > 0.125$ $1.12\left(2.33-\frac{P_u}{\Phi_b P_y}\right) \ge 1.48$
Hollow circular sections (pipes)	$\frac{D}{t}$	$\frac{8950}{F_y}$	$\frac{200}{\sqrt{F_y}}$
Unstiffened rectangular tubes	$\frac{b}{t}$	$\frac{300}{\sqrt{F_y}}$	0.67
Legs of angles	$\frac{b}{t}$	$\frac{145}{\sqrt{F_y}}$	0.32

1. Width-to-thickness ratios of compression elements – Note that these are more stringent for members designed to dissipate hysteretic energy during earthquake than for other members (Article 6.9.4.2).

2. Limits expressed in format to satisfy the requirement $\frac{b}{t} \le \lambda_p$

3. Limits expressed in format to satisfy the requirement $\frac{b}{t} \le k\sqrt{\frac{E}{F_y}}$

4. Note: In the above, b_f and t_f are respectively the width and thickness of an I-shaped section, h_c is the depth of that section and t_w is the thickness of its web.

$$M_{nx} = 1.18 M_{px}\left[1 - \frac{P_u}{AF_{ye}}\right] \le M_{px} \qquad (8.7.4.1\text{-}1)$$

unless demonstrated otherwise by rational analysis, and where M_{px} is the column plastic moment under pure bending calculated using F_{ye}.

8.7.4.3 Panel Zones and Connections

Column-beam intersection panel zones, moment resisting connections and column base connections shall be designed as Capacity Protected Elements.

Panel zones shall be designed such that the vertical shearing resistance is determined in accordance with Article 6.10.7.2 of the AASHTO LRFD provisions.

Beam-to-column connections shall have resistance not less than the resistance of the beam stipulated in Article 8.7.4.2.

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. Their b/t shall meet the limits for projecting elements of Article 6.9.4.2 of the AASHTO LRFD provisions. These continuity plates shall be proportioned to meet the stiffener requirements stipulated in Article 6.10.8.2 of the AASHTO LRFD provisions and shall be connected to both flanges and the web.

Flanges and connection plates in bolted connections shall have a factored net section ultimate resistance calculated by Equation 6.8.2.1-2, at least equal to the factored gross area yield resistance given by Equation 6.8.2.1-1, with A_g and A_n in Article 6.8.2.1 taken here as the area of the flanges and connection plates in tension. These referenced equations and article are from the AASHTO LRFD provisions.

8.7.4.4 Multi-tier Frame Bents

For multi-tier frame bents, capacity design principles as well as the requirements of Article 8.7.4.1 may be modified by the engineer to achieve column plastic hinging only at the top and base of the column, and plastic hinging at the ends of all intermediate beams. Column plastic hinging shall not be forced at all joints at every tier.

8.7.5 Ductile Concentrically Braced Frames

Braces are the Ductile Substructure Elements in ductile concentrically braced frames.

8.7.5.1 Bracing Systems

Diagonal braces shall be oriented such that a nearly identical ultimate strength is achieved in both sway directions, when considering only the strength contribution of braces in tension. To achieve this, it is required that, at any level in any planar frame, the sum of the horizontal components of the strength of the braces in tension when the frame sway in one direction, shall be within 30% of the same value for sway in the other direction.

Article 8.7.5 is only applicable to braced frames for which all braces' action lines meet at beam-to-column intersection points (such as X-braces).

8.7.5.2 Design Requirements for Ductile Bracing Members

Bracing members shall have a slenderness ratio, KL/r, less than $2600/\sqrt{F_y}$ or Article 6.9.3 of the AASHTO LRFD Provisions.

The width-to-thickness ratios of bracing members should be limited as indicated in Table 8.7.4-1. For back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio shall not exceed $200/\sqrt{F_y}$ rather than the limit of Table 8.7.4-1

In built-up bracing members, the slenderness ratio of the individual parts between stitches shall be not greater than 0.4 times the slenderness ratio of the member as a whole. When it can be shown that braces will buckle without causing shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio of the individual parts does not exceed 0.75 times the slenderness ratio of the built-up member.

8.7.5.3 Brace Connections

The controlling overstrength capacity shall be taken as the axial tensile yield strength of the

brace (A_gF_{ye}). Brace connections shall be designed as Capacity Protected Elements.

Connections must be designed to ensure that the bracing member is capable of yielding the gross section. Consequently, brace strength calculated based on tension rupture on the effective net section and block shear rupture, shall be greater that the design tensile strength of brace given by gross section yielding.

Eccentricities in bracing connections shall be minimized.

Brace connections including gusset plates shall be detailed to avoid brittle failures due to rotation of the brace when it buckles. This ductile rotational behavior shall be allowed for, either in the plane of the frame or out of it, depending on the slenderness ratios.

The design of gusset plates shall also include consideration of buckling.

Stitches that connect the separate elements of built-up bracing members shall, if the overall buckling mode induces shear in the stitches, have a strength at least equal to the design tensile strength of each element. The spacing of stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

8.7.5.4 Columns, Beams and Other Connections

Columns, beams, beam-to-column connections and column splices that participate in the lateral-load-resisting system shall be designed as Capacity Protected Elements with the following additional requirements:

(a) Columns, beams and connections shall resist forces arising from load redistribution following brace buckling or yielding. The brace compressive resistance shall be taken as 0.3 $\phi_c P_n$ if this creates a more critical condition.

(b) Column splices made with partial penetration groove welds and subject to net tension forces due to overturning effects shall have Factored Resistances not less than 50% of the flange yield load of the smaller member at the splice.

8.7.6 Concentrically Braced Frames with Nominal Ductility

Braces are the Ductile Substructure Elements in nominally ductile concentrically braced frames.

8.7.6.1 Bracing Systems

Diagonal braces shall be oriented such that a nearly identical ultimate strength is achieved in both sway directions, when considering only the strength contribution of braces in tension. To achieve this, it is required that, at any level in any planar frame, the sum of the horizontal components of the strength of the braces in tension when the frame sway in one direction, shall be within 30% of the same value for sway in the other direction.

The categories of bracing systems permitted by this Article includes:

(a) tension-only diagonal bracing,
(b) chevron bracing (or V-bracing) and,
(c) direct tension-compression diagonal bracing systems of the geometry permitted in Article 8.7.5.1, but that do not satisfy all the requirements for ductile concentrically braced frames.

Tension-only bracing systems in which braces are connected at beam-to-column intersections are permitted in bents for which every column is fully continuous over the entire bent height, and where no more than 4 vertical levels of bracing are used along the bent height.

8.7.6.2 Design Requirements for Nominally Ductile Bracing Members

Bracing members shall have a slenderness ratio, KL/r, less than $3750/\sqrt{F_y}$ or Article 6.9.3 of the AASHTO LRFD provisions. This limit is waived for members designed as tension-only bracing.

In built-up bracing members, the slenderness ratio of the individual parts shall be not greater than 0.5 times the slenderness ratio of the member as a whole.

For bracing members having KL/r less than $2600/\sqrt{F_y}$ or Article 6.9.3 of the AASHTO

LRFD Provisions, the width-to-thickness ratios of bracing members should be limited as indicated in Table 8.7.4-1. For bracing members that exceed that value, the width-to-thickness ratio limits can be obtained by linear interpolation between the values in Table 8.7.4-1 when KL/r is equal to $2600/\sqrt{F_y}$. and 1.3 times the values in Table 8.7.4-1 when KL/r is equal to $3750/\sqrt{F_y}$..

For back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio limit can be taken as $200/\sqrt{F_y}$.

No width-to-thickness ratio limit is imposed for braces designed as tension-only members and having KL/r greater than $3750/\sqrt{F_y}$.

8.7.6.3 Brace Connections

Brace connections shall be designed as Capacity Protected Elements. The controlling overstrength capacity shall be taken as the axial tensile yield strength of the brace (A_gF_{ye}).

For tension-only bracing the controlling probable resistance shall be multiplied by an additional factor of 1.10.

Connections must be designed to ensure that the bracing member is capable of yielding the gross section. Consequently, brace strength calculated based on tension rupture on the effective net section and block shear rupture, shall be less that the design tensile strength of brace given by gross section yielding.

Stitches that connect the separate elements of built-up bracing members shall, if the overall buckling mode induces shear in the stitches, have a strength at least equal to one-half of the design tensile strength of each element. The spacing of stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

8.7.6.4 Columns, Beams and Other Connections

Columns, beams, and connections shall be designed as Capacity Protected Elements.

8.7.6.5 Chevron Braced and V-Braced Systems

Braces in chevron braced frames shall conform to the requirements of Article 8.7.6.2, except that bracing members shall have a slenderness ratio, KL/r, less than $2600/\sqrt{F_y}$. Tension-only designs are not permitted.

The beam attached to chevron braces or V-braces shall be continuous between columns and its top and bottom flanges shall be designed to resist a lateral load of 2% of the flange yield force ($F_yb_ft_{bf}$) at the point of intersection with the brace.

Columns, beams and connections shall be designed to resist forces arising from load redistribution following brace buckling or yielding, including the maximum unbalanced vertical load effect applied to the beam by the braces. The brace compressive resistance shall be $0.3\ \phi_cP_n$ if this creates a more critical condition.

A beam that is intersected by chevron braces shall be able to support its permanent dead and live loads without the support provided by the braces.

8.7.7 Concrete Filled Steel Pipes

Concrete-filled steel pipes use 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, 6.12.3.2.2, of the AASHTO LRFD provisions as well as the requirements in this article.

8.7.7.1 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 so that:

$$\frac{P_u}{P_r} + \frac{BM_u}{M_{rc}} \le 1.0 \qquad (8.7.7.1\text{-}1)$$

and

$$\frac{M_u}{M_{rc}} \le 1.0 \qquad (8.7.7.1\text{-}2)$$

where P_r is defined in Articles 6.9.2.1 and 6.9.5.1 of the AASHTO LRFD provisions, and M_{rc} is defined in Article 8.7.7.2

$$B = \frac{P_{ro} - P_{rc}}{P_{rc}} \qquad (8.7.7.1\text{-}3)$$

P_{ro} = factored compressive resistance (Articles 6.9.2.1 and 6.9.5.1 of the AASHTO LRFD provisions) with $\lambda = 0$

$$P_{rc} = \phi_c A_c f'_c \qquad (8.7.7.1\text{-}4)$$

M_u is the maximum resultant moment applied to the member in any direction, calculated as specified in Article 4.5.3.2.2 of the AASHTO LRFD provisions

8.7.7.2 Flexural Strength

The factored moment resistance of a concrete filled steel pipe for Article 8.7.7.1 shall be calculated using either of the following two methods:

(a) Method 1 – Using Exact Geometry

$$M_{rc} = \phi_f \ [C_r e + C'_r e'] \qquad (8.7.7.2\text{-}1)$$

where

$$C_r = F_y \beta \frac{Dt}{2} \qquad (8.7.7.2\text{-}2)$$

$$C'_r = f'_c \left[\frac{\beta D^2}{8} - \frac{b_c}{2}\left(\frac{D}{2} - a\right)\right] \qquad (8.7.7.2\text{-}3)$$

$$e = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{1}{\beta} \right] \qquad (8.7.7.2\text{-}4)$$

$$e' = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{b_c^2}{1.5\beta D^2 - 6b_c(0.5D - a)} \right] \qquad (8.7.7.2\text{-}5)$$

$$a = \frac{b_c}{2} \tan\left(\frac{\beta}{4}\right) \qquad (8.7.7.2\text{-}6)$$

$$b_c = D \sin\left(\frac{\beta}{2}\right) \qquad (8.7.7.2\text{-}7)$$

where β is in radians and found by the recursive equation:

$$\beta = \frac{A_s F_y + 0.25 D^2 f'_c \left[\sin(\beta/2) - \sin^2(\beta/2)\tan(\beta/4) \right]}{\left(0.125 \ D^2 f'_c + \ DtF_y\right)} \qquad (8.7.7.2\text{-}8)$$

(b) Method 2 – Using Approximate Geometry

A conservative value of M_{rc} is given by

$$M_{rc} = \phi_f \ \left[(Z - 2th_n^2)Fy + \left[\frac{2}{3}(0.5D - t)^3 - (0.5D - t)h_n^2 \right] f'_c \right] \qquad (8.7.7.2\text{-}9)$$

where

$$h_n = \frac{A_c f'_c}{2Df'_c + 4t(2F_y - f'_c)} \qquad (8.7.7.2\text{-}10)$$

and Z is the plastic modulus of the steel section alone.

For capacity design purposes, in determining the force to consider for the design of capacity protected elements, the moment calculated by this approximate method shall be increased by 10%.

8.7.7.3 Beams and Connections

Capacity-protected members must be designed to resist the forces resulting from hinging in the concrete-filled pipes calculated from Article 8.7.7.2.

8.7.8 Other Systems

This Article provides minimum considerations that must be addressed for the design of special systems.

8.7.8.1 Ductile Eccentrically Braced Frames

Ductile eccentrically braced frames for bents and towers may be used provided that the system, and in particular the eccentric link and link beam, can be demonstrated to remain stable up to the

expected level of inelastic response. This demonstration of performance shall be preferably achieved through full-scale cyclic tests of specimens of size greater or equal to that of the prototype.

Seismic design practice for eccentrically braced frames used in buildings can be used to select width-to-thickness ratios, stiffeners spacing and size, and strength of the links, as well as to design diagonal braces and beams outside of the links, columns, brace connections, and beam-to-column connections.

Only the eccentric brace configuration in which the eccentric link is located in the middle of a beam is permitted.

8.7.8.2 Ductile End-Diaphragm in Slab-on-Girder Bridge

Ductile end-diaphragms in slab-on-girder bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

(a) Specially detailed diaphragms capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;
(b) Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;
(c) Design considers the combined and relative stiffness and strength of end-diaphragms and girders (together with their bearing stiffeners) in establishing the diaphragms strength and design forces to consider for the capacity protected elements;
(d) The response modification factor to be considered in design of the ductile diaphragm is given by:

$$R = \left(\frac{\mu + \frac{K_{DED}}{K_{SUB}}}{1 + \frac{K_{DED}}{K_{SUB}}} \right) \qquad (8.7.8.2\text{-}1)$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrated otherwise, μ should not be taken greater than 4;

(e) All details/connections of the ductile end-diaphragms are welded.
(f) 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.
(g) An effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the slab to the diaphragm.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be based on available experimental research results.

8.7.8.3 Ductile End Diaphragms in Deck Truss Bridges

Ductile end-diaphragms in deck-truss bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

(a) Specially detailed diaphragms capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;
(b) Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;
(c) The last lower horizontal cross-frame before each support is also designed as a ductile panel equivalent and in parallel to its adjacent vertical end-diaphragm;
(d) Horizontal and vertical energy dissipating ductile panels are calibrated to have a ratio of stiffness approximately equal to their strength ratio;
(e) The concrete deck is made continuous between supports (and end-diaphragms),

and an effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the deck to the diaphragms.;

(h) The response modification factor to be considered in design of the ductile diaphragm is given by:

$$R = \left(\frac{\mu + \frac{K_{DED}}{K_{SUB}}}{1 + \frac{K_{DED}}{K_{SUB}}} \right) \qquad (8.7.8.2\text{-}2)$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrated otherwise, μ should not be taken greater than 4;

(i) All capacity-protected members are demonstrated able to resist without damage or instability the maximum calculated seismic displacements.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be based on available experimental research results.

8.7.8.4 Other Systems

Other framing systems and frames that incorporate special bracing, active control, or other energy absorbing devices, or other types of special ductile superstructure elements shall be designed on the basis of published research results, observed performance in past earthquakes, or special investigation, and provide a level of safety comparable to those in the AASHTO LRFD Specifications.

8.7.9 Plastic Rotational Capacities

The plastic rotational capacity shall be based on the appropriate performance limit state for the bridge. In lieu of the prescriptive values given below, the designer may determine the plastic rotational capacity from tests and/or a rational analysis.

8.7.9.1 Life Safety Performance

A conservative values of θ_p=0.035 radians may be assumed.

8.7.9.2 Immediate Use Limit State

To ensure the immediate use of the bridge structure following a design ground motion, the maximum rotational capacity should be limited to θ_p=0.005 radians.

8.7.9.3 In Ground Hinges

The maximum rotational capacity for in-ground hinges should be restricted to θ_p=0.01 radians.

8.8 REINFORCED CONCRETE DESIGN REQUIREMENTS

8.8.1 General

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 Article 9.2 of the AASHTO *LRFD Bridge Construction Specifications*.

High strength high alloy bars, with an ultimate tensile strength of up to 1600 MPa, may be used for longitudinal column reinforcement for seismic loading providing it can be demonstrated through tests that the low cycle fatigue properties is not inferior to normal reinforcing steels with yield strengths of 520 MPa or less.

Wire rope or strand may be used for spirals in columns if it can be shown through tests that the modulus of toughness exceeds 100MPa.

In compression members, all longitudinal bars shall be enclosed by perimeter hoops. Ties shall be used to provide lateral restraint to intermediate longitudinal bars within the reinforced concrete cross section.

Transverse hoops and ties that shall be equivalent to:

- No. 10 bars for No. 29 or smaller bars,
- No. 16 bars for No. 36 or larger bars, and
- No. 16 bars for bundled bars.

The spacing of transverse hoops and ties shall not exceed the least dimension of the compression member or 300 mm. Where two or more bars larger than No. 36 are bundled together, the spacing shall not exceed half the least dimension of the member or 150 mm.

Deformed wire, wire rope or welded wire fabric of equivalent area may be used instead of bars.

Hoops and ties shall be arranged so that every corner and alternate longitudinal bar has lateral support provided by the corner of a tie having an included angle of not more than 135°. Except as specified herein, no bar shall be farther than 150 mm center-to-center on each side along the tie from such a laterally supported bar.

Where the column design is based on plastic hinging capability, no longitudinal bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where the bars are located around the periphery of a circle, a complete circular tie may be used if the splices in the ties are staggered.

Ties shall be located vertically not more than half a tie spacing above the footing or other support and not more than half a tie spacing below the lowest horizontal reinforcement in the supported member.

8.8.2 Column Requirements

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 not less 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 Article 8.8.3 shall apply.

A pier may be designed as a pier in its strong direction and a column in its weak direction.

The piles of pile bents as well as drilled shaft 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 either "structurally isolated" in such a way that they do not add to the flexural strength capacity of the columns or the column and adjacent structural elements shall be designed to resist the forces generated by increased flexural strength capacity.

The size of the gap required for structural separation is 0.05 times the distance from the center of the column to the extreme edge of the flare, or 1.5 times the calculated plastic rotation from the pushover analysis times the distance from the center of the column to the extreme edge of the flare. Equation 8.8.6-4 provides an estimate of the reduced plastic hinge length at this location.

For oversized or architectural portions of piers or columns, minimum longitudinal and transverse reinforcement that complies with temperature and shrinkage requirements elsewhere in these specifications shall be provided.

8.8.2.1 Longitudinal Reinforcement

The area of longitudinal reinforcement shall not be less than 0.008 or more than 0.04 times the gross cross-section area A_g.

8.8.2.2 Flexural Resistance

The biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.6. The column shall be investigated for both extreme load cases (50% PE in 75 year and 3% PE in 75 year/1.5 mean deterministic as per Articles 4.4, 4.5 and 4.6. The resistance factors of Article 5.5.4.2 of the AASHTO LRFD provisions shall be replaced for both spirally and tied reinforcement columns by the value $\phi = 1.0$, providing other member actions have been designed in accordance with the principles of capacity design.

8.8.2.3 Column Shear and Transverse Reinforcement

Provision of transverse reinforcement for shear shall be determined by one of the following two methods: implicit approach or an explicit approach. The implicit approach may be used for all Seismic Hazard Levels. However, for Seismic Hazard Level IV with a two-step design (SDAP E), the shear strength shall be checked using the explicit approach.

Method 1: Implicit Shear Detailing Approach

(a) In potential plastic hinge zones (Article 4.9)

- For circular sections
- For rectangular sections

$$\rho_v = K_{shape}\,\Lambda\,\frac{\rho_t}{\phi}\,\frac{f_{su}}{f_{yh}}\,\frac{A_g}{A_{cc}}\tan\alpha\,\tan\theta \qquad (8.8.2.3\text{-}1)$$

in which

ρ_v = ratio of transverse reinforcement given by either equation 8.8.2.3-2 or 8.8.2.3-3.

- for rectangular sections

$$\rho_V = \frac{A_{sh}}{b_W s} \qquad (8.8.2.3\text{-}2)$$

and

- for circular columns

$$\rho_V = \frac{\rho_s}{2} = \frac{2A_{bh}}{sD''} \qquad (8.8.2.3\text{-}3)$$

where

A_{sh} = the area of the transverse hoops and cross-ties transverse to the axis of bending

A_{bh} = the area of one spiral bar or hoop in a circular section

S = the center-to-center spacing of hoopsets or the pitch the spiral steel

b_w = the web width resisting shear in a rectangular section D'' = spiral diameter in a circular section

The terms in equation (8.8.2.3-1) are defined below:

K_{shape} = factor that depends on the shape of the section and shall be taken as

- for circular sections $K_{shape} = 0.32$
- for square sections with 25 percent of the longitudinal reinforcement placed in each face $K_{shape} = 0.375$
- for walls with strong axis bending $K_{shape} = 0.25$
- for walls with weak axis bending $K_{shape} = 0.5$

Λ = fixity factor,
$\Lambda = 1$ fixed-free (pinned one end)
$\Lambda = 2$ fixed-fixed

f_{su} = the ultimate tensile stress of the longitudinal reinforcement. If f_{su} is not available from coupon tests, then it shall be assumed that $f_{su} = 1.5.\ f_y$. For SDR 2 f_{su} may be taken as f_y.

θ = angle of the principal crack plane given by

$$\tan\theta = \left(\frac{1.6}{\Lambda}\,\frac{\rho_v}{\rho_t}\,\frac{A_v}{A_g}\right)^{0.25} \qquad (8.8.2.3\text{-}4)$$

with $\theta \geq 25^\circ$ and $\theta \geq \alpha$

α = geometric aspect ratio angle given by

$$\tan\alpha = \frac{D'}{L}$$

where D' = pitch circle diameter of the longitudinal reinforcement in a circular section, or the distance between the outer layers of the longitudinal steel in other section shapes.

A_v = shear area of concrete which may be taken as $0.8A_g$ for a circular section, or $A_v = b_w d$ for a rectangular section.

The spacing of the spirals or hoopsets shall not exceed 250mm or one-half the member width.

(b) Outside the Potential Plastic Hinge Zone

Outside the potential plastic hinge zone (Article 4.9) the transverse reinforcement may be reduced to account for some contribution of the concrete in shear resistance. The required amount of transverse reinforcement, outside the potential plastic hinge zone ρ_v^*, shall be given by

$$\rho_v^* = \rho_v - 0.17\frac{\sqrt{f_c'}}{f_{yh}} \qquad (8.8.2.3\text{-}5)$$

where ρ_v = the steel provided in the potential

plastic hinge zone.

ρ_v^* shall not be less than the minimum amount of transverse reinforcement required elsewhere in these specifications based on non-seismic requirements.

Method 2: Explicit Approach

The design shear force, V_u, on each principal axis of each column and pile bent shall be determined from considerations of the flexural overstrength being developed at the most probable locations of critical sections within the member, with a rational combination of the most adverse end moments.

In the end regions, the shear resisting mechanism shall be assumed to be provided by a combination of truss (V_s) and arch (strut) action (V_p) such that

$$\phi V_s \geq V_u - \phi(V_p + V_c) \qquad (8.8.2.3\text{-}6)$$

where V_p = the contribution due to arch action given by

$$V_p = \frac{\Lambda}{2} P_e \tan\alpha \qquad (8.8.2.3\text{-}7)$$

where

$$\tan\alpha = \frac{D'}{L} \qquad (8.8.2.3\text{-}8)$$

P_e = compressive axial force including seismic effects

D' = pitch circle diameter of the longitudinal reinforcement in a circular column, or the distance between the outermost layers of bars in a rectangular column

L = column length

Λ = fixity factor defined above

V_c = the tensile contribution of the concrete towards shear resistance. At large displacement ductilities only a minimal contribution can be assigned as follows

$$V_c = 0.05\sqrt{f_c'}\, b_w d \qquad (8.8.2.3\text{-}9)$$

Outside the plastic hinge zone

$$V_c = 0.17\sqrt{f_c'}\, b_w d \qquad (8.8.2.3\text{-}10)$$

where

f_c' = concrete strength in MPa,

b_w = web width of the section, and

d = effective depth

V_s = the contribution of shear resistance provided by transverse reinforcement given by:

(i) for circular columns:

$$V_s = \frac{\pi}{2}\frac{A_{bh}}{s} f_{yh} D'' \cot\theta \qquad (8.8.2.3\text{-}12)$$

(ii) for rectangular sections

$$V_s = \frac{A_v}{s} f_{yh} D'' \cot\theta \qquad (8.8.2.3\text{-}13)$$

where

A_{bh} = area of one circular hoop/spiral reinforcing bar

A_{sh} = total area of transverse reinforcement in one layer in the direction of the shear force

f_{yh} = transverse reinforcement yield stress

D'' = centerline section diameter/width of the perimeter spiral/hoops

θ = principal crack angle/plane calculated as follows:

$$\tan\theta = \left(\frac{1.6\rho_v A_v}{\Lambda \rho_t A_g}\right)^{0.25} \geq \tan\alpha \qquad (8.8.2.3\text{-}14)$$

where

ρ_v = volumetric ratio of shear reinforcement given by

$$\rho_v = \frac{A_{sh}}{b_w s} \quad \text{for rectangular section}$$

$$\rho_v = \frac{\rho_s}{2} = \frac{2A_{bh}}{sD''} \quad \text{for circular columns.}$$

and A_v = shear area of concrete which may be taken as $0.8A_v$ for a circular section, or $A_v = b_w d$ for a rectangular section.

Extent of Shear Steel

Shear steel shall be provided in all potential plastic hinge zones as defined in Article 4.9.

8.8.2.4 Transverse Reinforcement for Confinement at Plastic Hinges

The core concrete of columns and pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions. The spacing shall be taken as specified in Article 8.8.2.6.

For a circular column, the volumetric ratio of spiral reinforcement, ρ_s, shall not be less than:

a) for circular sections

$$\rho_s = 0.008\frac{f_c'}{U_{sf}}\left[12\left(\frac{P_e}{f_c' A_g} + \rho_t \frac{f_y}{f_c'}\right)^2\left(\frac{A_g}{A_{cc}}\right)^2 - 1\right] \qquad (8.8.2.4\text{-}1)$$

b) for rectangular sections

$$\frac{A_{sh}}{sB''} + \frac{A_{sh}'}{sD''} = 0.008\frac{f_c'}{U_{sf}}\left[15\left(\frac{P_e}{f_c' A_g} + \rho_t \frac{f_y}{f_c'}\right)^2\left(\frac{A_g}{A_{cc}}\right)^2 - 1\right] \qquad (8.8.2.4\text{-}2)$$

where:

f_c' = specified compressive strength of concrete at 28 days, unless another age is specified (MPa)

f_y = yield strength of reinforcing bars (MPa)

P_e = factored axial load (N) including seismic effects

U_{sf} = strain energy capacity (modulus of toughness) of the transverse reinforcement = 110 MPa.

$\rho_s = \frac{4A_b}{D's}$ = ratio of transverse reinforcement where

A_b = area of longitudinal reinforcing bars restrained by rectilinear loops and/or cross ties.

D' = center-to-center diameter of perimeter hoop for spiral. Within plastic hinge zones, splices in spiral reinforcement shall be made by full-welded splices or by full-mechanical connections.

s = vertical spacing of hoops, not exceeding 100 mm (mm)

A_{cc} = area of column core concrete, measured to the centerline of the perimeter hoop or spiral (mm^2)

A_g = gross area of column (mm^2)

A_{sh} = total area of transverse reinforcement in the direction of the applied shear

A_{sh}' = total area of transverse reinforcement perpendicular to direction of the applied shear

B'' & D'' = core dimension of tied column in the direction under consideration (mm)

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 as specified in Article 5.10.2.2 of the AASHTO LRFD provisions.

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 75 mm at one end and a hook of not less than 90° with an extension not less than six diameters at the other end.

- Hooks shall engage all peripheral longitudinal bars.

Transverse reinforcement meeting the following requirements shall be 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 75 mm 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 75 mm extension that engages the longitudinal reinforcement.

8.8.2.5 Transverse Reinforcement for Longitudinal Bar Restraint in Plastic Hinges

The longitudinal reinforcement in the potential plastic hinge zone shall be restrained by antibuckling steel as follows:

(i) $s \leq 6d_b$ (8.8.2.5-1)

(ii) For circular sections confined by spirals or circular hoops

$$\rho_s = 0.016\left(\frac{D}{s}\right)\left(\frac{s}{d_b}\right)\rho_t \frac{f_y}{f_{yh}} \quad (8.8.2.5\text{-}2)$$

(iii) for rectangular sections confined by transverse hoops and/or cross ties the area of the cross tie or hoop legs (A_{bh}) shall be:

$$A_{bh} = 0.09 A_b \frac{f_y}{f_{yh}} \quad (8.8.2.5\text{-}3)$$

where

ρ_s = ratio of transverse reinforcement $\left(\rho_s = \frac{4A_{bh}}{sD},\right)$

D = diameter of circular column

d_b = diameter of longitudinal reinforcing bars being restrained by circular hoop or spiral

A_b = area of longitudinal reinforcing bars being restrained by rectilinear hoops and/or cross ties

A_{bh} = bar area of the transverse hoops or ties restraining The longitudinal steel

ρ_t = volumetric ratio of longitudinal reinforcement

f_y = yield stress of the longitudinal reinforcement

f_{yh} = yield stress of the transverse reinforcing bars

8.8.2.6 Spacing for Transverse Reinforcement for Confinement and Longitudinal Bar Restraint

Transverse reinforcement for confinement and longitudinal bar restraint (Articles 8.8.2.4 and 8.8.2.5 shall be provided at all plastic hinge zones as defined in Article 4.9 except that the requirements of Article 8.8.2.5 need not apply to the pile length from 3D to 10D below the pile cap.

The spacing of transverse reinforcement shall not be less than:

$$\frac{M}{V}\left(1 - \frac{M_y}{M_{po}}\right) \quad (8.8.2.6\text{-}1)$$

The spacing of transverse reinforcement shall not exceed one-quarter of the minimum member dimension or 150 mm center-to-center.

8.8.2.7 Splices

The provisions of Article 5.11.5 of the AASHTO LRFD provisions shall apply for the design of splices.

Lap splices in longitudinal reinforcement shall be used only within the center half of column height, and the splice length shall not be less than 400mm or 60-bar diameters.

The spacing of the transverse reinforcement over the length of the splice shall not exceed one-quarter of the minimum member dimension.

Full-welded or full-mechanical connection splices conforming to Article 5.11.5 of the AASHTO LRFD provisions may be used, provided that not more than alternate bars in each

layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is greater than 450mm measured along the longitudinal axis of the column.

8.8.2.8 Flexural Overstrength

Article 4.8 provides several alternate methods for calculating the flexural moment overstrength capacity (M_{po}) for columns/ piles/ drilled shafts that are part of the ERS. The plastic moment-axial load interaction formula of Equation C8.8.2.8-1 may be used to calculate the overstrength moment of a column or drilled shaft:

8.8.3 Limited Ductility Requirements for Wall Type Piers

These limited ductility provisions, herein specified, shall apply to the design for the strong direction of a pier. Providing ductile detailing is used, either direction of a pier may be designed as a column conforming to the provisions of Article 8.8.2, with the response modification factor for columns used to determine the design forces. If the pier is not designed as a column in either direction, then the limitations for factored shear resistance herein specified shall apply.

The minimum reinforcement ratio, both horizontally, ρ_h, and vertically, ρ_v, in any pier shall not be less than 0.0025. The vertical reinforcement ratio shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 450 mm. The reinforcement required for shear shall be continuous and shall be distributed uniformly.

The factored shear resistance, V_r, in the pier shall be taken as the lesser of:

$$V_r = 0.253\sqrt{f_c'}bd \qquad (8.8.3\text{-}1)$$

$$V_r = \phi V_n \qquad (8.8.3\text{-}2)$$

for which:

$$V_n = \left[0.063\sqrt{f_c'} + \rho_h y_y\right] bd \qquad (8.8.3\text{-}3)$$

Horizontal and vertical layers of reinforcement should be provided on each face of a pier. Splices in horizontal pier reinforcement shall be staggered and splices in the two layers shall not occur at the same location.

8.8.4 Moment Resisting Connection Between Members (Column/Beam and Column/Footing Joints)

8.8.4.1 Implicit Approach: Direct Design

Flexural reinforcement in continuous, restrained, or cantilever members or in any member of a rigid frame shall be detailed to provide continuity of reinforcement at intersections with other members to develop the nominal moment resistance of the joint.

In SDR 3 and above, joints shall be detailed to resist shears resulting from horizontal loads through the joint.

Transverse reinforcement in cap beam-to-column or pile cap-to-column joints should consist of the greater of:

(a) Confinement reinforcement given in Article 8.8.2.4.

(b) Longitudinal bar restraint reinforcement given by Article 8.8.2.5; this article can be waived if the longitudinal bars framing into the joint are surrounded by sufficient concrete to inhibit bar buckling. For the purpose of waiving this article cover to the longitudinal steel shall be taken as the greater of 150 mm or 6 longitudinal bar diameters.

(c) Shear reinforcement given by Article 8.8.2.3 where the principal crack angle θ is given by the aspect ratio of the member and is defined by the joint dimensions as follows

$$\tan\theta = \tan\alpha = \frac{D}{H_c}$$

where

$D =$ width or diameter of the column framing into the joint

H_c = the height of the cap beam/joint. Thus the joint shear horizontal (transverse) reinforcement is given by:

For circular columns with spirals or circular hoops

$$\rho_s \geq 0.76 \frac{\rho_t}{\phi} \frac{f_{su}}{f_{yh}} \frac{A_g}{A_{cc}} \tan^2 \alpha. \quad (8.8.4.1\text{-}1)$$

for rectangular sections with rectilinear hoops and/or ties

$$\frac{A_{sh}}{sB''} \geq 1.2 \frac{B'/D'+0.5}{2B'/D'+2} \frac{\rho_t}{\phi} \frac{f_{su}}{f_{yh}} \frac{A_g}{A_{cc}} \tan^2 \alpha \quad (8.8.4.1\text{-}2)$$

If the above equations lead to congested steel placement details, then alternative details may be adopted through the use of rational strut and tie models as given in Article 8.8.4.2.

where

ρ_s = ratio of transverse hoops/spirals $\left(\rho_s = \frac{4A_{bh}}{sD'}\right)$

ρ_t = ratio of longitudinal reinforcement area to gross area of section

A_{sh} = area of transverse reinforcement in the direction of the applied shear

f_{su} = yield strength of transverse reinforcement

A_g = gross area of section

A_{cc} = confined core area (take as $0.8A_g$ for a circular section

ϕ = resistance factor for seismic shear (0.85)

8.8.4.2 Explicit Approach: Detailed Design

8.8.4.2.1 Design Forces and Applied Stresses

Moment-resisting connections between members shall be designed to transmit the maximum forces applied by the connected members. Connection forces shall be based on the assumption of maximum plastic moment.

Forces acting on the boundaries of connections shall be considered to be transmitted by mechanisms involving appropriate contributions by concrete and reinforcement actions. Mechanisms shall be based on an analysis of force-transfer within the connection, and shall be supported by relevant test results.

Principal stresses is any vertical plane within a connection shall be calculated in accordance with Eq. (8.8.4.2-1) and (8.8.4.2-2)

Principal tension stress is given by:

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \quad (8.8.4.2\text{-}1)$$

Principal compression stress is given by:

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \quad (8.8.4.2\text{-}2)$$

where

f_h and f_v = the average axial stresses in the horizontal and vertical directions within the plane of the connection under consideration (compression stress positive) and

v_{hv} = the average shear stress within the plane of the connection.

8.8.4.2.2 Minimum Required Horizontal Reinforcement

When the principal tension stress is less than $P_t = 0.29\sqrt{f_c'}$ MPa, the minimum amount of horizontal joint shear reinforcement to be provided shall be capable of transferring 50 percent of the cracking stress resolved to the horizontal direction. For circular columns, or columns with intersecting spirals, the volumetric ratio of transverse reinforcement in the form of spirals or circular hoops to be continued into the cap or footing shall not be less than

$$\rho_s = \frac{0.29\sqrt{f_c'}}{f_{yh}} \quad (8.8.4.2\text{-}3)$$

where

f_{yh} = yield stress of horizontal hoop/tie reinforcement in the joint.

8.8.4.2.3 Maximum Allowable Compression Stresses

Principal compression stress in a connection, calculated in accordance with Equation 8.8.4.2-2 shall not exceed $p_c = 0.25 f_c^{'}$.

8.8.4.3 Reinforcement for Joint Force Transfer

8.8.4.3.1 Acceptable Reinforcement Details

Where the magnitude of principal tension stress values (calculated in accordance with Equation 8.8.4.2-1), exceed $\rho_t = 0.29\sqrt{f_c^{'}}$ MPa, vertical and horizontal joint rein-forcement, placed in accordance with Articles 8.8.4.3.2, 8.8.4.3.3 and 8.8.4.3.4.is required.

8.8.4.3.2 Vertical Reinforcement

Stirrups

On each side of the column or pier wall, the beam member that is subject to bending forces shall have vertical stirrups, with a total area $A_{jv} = 0.16 A_{st}$ located within a distance $0.5D$ or $0.5h$ from the column or pier wall face. These vertical stirrups shall be distributed over a width not exceeding $2D$.

where

A_{st} = total area of longitudinal steel

D = diameter of circular column

h = depth of rectangular column

Clamping Reinforcement

Longitudinal reinforcement contributing to cap beam or footing flexural strength (i.e., superstructure top reinforcement, cap top reinforcement, footing bottom reinforcement) shall be clamped into the joint by vertical bars providing a total area of $0.08 A_{ST}$. These bars shall be hooked around the restrained longitudinal reinforcement and extend into the joint a distance not less than two-thirds of the joint depth. If more than 50 percent of the superstructure moment capacity and/or cap-beam moment capacity is provided by prestress, this reinforcement may be omitted, unless needed for the orthogonal direction of response.

8.8.4.3.3 Horizontal Reinforcement

Additional longitudinal reinforcement in the cap beam, superstructure, and footing of total amount $0.08 A_{ST}$ over and above the required for flexural strength, shall be placed in the face adjacent to the column (i.e., bottom of cap beam or superstructure; top of footing), extending through the joint and for a sufficient distance to develop its yield strength at a distance of $0.5D$ from the column face, as shown in Figure 8.8.4.2-1

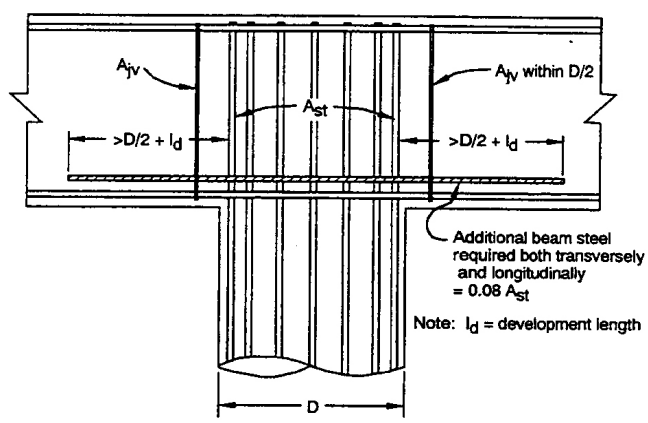


Figure 8.8.4.2-1 Additional cap beam bottom reinforcement for joint force transfer.

8.8.4.3.4 Hoop or Spiral Reinforcement

The required volumetric ratio of column joint hoop or spiral reinforcement to be carried into the cap or footing shall not be less than

$$\rho_s \geq \frac{0.4 A_{ST}}{\ell_{ac}^2} \qquad (8.8.4.3\text{-}1)$$

8.8.4.4 Footing Strength

In determining the flexural strength of footings resisting gravity plus seismic overloads, with monolithic column/footing connections, the effective width of the footing shall not be taken to

be greater than the width of the column plus a tributary footing width, equal to the effective depth of the footing, on either side of the column.

The effective width for determining the shear strength of footings for gravity plus seismic overloads shall be as for flexural overstrength

When the nominal shear strength in footings arising from the maximum flexural overstrength, vertical stirrups or ties shall be provided to carry the deficit in shear strength. These stirrups shall be placed within the effective width as defined above.

8.8.5 Concrete Piles

8.8.5.1 Transverse Reinforcement Requirements

The upper end of every pile shall be reinforced and confined as a potential plastic hinge region as specified in Article 4.9, except where it can be established that there is no possibility of any significant lateral deflection in the pile. If an analysis of the bridge and pile system indicates that a plastic hinge can form at a lower level, the plastic hinge zone shall extend 3D below the point of maximum moment. The transverse reinforcement in the top 3D of the pile shall be detailed for the maximum of shear, confinement, and longitudinal bar restraint as for concrete columns described in Article 8.8.2. The top 10D of the pile shall be detailed for the maximum of shear and confinement as for concrete columns and described in Articles 8.8.2.3 and 8.8.2.4.

8.8.5.2 Volumetric Ratio of Transverse Reinforcement

In lieu of a precise soil structure interaction analysis to ascertain the shear demand, a value of α = 25 degrees may be assumed for use in the implicit shear design equations.

8.8.5.3 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, provided by not less than four bars, shall not be less than 0.008.

8.8.6 Plastic Rotation Capacities

The plastic rotational capacity shall be based on the appropriate performance limit state for the bridge. In lieu of the prescriptive values given below, the designer may determine the plastic rotational capacity from tests and/or a rational analysis.

8.8.6.1 Life Safety Performance

The plastic rotational capacity of hinges shall be based on

$$\theta_p = 0.11\frac{L_p}{D}, \left(N_f\right)^{-0.5} \qquad rad \qquad (8.8.6\text{-}1)$$

in which

N_f = number of cycles of loading expected at the maximum displacement amplitude which may be estimated from

$$N_f = 3.5\ \left(T_n\right)^{-1/3}$$
$$2 \le N_f \le 10 \qquad (8.8.6\text{-}2)$$

where T_n = natural period of vibration of the structure.

For liquifiabile soils and piled foundation assessment, use $N_f = 2$

L_p = effective plastic hinge length give by

$$L_p = 0.08\frac{M}{V} + 4400\varepsilon_y d_b \qquad (8.8.6\text{-}3)$$

where

M/V = shear span of the member (*M* = end moment *V* = shear force)

ε_y = yield strain of the longitudinal reinforcement;

When an isolation gap of length L_g is provided between a structurally separated flare and an adjacent structural element, the plastic hinge length is given by

$$L_p = L_g + 8800\varepsilon_y d_b \qquad (8.8.6\text{-}4)$$

where L_g is the gap between the flare and the adjacent element.

D' = the distance between the outer layers of the longitudinal reinforcement on opposite faces of the member, equal to the pitch circle diameter for a circular section.

d_b = diameter of the main longitudinal reinforcing bars.

In lieu of the precise analysis given above, a conservative value of $\theta_p = 0.035 \quad rad$ shall be assumed.

For life-safety assessment of pile foundations that are potentially liquifiable, then $\theta_p = 0.05 rad$

8.8.6.2 Immediate Use Performance

To ensure the immediate use of the bridge structure following a design ground motion, the maximum rotational capacity should be limited to $\theta_p = 0.01 \quad rad$.

8.8.6.3 In-Ground Hinges

The maximum rotational capacity for in-ground hinges shall be restricted to $\theta_p = 0.02$ rad.

8.9 BEARING DESIGN REQUIREMENTS

There are three design or testing alternates for bearings that are not designed and tested as seismic isolation bearings as per Article 8.10. Alternate 1 requires both prototype and quality control testing of bearings as per Article 8.9.1. If testing of bearings is not performed for the required forces and displacements, then Alternate 2 provides a design option to provide a positive restraint system for the bearing. The restraint shall be capable of resisting the forces generated in the 3% PE in 75 year/1.5 mean deterministic event utilizing an analytical model that assumes that all bearings so designed are restrained. Alternate 3 provides a design option that permits a bearing to fail, provided there is a flat surface on which the girders can slide. The bearing or masonry plinth cannot impede the movement. The bridge must be analyzed in this condition and allowance for 150% of the calculated movement shall be provided.

If Alternate 3 is selected then a non-linear time history analysis is required using an appropriate coefficient of friction for the sliding surface to determine the amount of displacement that will result. The bearings shall be assumed to have failed early in the time history so a conservative value of the displacement is obtained.

8.9.1 Prototype and Quality Control Tests

Prototype Tests – each manufacturer shall perform a set of prototype tests on two full size bearings to qualify that particular bearing type and size for the rated forces or displacements of it's application. The sequence of tests shall be those given in Article 15.10.2 for the displacement or force for which it is to be qualified. For fixed bearings, the sequence of tests shall be performed for 110% of the lateral force capacity of the bearing where 110% of the force capacity replaces the total design displacement in Article 15.10.2. For bearings that permit movement, the total design displacement shall be 110% of the displacement for which they are to be qualified.

Quality Control Tests – a set of quality control tests shall be performed on 1 out of every 10 bearings of a given type and size. The tests shall be similar to those required for isolation bearings as specified in Articles 15.12.2, 15.14.2 and 15.15.6. For fixed bearings, the total design displacement shall be replaced by the lateral force capacity for which they are qualified.

8.10 SEISMIC ISOLATION DESIGN REQUIREMENTS

The design and testing requirements for the isolators are given in Articles 15.12 through 15.15

The analysis requirements for a seismically isolated bridge are given in Article 5.3.6 and Article 5.4.1.1 for the capacity spectrum method and Article 5.4.2.3 for a multi-mode analysis and Article 5.4.4 for a nonlinear time-history analysis. Other analysis and modeling issues are given in Article 15.4 and design properties of the isolators are given in Article 15.5. If an upper and lower bound analysis is performed as per Article 15.4, then the design forces and displacement shall be

the maximum of those obtained from the upper and lower bound analyses respectively.

The supporting substructures may be all designed elastically using the provisions of Article 4.10. If an R of 1.5 as per Table 4.7-1 is used to design the substructure, all other elements connected to the column shall be designed using the Capacity Design procedures of Article 4.8. The design and testing of the isolator units is given in Article 15.10 and other design issues related to the isolators are given in Section 15.

8.11 SUPERSTRUCTURE DESIGN REQUIREMENTS

The provisions of this section apply in SDAP C, D and E for SDR 4, 5, and 6. Unless noted otherwise these provisions apply to both levels of earthquake.

8.11.1 General

The superstructure shall either be capacity-protected, such that inelastic response is confined to the substructure or designed for the elastic seismic forces of the 3% PE in 75-year event/1.5 mean deterministic. If capacity protection is used, the overstrength forces developed in the piers and the elastic forces at the abutments shall be used to define the forces that the superstructure must resist. In addition to the earthquake forces, the other applicable forces for the Extreme Event combination shall be used. The combined action of the vertical loads and the seismic loads shall be considered. The superstructure shall remain essentially elastic using nominal properties of the members under the overstrength forces or elastic forces corresponding to the 3% PE in 75-year/1.5 mean deterministic earthquake, whichever are selected by the designer.

8.11.2 Load Paths

Load paths for resistance of inertial forces, from the point of origin to the points of resistance, shall be engineered. Positive connections between elements that are part of the earthquake resisting system (ERS) shall be provided. Bridges with a series of multi – simple spans cannot use the abutments to resist longitudinal forces from spans other than the two end spans. Longitudinal forces from interior spans may only be transferred to the abutments when the superstructure is continuous.

8.11.3 Effective Superstructure Width

The width of superstructure that is effective in resisting longitudinal seismic forces is dependent on the ability of the piers and abutments to effectively resist such forces. In the case of longitudinal moment transfer from the superstructure to the substructure, the pier cap beam shall be designed to resist forces transferred at the connection locations with the substructure. If such resistance is not provided along the cap beam, then a reduced effective superstructure width shall be used. This width shall be the sum of the column width along the transverse axis and the superstructure depth for open-soffit superstructures (e.g. I-girder bridges) or the column width plus twice the superstructure depth for box girders and solid superstructures. The effective width is to be taken transverse to the column at the pier and may be assumed to increase at a 45-degree angle as one moves along the superstructure until the full section becomes effective.

For superstructures with integral cap beams at the piers, the effective width of the cap beam may be as defined in Section 4.6.2.6 of the AASHTO LRFD provisions.

8.11.4 Superstructure to Substructure Connections

The provisions of this section apply in SDAP B, D, and E. These provisions apply to both levels of earthquake.

8.11.4.1 Connection Design Forces

The forces used for the design of connection elements shall be the lesser of the 3% PE in 75-year/1.5 mean deterministic elastic forces or the overstrength forces developed in the substructure below the connection as per Article 4.8.

8.11.4.2 Fuse Elements and Abutment Connections

Where connections or adjacent structure is designed to fuse (e.g. shear keys at abutments that

might be intended to breakaway in the 3% in 75-year/1.5 mean deterministic earthquake), the design forces shall correspond to an upper-bound estimate of the force required to fuse the element. The materials and details used to create fuse elements shall be chosen such that reasonable predictability of the fuse strength is assured.

Section 15
SEISMIC ISOLATION

15.1 SCOPE

Criteria provided herein for bearings used in implementing seismic isolation design are supplemental to Section 14 of the AASHTO *LRFD Bridge Design Specifications*. These provisions are necessary to provide a rational design procedure for isolation systems incorporating the displacements resulting from the seismic response. If a conflict arises between the provisions of the LRFD Sections 14 and this section, the provisions contained herein govern. The seismic isolation provisions in this Guide Specification maintain their Section 15 nomenclature because this is a new section that will be added to the existing LRFD provisions. Sections 1 through 8 of this Guide Specification will be inserted in existing sections of the LRFD provisions.

These specifications are intended for systems that isolate in the horizontal plane only – that is, the system is assumed to be essentially rigid in the vertical direction. In addition, the criteria are currently intended for passive isolation systems only.

15.2 DEFINITIONS

- DESIGN DISPLACEMENT is the lateral seismic displacement at the center of rigidity, required for design of the isolation system.

- EFFECTIVE DAMPING is the value of equivalent viscous damping corresponding to the energy dissipated during cyclic response at the design displacement of the isolated structure.

- EFFECTIVE STIFFNESS is the value of the maximum lateral force at instance of maximum lateral displacement in the isolation system, or an element thereof, divided by the maximum lateral displacement.

- ELASTIC RESTRAINT SYSTEM is the collection of structural elements that provide restraint of the seismically isolated structure for nonseismic lateral loads. The elastic restraint system may be either an integral part of the isolation system or may be a separate device.

- ISOLATION SYSTEM is the collection of all the elements that provide vertical stiffness, lateral flexibility, and damping to the system at the isolation interface. It includes the isolator units and the elastic restraint system, if one is used.

- ISOLATOR UNIT is a horizontally flexible and vertically stiff bearing of the isolation system, which permits large lateral deformation under seismic load. The isolator unit may or may not provide energy dissipation.

- OFFSET DISPLACEMENT is the lateral displacement of an isolator unit resulting from creep, shrinkage, and 50 percent of the thermal displacement.

- TOTAL DESIGN DISPLACEMENT is the maximum lateral seismic displacement of an isolator unit resulting from the analysis and required for design of the isolation system, including both translational displacement at the center of rigidity, Δ_i, and the component of torsional displacement in the direction under consideration.

15.3 NOTATION

A_b = Bonded area of elastomer.

A_r = Overlap area between the top-bonded and bottom-bonded elastomer areas of displaced bearing (figure C15.3-1).

B = Numerical coefficient related to the effective damping of the isolation system as set forth in Table 15.4.1-1.

B_d = Bonded plan dimension or bonded diameter in loaded direction of rectangular bearing or diameter of circular bearing (FigureC15.3-1).

C_d = Elastic seismic demand response coefficient.

DL = Dead load.

E = Young's modulus of elastomer.

EDC = Energy dissipated per cycle (area of hysteresis loop).

V = Statically equivalent seismic force.

V_A = Design force for connections for bridges in Seismic Design and Analysis Procedure (SDAP A).

F_i = Force in the isolator unit at displacement Δi.

F_n = Maximum negative force in an isolator unit during a single cycle of prototype testing.

$F_{n,\,max}$ = Maximum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude.

$F_{n,\,min}$ = Minimum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude.

F_p = Maximum positive force in an isolator unit during a single cycle of prototype testing.

$F_{p,\,max}$ = Maximum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude.

$F_{p,\,min}$ = Minimum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude.

F_v = Site soil coefficient given in Article 3.4.2.

G = Shear modulus of elastomer.

g = Acceleration due to gravity.

$\overline{k}$ = Elastomer material constant.

k_{iso} = Effective stiffness of an isolator unit determined by prototype testing.

k_{max} = Maximum effective stiffness of the isolator unit at the design displacement in the horizontal direction under consideration.

k_{min} = Minimum effective stiffness of the isolator unit at the design displacement in the horizontal direction under consideration.

k_{sub} = Stiffness of the substructure protected by the isolation unit(s)

K = Bulk modulus of the elastomer (Article 15.11).

K_d = The second slope stiffness of the bilinear hysteresis curve.

K_{eff} = The sum of the effective linear stiffnesses of all bearings and substructures supporting the superstructure segment as calculated at displacement Δi for the bearings and displacement Δsub for the substructure.

LL = Live load.

LLs = Seismic live load.

OT = Additional vertical load on bearing resulting from overturning moment effect of horizontal loads.

P = Maximum vertical load resulting from the combination of dead load plus live load (including seismic live load, if applicable), plus overturning moment effect of horizontal loads.

Q_d = Characteristic strength of the isolator unit. It is the ordinate of the hysteresis loop at zero bearing displacement. Refer to Figure C15.1-4.

S_1 = The one-second period spectral acceleration given in Article 3.10.2.1.

S = Shape factor (Article 15.11).

S_A = Spectral acceleration.

S_D = Spectral displacement.

T_{eff} = Period of seismically isolated structure, in seconds, in the direction under consideration.

T_r = Total elastomer thickness.

t_i = Thickness of elastomer layer number i, which is equivalent to the term hri in Article 14.7.5.1.

W = The total vertical load for design of the isolation system (DL + LLs).

Δ = Total deck displacement relative to ground ($\Delta_i + \Delta_{sub}$).

Δ_i = Design displacement at the center of rigidity of the isolation system in the direction under consideration.

Δ_{os} = Offset displacement of the isolator unit, including creep, shrinkage, and 50 percent of the thermal displacement.

Δ_{sub} = Substructure displacement.

Δ_t = Total design displacement.

Δ_n = Maximum negative displacement of an isolator unit during each cycle of prototype testing.

Δ_p = Maximum positive displacement of an isolator unit during each cycle of prototype testing.

Δ_s = Shear deformation of bearing from non-seismic displacement of the superstructure (including temperature, shrinkage, and creep).

ß = Equivalent viscous damping ratio for the isolation system.

$ß_i$ = Equivalent viscous damping ratio for isolator.

γ_c = Shear strain due to vertical loads.

$\gamma_{s,eq}$ = Shear strain due to Δ_t, the total seismic design displacement.

$\gamma_{s,s}$ = Shear strain due to maximum horizontal displacement resulting from creep, post-tensioning, shrinkage, and thermal effects computed between the installation temperature and the least favorable extreme temperature.

γ_r = Shear strain due to imposed rotation.

θ = Rotation imposed on bearing.

$\lambda_{max}, \lambda_{min}$ = System property modification factors to account for effects of temperature, aging, scragging, velocity, and variability of materials (Article 15.5.2).

15.4 ANALYSIS PROCEDURES

Articles 3.7 and 5.1 shall be used to define the analysis procedures.

The analysis of the bridge shall be performed using the design properties of the isolation system. To simplify the nonlinear behavior of the isolator unit, a bilinear simplification may be used. The analysis shall be repeated using upper-bound properties (Qd,max, Kd,max) in one analysis and lower-bound properties (Qd,min, Kd,min) in another, where the maximum and minimum values are defined in Article 15.5.1.2. The purpose of this upper- and lower-bound analysis is to determine the maximum forces on the substructure elements and the maximum displacements of the isolation system.

An upper- and lower-bound analysis is not required if the displacements, using Equation 15.4.1-3, and the statically equivalent seismic force, using Equation 15.4.1-2a, do not vary from the design values by more than ±15 percent when the maximum and minimum values of the isolator units properties are used. For these simplified calculations, B values corresponding to more than 30-percent damping can be used to establish the ±15-percent limits.

A nonlinear time-history analysis is required for structures with effective periods greater than 3 seconds.

For isolation systems where the effective damping expressed as a percentage of critical damping exceeds 30 percent of critical, a three-dimensional nonlinear time-history analysis shall be performed utilizing the hysteresis curves of the isolation system.

15.4.1 Capacity Spectrum Method

This method of analysis can be used when the regularity requirements of Table 5.4.2.1-1 are met.

The statically equivalent seismic force is given by

$$V = C_d W \qquad (15.4.1\text{-}1)$$

The elastic seismic response demand coefficient, C_d, used to determine the equivalent force, is given by the dimensionless relationship

$$C_d = \frac{K_{eff} \times \Delta}{W} = \frac{F_v S_1}{T_{eff} B} \qquad (15.4.1\text{-}2)$$

The displacement Δ is given by

$$\Delta = \frac{0.25 F_v S_1 T_{eff}}{B} \text{ (m)} \qquad (15.4.1\text{-}3a)$$

$$\Delta = \frac{10 F_v S_1 T_{eff}}{B} \text{ (inches)} \qquad (15.4.1\text{-}3b)$$

$$T_{eff} = 2\pi \sqrt{\frac{W}{K_{eff} g}} \qquad (15.4.1\text{-}4)$$

Note: This method of analysis shall not be used if Type E and F soils are present. For systems that include a viscous damper, the maximum force in the system may not correspond to the point of maximum displacement (Equation 15.4.1-1). The procedure described in the commentary shall be used.

Table 15.4.1-1 Damping Coefficient *B*

	Damping (Percentage of Critical)*						
	≤2	5	10	20	30	40	50
B	0.8	1.0	1.2	1.5	1.7	1.9	2.0

*The percentage of critical damping depends on the energy dissipated and stored by the isolation system, which shall be determined by test of the isolation system's characteristics, and by the substructure. The damping coefficient shall be based on linear interpolation for damping levels other than those given. Note that for isolation systems where the effective damping exceeds 30 percent, a nonlinear time-history analysis shall be performed utilizing the hysteresis curves of the system.

15.4.2 Uniform Load Method

The statically equivalent force determined according to Article 15.4.1, which is associated with the displacement across the isolation bearings, shall be applied using the uniform load method of analysis described in Article 5.4.2.2 independently along two perpendicular axes and combined as specified in Article 3.6. The effective stiffness of the isolators used in the analysis shall be calculated at the design displacement.

15.4.3 Multimode Spectral Method

An equivalent linear response spectrum shall be performed using the requirements of Article 5.4.2.3 when required by the regularity limitations of Article 5.4.2.1. The 5% damped spectra may be scaled by the damping coefficient (B), as defined in Article 15.4.1, to represent the actual seismic hazard and the effective damping of the isolation system for the isolated modes. Scaling by the damping coefficient B shall apply only for periods greater than 0.8 Teff. The 5-percent ground-motion response spectra shall be used for all other modes. The effective linear stiffness of the isolators shall correspond to the design displacement.

The combination of orthogonal seismic forces shall be as specified in Article 3.6.

15.4.4 Time-History Method

For isolation systems requiring a time-history analysis, the following requirements and Article 5.4.4 shall apply:

(a) The isolation system shall be modeled using the nonlinear deformational characteristics of the isolators determined and verified by test in accordance with the requirements of Article 15.10.

(b) Pairs of horizontal ground-motion time-history components shall be selected from no fewer than three earthquakes as required by Article 3.4.4.

(c) Time-history analysis shall be performed with at least three appropriate pairs of horizontal time-history components.

Each pair of time histories shall be applied simultaneously to the model. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the orthogonal displacements at each time step.

The parameter of interest shall be calculated for each time-history analysis. If three

time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

15.5. DESIGN PROPERTIES OF THE ISOLATION SYSTEM

15.5.1 Nominal Design Properties

The minimum and maximum effective stiffness of the isolation system (K_{min} and K_{max}) shall be determined from the minimum and maximum values of K_d and Q_d.

The minimum and maximum values of K_d and Q_d shall be determined as follows:

$$K_{d,max} = K_d \times \lambda_{max,K_d} \quad (15.5.1\text{-}1)$$

$$K_{d,min} = K_d \times \lambda_{min,K_d} \quad (15.5.1\text{-}2)$$

$$Q_{d,max} = Q_d \times \lambda_{max,Q_d} \quad (15.5.1\text{-}3)$$

$$Q_{d,min} = Q_d \times \lambda_{min,Q_d} \quad (15.5.1\text{-}4)$$

System property modification factors (λ) (defined in Article 15.5.2) used for design shall be established by system characterization tests and approved by the engineer. In lieu of the test values, the λ values given in Appendix 15A may be used

15.5.2 System Property Modification Factors (λ)

The mechanical properties of the isolator units are affected by temperature, aging, scragging, velocity, travel, and contamination.

15.5.2.1 Minimum and Maximum System Property Modification Factors

$$\lambda_{min,K_d} = \lambda_{min,t,K_d} \times \lambda_{min,a,K_d} \times \lambda_{min,v,K_d} \times \lambda_{min,tr,K_d} \times \lambda_{min,c,K_d} \times \lambda_{min,scrag,K_d} \quad (15.5.2\text{-}1)$$

$$\lambda_{max,K_d} = \lambda_{max,t,K_d} \times \lambda_{max,a,K_d} \times \lambda_{max,v,K_d} \times \lambda_{max,tr,K_d} \times \lambda_{max,c,K_d} \times \lambda_{max,scrag,K_d} \quad (15.5.2\text{-}2)$$

$$\lambda_{min,Q_d} = \lambda_{min,t,Q_d} \times \lambda_{min,a,Q_d} \times \lambda_{min,v,Q_d} \times \lambda_{min,tr,Q_d} \times \lambda_{min,c,Q_d} \times \lambda_{min,scrag,Q_d} \quad (15.5.2\text{-}3)$$

$$\lambda_{max,Q_d} = \lambda_{max,t,Q_d} \times \lambda_{max,a,Q_d} \times \lambda_{max,v,Q_d} \times \lambda_{max,tr,Q_d} \times \lambda_{max,c,Q_d} \times \lambda_{max,scrag,Q_d} \quad (15.5.2\text{-}4)$$

where:

λ_t = Factors to account for effects of temperature

λ_a = Factors to account for effects of aging (including corrosion)

λ_v = Factors to account for effects of velocity (including frequency for elastomeric systems)

$$\lambda_v = \frac{\text{Property value at relevant velocity}}{\text{Property value at velocity of testing}}$$

λ_{tr} = Factors to account for effects of travel (wear)

λ_c = Factors to account for effects of contamination (in sliding systems)

λ_{scrag}= Factors to account for effects of scragging a bearing (in elastomeric systems)

15.5.2.2 System Property Adjustment Factors

Adjustment factors are applied to individual λ factors to account for the probability of occurrence. The following adjustment factors shall apply to all λ factors except λ_v:

1.0 for operational bridges

0.67 for all other bridges

The adjustment factors shall apply to the portion of a λ that deviates from unity.

15.6 CLEARANCES

The clearances in the two orthogonal directions shall be the maximum displacement determined in each direction from the analysis. The clearance shall not be less than

$$\frac{0.20 F_v S_1 T_{eff}}{B} \text{(m)} \quad (15.6\text{-}1a)$$

$$\frac{8F_vS_1T_{eff}}{B}\text{(inches)} \qquad (15.6\text{-}1b)$$

or 1 inch (25 mm), whichever is greater.

Displacements in the isolators resulting from load combinations involving BR, WS, WL, CE, and T shall be calculated and adequate clearance provided.

The minimum design forces shall be consistent with the clearances calculated with Equation 15.6-1.

15.7 DESIGN FORCES FOR SDAP A1 AND A2

The seismic design force for the connection between superstructure and substructure at each bearing is given by

$$V_A = k_{eff}\Delta_t \qquad (15.7\text{-}1)$$

where Δ_t shall be based on a minimum value of F_vS_1, not less than 0.25.

15.8 DESIGN FORCES FOR SDAP C, D, AND E

The seismic design force for columns and piers shall not be less than the forces resulting from the yield level of a softening system, the friction level of a sliding system, or the ultimate capacity of a sacrificial service restraint system. In all cases the larger of static or dynamic conditions shall apply.

If the elastic foundation forces are less than the forces resulting from column hinging, they may be used for the foundation design. The foundation shall be designed using an R value equal to 1.0.

The seismic design force for the connection between the superstructure and substructure at each bearing is given by

$$V_a = k_{eff}\,\Delta_t \qquad (15.8\text{-}1)$$

Where Δ_t is the total design displacement and includes Δ_i the center of mass displacement from seismic forces plus any displacement resulting from torsional effects.

15.9 OTHER REQUIREMENTS

15.9.1 Non-Seismic Lateral Forces

The isolation system must resist all non-seismic lateral load combinations applied above the isolation interface. Such load combinations are those involving WS, WL, BR, CE, and T.

15.9.1.1 Service Force Resistance

Resistance to forces such as wind, centrifugal, and braking, and forces induced by restraint of thermal displacements, shall be established by testing in accordance with Article 15.10.2.

15.9.1.2 Cold Weather Requirements

Cold weather performance shall be considered in the design of all types of isolation systems. Low-temperature zones shall conform with Figure 14.7.5.2-1 in the absence of more site-specific data.

15.9.2 Lateral Restoring Force

The isolation system shall be configured to produce a lateral restoring force such that the period corresponding to its tangent stiffness based on the restoring force alone at any displacement, Δ, up to its design displacement shall be less than 6 seconds (Figure C15.9.2-1). Also the restoring force at Δ_i shall be greater than the restoring force at 0.5 Δ_i by not less than W/80. Isolation systems with constant restoring force need not satisfy the requirements above. In these cases, the combined constant restoring force of the isolation system shall be at least equal to 1.05 times the characteristic strength of the isolation system under service conditions.

Forces that are not dependent on displacements, such as viscous forces, may not be used to meet the minimum restoring force or tangent stiffness requirements.

15.9.3 Vertical Load Stability

The isolation system shall provide a factor of safety of at least three (3) for vertical loads (dead load plus live load) in its laterally undeformed state. It shall also be designed to be stable under 1.2 times the dead load plus any vertical load resulting from seismic live load, plus overturning at a horizontal displacement equal to the offset displacement plus 1.1 times the total design displacement, plus 0.5 times the design rotation.

15.9.4 Rotational Capacity

The design rotation capacity of the isolation unit shall include the effects of dead load, live load, and construction misalignments. In no case shall the design rotation for the construction misalignment be less than 0.005 radians.

15.10 REQUIRED TESTS OF ISOLATION SYSTEMS

All isolation systems shall have their seismic performance verified by testing. In general, there are three types of tests to be performed on isolation systems: (1) system characterization tests, described in Article 15.10.1; (2) prototype tests, described in Article 15.10.2; and (3) quality control tests, described in Articles 15.12, 15.14 and 15.15.

15.10.1 System Characterization Tests

The fundamental properties of the isolation system shall be evaluated by testing prior to its use. The purpose of system characterization tests is to substantiate the properties of individual isolator units as well as the behavior of an isolation system. Therefore, these tests include both component tests of individual isolator units and shake table tests of complete isolation systems.

At a minimum, these tests shall consist of

- Tests of individual isolator units in accordance with the National Institute of Standards and Technology (NIST) guidelines or the Highway Innovative Technology Evaluation Center (HITEC) guidelines.
- Shaking table tests at a scale no less than 1/4 full scale. Scale factors must be well-established and approved by the engineer.

15.10.1.1 Low-Temperature Test

If the isolators are for low-temperature areas, then Test 6 specified in section 15.10.2 shall be performed at temperatures of 20, 5, −5, or −15 degrees F (−7, −15, −21, or −26 degrees C) for temperature zones A, B, C, and D, respectively.

The specimen shall be cooled for a duration not less than the maximum number of consecutive days below freezing specified in Table 14.7.5.2-2.

15.10.1.2 Wear and Fatigue Tests

Wear or travel and fatigue tests are required to account for movements resulting both from imposed thermal displacements and live load rotations. Thermal displacements and live load rotations shall correspond to at least 30 years of expected movement. Tests shall be performed at the design contact pressure at 68 degrees F ± 15 degrees (20 degrees C ±8 degrees). The rate of application shall not be less than 2.5 inches/minute (63.5 mm/minute). As a minimum, the following displacements shall be used for the test:

- Bearings: 1 mile (1.6 km)
- Dampers (attached to the web at the neutral axis): 1 mile (1.6 km)
- Dampers (attached to the girder bottom): 2 miles (3.2 km)

Additional wear or travel and fatigue will occur in long structures with greater thermal movements, high traffic counts, and lively spans.

If the isolator units are for low-temperature areas, then 10 percent of the test shall be performed at temperatures of 20, 5, −5, or −15 degrees F (−7, −15, −21, −26 degrees C) for temperature zones A, B, C, and D, respectively.

In lieu of the low-temperature test criteria, the components may be tested for a cumulative travel of twice the calculated service displacements or twice the values above when approved by the engineer.

15.10.2 Prototype Tests

The deformation characteristics and damping values of the isolation system used in the design and analysis shall be verified by prototype tests. Tests on similarly sized isolator units may be used to satisfy the requirements of this section. Such tests must validate design properties that can be extrapolated to the actual sizes used in the design.

Prototype tests shall be performed on a minimum of two full-size specimens of each type and size similar to that used in the design. The test specimens shall include the elastic restraint system

if such a system is used in the design. Prototype test specimens may be used in construction, if they have the specified stiffness and damping properties and they satisfy the project quality control tests after having successfully completed all prototype tests. All sacrificial elements shall be replaced prior to use.

Reduced-scale prototype specimens will only be allowed when full-scale specimens exceed the capacity of existing testing facilities and approval is granted by the engineer of record.

If reduced-scale prototype specimens are used to quantify properties of isolator units, specimens shall be geometrically similar and of the same type and material. The specimens shall also be manufactured with the same processes and quality as full-scale prototypes, and shall be tested at a frequency that represents full-scale prototypes.

The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load similar to the typical or average dead load on the isolator units of a common type and size. The design displacement for these tests is defined in Article 15.4.

Test 1, Thermal – Three fully reversed cycles of loads at a lateral displacement corresponding to the maximum thermal displacement. The test velocity shall not be less than 0.003 inches per minute.

Test 2, Wind and Braking – Twenty fully reversed cycles between limits of plus and minus the maximum load for a total duration not less than 40 seconds. After the cyclic testing, the maximum load shall be held for 1 minute.

Test 3, Seismic – Three fully reversed cycles of loading at each of the following multiples of the total design displacement: 1.0, 0.25, 0.50, 0.75, 1.0, and 1.25, in the sequence shown.

Test 4, Seismic – 20 cycles of loading at 1.0 times the design displacement. The test shall be started from a displacement equal to the offset displacement.

Test 5, Wind and Braking – Three fully reversed cycles between limits of plus and minus the maximum load for a total duration not less than 40 seconds. After the cyclic testing, the maximum load shall be held for 1 minute.

Test 6, Seismic Performance Verification – Three fully reversed cycles of loading at the total design displacement.

Test 7, Stability Verification – The vertical load-carrying elements of the isolation system shall be demonstrated to be stable under one fully reversed cycle at the displacements given in Article 15.4. In these tests, the combined vertical load of

$$1.2\,D + LL_S + OT \qquad (15.10.2\text{-}1)$$

shall be taken as the maximum downward force, and the combined vertical load of

$$0.8\,D - OT \qquad (15.10.2\text{-}2)$$

shall be taken as the minimum downward force.

- If a sacrificial elastic restraint system is utilized, then its ultimate capacity shall be established by test.

- The prototype and quality control tests shall include all components that comprise the isolation system.

- For systems that are not restrained to perform unidirectionally, Test 6 shall be performed in the direction of loading orthogonal to the original direction of loading. For systems that include unidirectional devices, or those that are sensitive to orthogonal effects, Test 6 shall be repeated at 45 degrees to the primary axis of the unidirectional device.

- The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if there is greater than a plus or minus 15-percent difference in either K_d or Q_d for the test at the design displacement when dynamically tested at any frequency in the range of 0.5 to 1.5 times the inverse of the effective period of the isolated structure.

  If the force-deflection properties of the isolator units are dependent on the rate of

loading, then each set of tests specified in Article 15.10.2 shall be performed dynamically at a frequency equal to the inverse of the effective period of the isolated structure. If the test can not be performed dynamically, then a l factor must be established that relates properties K_d or Q_d determined at the actual speed of testing with the dynamic velocities in accordance with Article 15.5.2.1.

15.10.3 Determination of System Characteristics

(a) The force-deflection characteristics of the isolation system shall be based on the cyclic load test results for each fully reversed cycle of loading.

(b) The effective stiffness of an isolator unit shall be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F_p - F_n}{\Delta_p - \Delta_n} \qquad (15.10.3\text{-}1)$$

where Δ_p and Δ_n are the maximum positive and maximum negative test displacements, respectively, and F_p and F_n are the maximum positive and maximum negative forces at instance of displacements Δ_p and Δ_n, respectively.

(c) Equivalent Damping. The equivalent viscous damping ratio (ß) of the isolation system shall be calculated as

$$\beta = \frac{1}{2\pi} \times \frac{\text{Total EDC Area}}{\sum\left(k_{eff}\Delta_i^2\right)} \qquad (15.10.3\text{-}2)$$

The total EDC area shall be taken as the sum of the areas of the hysteresis loops of all isolator units. The hysteresis loop area of each isolator unit shall be taken as the minimum area of the three hysteresis loops established by the cyclic tests in Test 3 of Article 15.10.2 at a displacement amplitude equal to the design displacement.

15.10.3.1 System Adequacy

The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied:

- The force-deflection plots, excluding any viscous damping component, of all tests specified in Article 15.10.2 show a positive incremental force-carrying capacity consistent with the requirements of Article 15.9.2.
- For Test 1, the maximum measured force shall be less than the design value.
- For Tests 2 and 5, the maximum measured displacement shall be less than the design value.
- The average effective stiffness measured in the last three cycles to the total design displacement specified in Test 3 shall lie within 10 percent of the value used in design.

For each test displacement level specified for Test 3, the minimum effective stiffness measured during the three cycles shall not be less than 80 percent of the maximum effective stiffness.

For Test 4, the minimum effective stiffness measured during the specified number of cycles shall not be less than 80 percent of the maximum effective stiffness. At the discretion of the engineer, a larger variation may be accepted, provided that both the minimum and maximum values of effective stiffness are used in the design.

- For Test 4, the minimum EDC measured during the specified number of cycles shall not be less than 70 percent of the maximum EDC. At the discretion of the engineer, a larger variation may be accepted, provided that both the minimum and maximum values of EDC are used in the design.

All vertical load-carrying elements of the isolation system shall remain stable (positive incremental stiffness) at the

displacements specified in Article 15.9.3 for static loads as prescribed for Test 7.

Test specimens shall be visually inspected for evidence of significant deterioration. If any deterioration exists, then the adequacy of the test specimen shall be determined by the engineer.

15.11 ELASTOMERIC BEARINGS

15.11.1 General

The following shall be considered supplemental to Section 14 of the LRFD provisions.

Elastomeric bearings utilized in implementing seismic isolation design shall be designed by the procedures and specifications given in the following subsections. Additional test requirements for seismic isolation bearings are given in Article 15.12. The design procedures are based on service loads excluding impact. The elastomeric bearings must be reinforced using steel reinforcement. Fabric reinforcement is not permitted.

15.11.2 Shear Strain Components for Isolation Design

The various components of shear strain in the bearing shall be computed as follows:

- Shear strain (γ_c) due to compression by vertical loads is given by

$$\gamma_c = \frac{3SP}{2A_r G(1+2\bar{k}S^2)} \qquad (15.11.2\text{-}1)$$

if $S \leq 15$, or

$$\gamma_c = \frac{3P(1+8G\bar{k}S^2/K)}{4G\bar{k}SA_r} \qquad (15.11.2\text{-}2)$$

if $S > 15$,

where K is the bulk modulus of the elastomer. In absence of measured data, K may be taken as 300,000 psi (2,000 MPa). The shape factor S shall be taken as the plan area of the elastomer layer divided by the area of perimeter free to bulge.

- Shear strain ($\gamma_{s,s}$) due to imposed non-seismic lateral displacement is given by

$$\gamma_{s,s} = \frac{\Delta_s}{T_r} \qquad (15.11.2\text{-}3)$$

- Shear strain ($\gamma_{s,eq}$) due to earthquake-imposed lateral displacement is given by

$$\gamma_{s,eq} = \frac{\Delta_t}{T_r} \qquad (15.11.2\text{-}4)$$

- Shear strain (γ_r) due to rotation is given by

$$\gamma_r = \frac{B_d^2\theta}{2t_i T_r} \qquad (15.11.2\text{-}5)$$

The design rotation (θ) shall include the rotational effects of DL, LL, and construction.

15.11.3 Load Combinations

Elastomeric bearings shall satisfy

$$\gamma_c \leq 2.5 \qquad (15.11.3\text{-}1)$$

$$\gamma_c + \gamma_{s,s} + \gamma_r \leq 5.0 \qquad (15.11.3\text{-}2)$$

$$\gamma_c + \gamma_{s,eq} + 0.5\,\gamma_r \leq 5.5 \qquad (15.11.3\text{-}3)$$

15.12 ELASTOMERIC BEARINGS – CONSTRUCTION

15.12.1 General Requirements

The following shall be considered supplemental to article 18.2 of the AASHTO Standard Specifications (Division II). The provision of Article 15.12.2 replaces those in articles 18.2.7.6, 18.2.7.7, and 18.2.7.8 of the AASHTO Standard Specifications (Division II).

The layers of elastomeric bearings used in seismic isolation shall be integrally bonded during vulcanization. Cold bonding is not allowed.

15.12.2 Quality Control Tests

The following quality control tests shall also be performed on elastomeric bearings.

15.12.2.1 Compression Capacity

A 5-minute sustained proof load test shall be conducted on each bearing. The compressive load for the test shall be 1.5 times the maximum (dead load plus live load). If bulging suggests poor laminate bond, the bearing shall be rejected.

15.12.2.2 Combined Compression and Shear

All bearings shall be tested in combined compression and shear. The bearings may be tested in pairs. The compressive load shall be the average dead load of all bearings of that type, and the bearings shall be subjected to five fully reversed cycles of loading at the larger of the total design displacement or 50 percent of the elastomer thickness.

For each bearing, the effective stiffness and EDC shall be averaged over the five cycles of the test. For each group of similar bearings of the same type and size, the effective stiffness and EDC shall be averaged. The results shall not differ from the design values by more than the limits given in Table 15.12.2.2-1.

Table 15.12.2.2-1

	K_{eff}	EDC
Individual Bearings	±20%	–25%
Average of Group	±10%	–15%

15.12.2.3 Acceptance Criteria

After quality control testing, all bearings shall be visually inspected for defects. The following faults shall be cause for rejection:

- Lack of rubber-to-steel bond.
- Laminate placement fault.
- Surface cracks on the rubber that are wider or deeper than 2/3 of the rubber cover thickness.
- Permanent deformation.

15.13 SLIDING BEARINGS – DESIGN

15.13.1 General

Sliding bearings used in isolation systems may use flat or curved surfaces.

15.13.2 Materials

15.13.2.1 PTFE Bearing Liners

All PTFE surfaces, other than guides, shall satisfy the requirements specified herein. The PTFE bearing liner shall be made from virgin PTFE resin satisfying the requirements of ASTM D1457. It may be fabricated as unfilled sheet, filled sheet, or fabric woven from PTFE and other fibers.

Unfilled sheets shall be made from PTFE resin alone. Filled sheets shall be made from PTFE resin uniformly blended with glass fibers, carbon fibers, or other chemically inert reinforcing fibers.

Sheet PTFE may contain dimples to act as reservoirs for lubricant. Their diameter shall not exceed 0.32 inch (8 mm) at the surface of the PTFE and their depth shall be not less than 0.08 inch (2 mm) and not more than half the thickness of the PTFE. The reservoirs should cover more than 20 percent, but less than 30 percent of the contact surface. Dimples should not be placed to intersect the edge of the contact area. Lubricant shall be silicone grease, effective to –30½ F (–34º C). Silicone grease shall conform to Military specification MIL-S-8660.

15.13.2.2 Other Bearing Liner Materials

Other materials may be used for the bearing liner if test results demonstrate a stable long-term coefficient of friction, chemical stability, and wear resistance in accordance with Article 15.10.1.2, and are approved by the engineer.

15.13.2.3 Mating Surface

Mating surfaces shall be stainless steel (welded overlay, solid, or sheet metal). Stainless steel shall have a corrosion resistance and strength equal to or exceeding type 304, conforming to ASTM A167/A264. The average surface roughness shall not exceed 32 micro inches (0.8

micro meters) R_a (arithmetic average) as determined by procedures described in ANSI/ASME B46.1-1985 (ASME, 1985).

15.13.3 Geometry

15.13.3.1 Minimum Thickness

15.13.3.1.1 PTFE Bearing Liner

The minimum thickness for PTFE shall be at least 0.0625 inch (1.6 mm) after compression. Recessed sheet PTFE shall be at least 0.1875 inch (4.8 mm) thick when the maximum dimension of the PTFE is less than or equal to 24.0 inches (610 mm), and 0.25 inch (6.4 mm) when the maximum dimension of the PTFE is greater than 24.0 inches (610 mm). Woven fabric PTFE shall have, after compression, a minimum thickness of 0.0625 inch (1.6 mm) and a maximum thickness of 0.125 inch (3.2 mm).

15.13.3.1.2 Other Bearing Liner Materials

The minimum thickness for all other bearing liners shall be determined by conducting wear tests in accordance with Article 15.10.1.2.

15.13.3.2 Mating Surface

The thickness of the stainless steel mating surface sheet shall be at least 16 gauge when the maximum dimension of the surface is less than or equal to 12.0 inches (305 mm), and at least 13 gauge when the maximum dimension is larger than 12.0 inches (305 mm) and less than or equal to 36.0 inches (915 mm). When the maximum dimension is larger than 36.0 inches (915 mm), the thickness of the stainless steel mating surface shall be verified by performance of suitable system characterization tests.

The minimum thickness of stainless steel weld overlays shall be 3/32 inch (2.4 mm) thick after welding, grinding, and polishing.

15.13.3.3 Displacement Capacity

The mating surface dimensions shall be large enough to ensure that the sliding surface does not come into contact with the edge of the mating surface at the total design displacement plus the offset displacement.

15.13.4 Loads and Stresses

15.13.4.1 Contact Pressure

Contact stresses for bearing liners shall be established by testing. Test pressures shall be at least 110 percent of the value used in design and must satisfy the wear requirements in Article 15.10.1.2. As a minimum, 50 percent of the usable bearing liner thickness must remain after completion of the wear test. Allowable contact stresses for PTFE liners tabulated in Table 15.13.4.1-1 may be used without completing the wear test, provided that the stainless steel mating surface has a surface roughness less than 20 micro inches (0.5 micro meter) Ra.

Table 15.13.4.1-1
Allowable Average Contact Stress for PTFE

Material	Allowable Contact Stress					
	Service Loads				Seismic Loads	
	Average Stress		Edge Stress		Average Stress	
	ksi	MPa	ksi	MPa	ksi	MPa
Unfilled sheets (recessed)	3.5	24	5.0	34	6.0	41
Filled sheets (recessed)	3.5	24	5.0	34	6.0	41
Woven PTFE fiber over a metallic substrate	3.5	24	10.0	69	6.0	41

15.13.4.2 Coefficient of Friction

15.13.4.2.1 Service Coefficient of Friction

The service limit state coefficient of friction of the PTFE sliding surface shall be taken as specified in Table 15.13.4.2.1-1. Intermediate values may be determined by interpolation. The coefficient of friction shall be determined by using the stress level associated with the service load combination specified in Table 3.4.1-1. Different

values may be used if verified by tests and adjusted by the appropriate λ values in accordance with Article 15.5.

Table 15.13.4.2.1-1 Service Coefficients of Friction

Type of Surface	Temp.		Average Bearing Stress				
			0.5	1.0	2.0	≥3.0	ksi
	°F	°C	3.5	6.9	13.8	20.7	MPa
Dimpled lubricated PTFE sheets	68	20	0.04	0.03	0.025	0.02	
	–13	–25	0.06	0.045	0.04	0.03	
	–49	–45	0.10	0.075	0.06	0.05	
Unfilled PTFE sheets	68	20	0.08	0.07	0.05	0.03	
	–13	–25	0.20	0.18	0.13	0.10	
	–49	–45	0.20	0.18	0.13	0.10	
Filled PTFE sheets	68	20	0.24	0.17	0.09	0.06	
	–13	–25	0.44	0.32	0.25	0.20	
	–49	–45	0.65	0.55	0.45	0.35	
Woven PTFE fiber	68	20	0.08	0.07	0.06	0.045	
	–13	–25	0.20	0.18	0.13	0.10	
	–49	–45	0.20	0.18	0.13	0.10	

Service coefficients of friction for other surface finishes, stresses, and bearing liners shall be established by testing. The testing procedures and results shall be subject to the approval of the engineer.

15.13.4.2.2 Seismic Coefficient of Friction

The seismic coefficient of friction may be determined from the area under the force displacement loops of three cycles divided by the total travel distance and vertical load (Q_d/vertical load).

15.13.5 Other Details

15.13.5.1 Bearing Liner Attachment

All sheet PTFE shall be recessed for one-half of its thickness and bonded into a metal backing plate.

All bearing liners shall be attached to resist a shear force of 0.15 times the applied compressive force or 2 times Q_d, whichever is greater.

15.13.5.2 Mating Surface Attachment

The mating surface for the bearing liner shall be attached to a backing plate by welding or other suitable means in such a way that it remains free of undulations and in full contact with its backing plate throughout its service life. The attachment shall include an effective moisture seal around the entire perimeter of the mating surface to prevent interface corrosion. The attachment shall be capable of resisting the maximum friction force that can be developed by the bearing under service limit state and seismic load combinations. The welds used for the attachment shall be clear of the contact and sliding area of the bearing liner.

15.13.6 Materials for Guides

Bearing guides may be made from materials not described in Article 15.13.2. The materials used shall have sufficient strength, stiffness, and resistance to creep and decay to ensure the proper functioning of the guide throughout its design life.

15.14 SLIDING BEARINGS – CONSTRUCTION

15.14.1 General Requirements

Isolator units that use sliding bearings shall be constructed in accordance with the applicable provisions of articles 18.4 and 18.8.2 of the AASHTO Standard Specifications (Division II).

15.14.2 Quality Control Tests

The following quality control tests shall also be performed on sliding isolation bearings.

15.14.2.1 Compression Capacity

A 5-minute sustained proof load test shall be conducted on each bearing. The compressive load for the test shall be 1.5 times the maximum (dead load plus live load). If flow of the bearing liner suggests inadequate bonding, or it leaves a permanent deformation in the mating surface, the bearing shall be rejected.

15.14.2.2 Combined Compression and Shear

All bearings shall be tested in combined compression and shear. The bearings may be

tested in pairs. The compressive load shall be the average dead load of all bearings of that type, and the bearings shall be subjected to five fully reversed cycles of loading at the total design displacement.

For each bearing, the effective stiffness and EDC shall be averaged over the five cycles of the test. For each group of similar bearings of the same type and size, the effective stiffness and EDC shall be averaged. The results shall not differ from the design values by more than the limits given in table 15.12.2.2-1.

15.14.2.3 Acceptance Criteria

After quality control testing, all bearings shall be visually inspected and, if applicable, disassembled and inspected for defects. The following faults shall be cause for rejection:

(1) Lack of bearing-liner-to-metal bond.

(2) Scoring of stainless steel plate.

(3) Permanent deformation.

(4) Leakage.

15.15 OTHER ISOLATION SYSTEMS

15.15.1 Scope

All isolation units or systems that contain a flexible element, restoring force capacity, and energy dissipation capacity, and that are not covered in Articles 15.11 to 15.14 of this specification, shall be subject to the requirements of this section and approved by the engineer.

Isolation bearings that depend on a metal roller element for lateral displacement shall satisfy the requirements of Article 14.7.

Acceptance of the system shall be based on satisfying the requirements of Articles 15.15.2 through 15.15.6.

Materials used for contact surfaces, such as sliding or rolling elements, shall be selected so as to provide the least possible change in those properties over time.

15.15.2 System Characterization Tests

The characteristics of the isolation system that are used in design shall be verified by tests and approved by the engineer. At a minimum, the following tests shall be conducted:

- Lateral load tests to determine properties and capacities in accordance with tests prescribed in the NIST report (National Institute of Standards and Technology 1996; ASCE Standards Committee on Testing of Base Isolation Systems 1996) or HITEC report (Highway Innovation Technology Center 1996).
- Shaking table tests at a scale no less than 1/4 full scale. Scale factors must be well-established and approved by the engineer.
- Tests to investigate the variations in system properties and their effects on response. At a minimum, the effects on temperature, rate-dependency, prior loading (including wear), and environmental effects shall be investigated. Values for λ_{min} and λ_{max}, similar to those defined in Article 15.5, shall be developed from these tests.

In addition to the foregoing test data, information from previous field experience in other applications may be used to demonstrate the system characteristics.

For all tests, no adjustments to the system may be made except those that are explicitly included in the maintenance plan, which must be given to the engineer prior to the start of prototype testing.

15.15.3 Design Procedure

A complete, rational design procedure for the isolation system shall be provided to the engineer prior to the start of the prototype testing defined in section 18.5. This procedure shall include

- the basis for the selection of the limiting material stresses, deformations, or other critical response quantities;
- the method for predicting the cyclic load deformation relationship of the system; and
- the method for predicting the stability limit of the system.

At least one design example shall be submitted with the design procedure, including the calculations for obtaining the maximum force response and maximum displacement response.

15.15.4 Fabrication, Installation, Inspection, and Maintenance Requirements

All special requirements for fabrication, installation, inspection, and maintenance shall be submitted, in writing, to the engineer prior to the start of prototype testing. At a minimum, these shall include

- materials to be used and the specifications they must satisfy,
- any special material testing requirements,
- fabrication sequence and procedures,
- fabrication tolerances and surface finish requirements,
- any special handling requirements,
- installation procedures and tolerances, and
- maintenance requirements, including a schedule for replacement of any components, for the lifetime of the system.

15.15.5 Prototype Tests

Prototype testing shall be conducted for each job in order to demonstrate that the design achieves the performance requirements set out in the job specifications. Insofar as possible, the tests shall conform to those defined in Article 15.10.2. The engineer may, at his or her discretion, require additional tests to verify particular characteristics of the system.

Prior to the start of testing, design values for critical response quantities shall be submitted to the engineer, and the engineer shall establish criteria for accepting the system on the basis of the prototype tests. At a minimum, those criteria shall include permissible variations from the design values of the resistance and energy dissipation at critical displacements, velocities, or accelerations.

15.15.6 Quality Control Tests

Quality control testing shall be conducted on every bearing. Test requirements and acceptance requirements shall be established by the engineer.

15.15.6.1 Compression Capacity

A 5-minute sustained proof load test shall be conducted on each bearing. The compressive load for the test shall be 1.5 times the maximum (dead load plus live load).

15.15.6.2 Combined Compression and Shear

All bearings shall be tested in combined compression and shear. The bearings may be tested in pairs. The compressive load shall be the average dead load of all bearings of that type, and the bearings shall be subjected to five fully reversed cycles of loading at the total design displacement.

15.15.6.3 Acceptance Criteria

Acceptance criteria for requirements specified in this section shall be determined by the engineer.

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Appendix 15A
ISOLATION DESIGN PARAMETERS

15A.1 SLIDING ISOLATION SYSTEMS

The λ factors on sliding systems are applied to Q_d.

15A.1.1 Factors for Establishing λ_{min}

$\lambda_{min} = 1.0$

15A.1.2 Factors for Establishing λ_{max}

15A.1.2.1 $\lambda_{max,a}$

	Unlubricated PTFE		Lubricated PTFE		Bimetallic Interfaces	
Condition / Environment	Sealed	Unsealed	Sealed	Unsealed	Sealed	Unsealed
Normal	1.1	1.2	1.3	1.4	2.0	2.2
Severe	1.2	1.5	1.4	1.8	2.2	2.5

Notes:

- Values are for 30-year exposure of stainless steel. For chrome-plated carbon steel, multiply values by 3.0.
- Unsealed conditions assumed to allow exposure to water and salt, thus promoting further corrosion.
- Severe environments include marine and industrial environments.
- Values for bimetallic interfaces apply for stainless steel and bronze interfaces.

15A.1.2.2 $\lambda_{max,v}$

Established by test.

C15A.1 SLIDING ISOLATION SYSTEMS

Woven PTFE shall be treated as unlubricated PTFE.

C15A.1.2.1 $\lambda_{max,a}$

The aging factor is based on friction data for rough stainless steel plates with PTFE or other materials. It is assumed that the plate has uniform corrosion, which creates a rougher sliding surface.

For bimetallic interfaces, the factor is based on data for stainless steel and leaded bronze interfaces (Lee 1993). Increases in friction due to stress effects have been observed in the absence of corrosion.

15A.1.2.3 $\lambda_{max,c}$

	Unlubricated PTFE	Lubricated PTFE	Bimetallic Interfaces
Sealed with stainless steel surface facing down	1.0	1.0	1.0
Sealed with stainless steel surface facing up*	1.1	1.1	1.1
Unsealed with stainless steel surface facing down	1.1	3.0	1.1
Unsealed with stainless steel surface facing up	Not Allowed	Not Allowed	Not Allowed

* Use factor of 1.0 if bearing is galvanized or painted for 30-year lifetime.

15A.1.2.4 $\lambda_{max,tr}$

Cumulative Travel		Unlubricated PTFE*	Lubricated PTFE	Bimetallic Interfaces
ft	m			
<3300	1005	1.0	1.0	To be established by test
<6600	2010	1.2	1.0	To be established by test
>6600	2010	To be established by test	To be established by test	To be established by test

* Test data based on 1/8-inch sheet, recessed by 1/16 inch and bonded.

15A.1.2.5 $\lambda_{max,t}$

Minimum Temp for Design		Unlubricated PTFE	Lubricated PTFE	Bimetallic Interfaces
°F	°C			
70	21	1.0	1.0	To be established by test
32	0	1.1	1.3	
14	–10	1.2	1.5	
–22	–30	1.5	3.0	

C15A.1.2.3 $\lambda_{max,c}$

Values shown in the table assume that the sliding interface will not be separated.

Sealed bearings shall have a protective barrier to prevent contamination of the sliding interface. The protective barrier shall remain effective at all service load displacements.

15A.2 ELASTOMERIC BEARINGS

The λ factors on elastomeric systems are applied to K_d and Q_d.

15A.2.1 Factors for Establishing λ_{min}

$\lambda_{min} = 1.0$

15A.2.2 Factors for Establishing λ_{max}

15A.2.2.1 $\lambda_{max,a}$

The aging factor depends significantly on the rubber compound. As a general rule, it is expected that this factor is close to unity for low-damping natural rubber and to be more for high-damping rubber.

C15A.2 ELASTOMERIC BEARINGS

Elastomeric bearings are produced in a variety of compounds (particularly high-damping rubber bearings), so that a vast number of experiments are needed to establish the relevant λ factors.

Moreover, available data on the behavior of rubber bearings are limited to a small range of parameters, usually established for a particular application. Even in the case of lead-rubber bearings (which found wide application in bridges), data on the effect of temperature are scarce and include one bearing tested in New Zealand at temperatures of –31, 5, 64, and 113° F (–35, –15, 18, and 45°C); one tested in the United States (Kim et al. 1996) at temperatures of – 18 and 68°F (–28 and 20°C); and one in Japan tested at –4 and 68°F (–20 and 20°C).

The factors listed herein are based on the available limited data. In some cases the factors could not be established and need to be determined by test.

It is assumed that elastomeric bearings are tested when unscragged at temperature of 70ºF ±10ºF (21°C ±5°C) to establish the relevant properties. Testing is performed at the design displacement and a frequency less than the inverse of period T_{eff}. The first cycle loop is used to establish the maximum value of effective stiffness (k_{max}) and area under loop (A_{max}). The minimum values (as a result of scragging) are established as the average of three cycles to be k_{min} and A_{min}.

It is also assumed here that scragging is a reversible phenomenon – that is, rubber recovers after some time its initial, unscragged properties. High-damping rubber bearings may exhibit significant difference between unscragged and scragged properties, although this difference depends entirely on the rubber compound.

C15A.2.2.1 $\lambda_{max,a}$

The relationship between aging and scragging was assumed in the table. However, such a relationship has not been verified by testing.

	K_d	Q_d
Low-Damping natural rubber	1.1	1.1
High-Damping rubber with small difference between scragged and unscragged properties	1.2	1.2
High-Damping rubber with large difference between scragged and unscragged properties	1.3	1.3
Lead	–	1.0
Neoprene	3.0	3.0

Notes:

- A large difference is one in which the unscragged properties are at least 25 percent more than the scragged ones.

15A.2.2.2 $\lambda_{max,v}$

Established by test.

15A.2.2.3 $\lambda_{max,c}$

$\lambda_{max,c} = 1$

15A.2.2.4 $\lambda_{max,tr}$

Established by test.

15A.2.2.5 $\lambda_{max,t}$

Minimum Temp for Design		Q_d			K_d		
°F	ºC	HDRB[1]	HDRB[2]	LDRB[2]	HDRB[1]	HDRB[2]	LDRB[2]
70	21	1.0	1.0	1.0	1.0	1.0	1.0
32	0	1.3	1.3	1.3	1.2	1.1	1.1
14	–10	1.4	1.4	1.4	1.4	1.2	1.1
–22	-30	2.5	2.0	1.5	2.0	1.4	1.3

HDRB = High-Damping Rubber Bearing
LDRB = Low-Damping Rubber Bearing

1. Large difference (at least 25%) between scragged and unscragged properties.
2. Small difference (<25%) between scragged and unscragged properties.

15A.2.2.6 $\lambda_{max,scrag}$

Q_d			K_d		
LDRB	HDRB with β_{eff} - 0.15	HDRB with $\beta_{eff} > 0.15$	LDRB	HDRB with β_{eff} - 0.15	HDRB with $\beta_{eff} > 0.15$
1.0	1.2	1.5	1.0	1.2	1.8

C15A.2.2.5

Values for lead-rubber bearings are based on grade 3 natural rubber.

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