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NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provisions

2026 Edition

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NEHRP (National Earthquake Hazards Reduction Program)

Recommended Seismic Provisions

for New Buildings and Other Structures

2026 Edition

Volume I: Part 1 Provisions and Part 2 Commentary

This was prepared by the National Institute of Building Sciences Building Seismic Safety Council in cooperation with and with the support of the NEHRP Agencies: the Federal Emergency Management Agency (FEMA), the National Institute of Standards and Technology (NIST), the National Science Foundation (NSF), and the U.S. Geological Survey (USGS).

**NATIONAL INSTITUTE OF BUILDING SCIENCES
BUILDING SEISMIC SAFETY COUNCIL**

Washington, D.C.

2026

Established by Congress in 1974, the **National Institute of Building Sciences** (NIBS) is a nonprofit, nonpartisan, non-governmental organization dedicated to improving the built environment. NIBS serves as a trusted, neutral partner, bringing together government, industry, academia, and other stakeholders to solve complex challenges and advance safer, more resilient, and higher-performing buildings and infrastructure.

NIBS established the **Building Seismic Safety Council** (BSSC) in 1979 as an expert council under the NIBS organization to provide a forum-based mechanism for addressing the complex regulatory, technical, social, and economic issues involved in developing and promulgating building earthquake hazard mitigation regulatory provisions that are national in scope. By bringing together, under NIBS and the BSSC, the needed expertise and relevant public and private interests, it was believed that issues related to the seismic safety of the built environment could be resolved and jurisdictional problems overcome through authoritative guidance and assistance backed by broad consensus.

The NIBS BSSC is a voluntary membership body representing a wide variety of building community interests, and as a voluntary council under NIBS, is independent and non-partisan. NIBS created the BSSC with the fundamental purpose of enhancing public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings.

For further information on Building Seismic Safety Council activities and products, see the Council's website <https://nibs.org/bssc> (or contact the Building Seismic Safety Council, National Institute of Building Sciences, 2121 K Street NW, Suite 800, Washington, DC 20037; phone 202-289-7800; e-mail nibs@nibs.org).

This resource document can be downloaded in pdf form from the National Institute of Building Sciences website at <https://nibs.org/?p=28276>. The National Institute of Building Sciences (NIBS) and its Building Seismic Safety Council (BSSC) caution users of this Provisions document to be alert to patent and copyright concerns, especially when applying prescriptive requirements.

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FOREWORD TO THE 2026 NEHRP RECOMMENDED SEISMIC PROVISIONS

The National Earthquake Hazards Reduction Program (NEHRP) is proud to present the 2026 *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (Provisions). Established in 1977, NEHRP has remained steadfast in providing the leadership and technical resources needed to reduce human, economic, and societal losses caused by earthquakes in the U.S. Now, as the 50th anniversary of NEHRP approaches, this 2026 edition represents the current state of knowledge derived from nearly five decades of multidisciplinary research, post-earthquake reconnaissance, and rigorous engineering consensus. Ultimately, it translates the most advanced scientific understanding of seismic hazards and risk reduction into practical, actionable analysis and design recommendations for new buildings and other structures.

As the landscape that fueled the need for this resource document has been ever-evolving since its first edition in 1985, so too must our approach to seismic safety. This evolution now provides NEHRP a valuable opportunity to evaluate the modern landscape and societal needs in order to develop a resource document poised for the future. The economic and social disruptions caused by recent significant global earthquakes underscore that simply surviving a seismic event is no longer enough for our growing communities; our built environment must be designed and constructed to support communities that withstand, respond to, and recover from these unavoidable natural forces in a timely manner.

Crucially, this edition of the Provisions marks a pivotal shift toward a resilience-based design perspective, introducing foundational frameworks designed to support communities aiming to implement functional recovery targets into their local building practices. Building on this paradigm shift, the document incorporates several other critical advancements. It integrates the latest United States Geological Survey (USGS) National Seismic Hazard Models, which utilize updated fault data, ground motion models, and site amplification factors to provide the most precise characterization of seismic hazard to date. Furthermore, the Provisions offer refined guidance on performance-based seismic design, advanced lateral force-resisting systems, and nonstructural component bracing.

The development of the Provisions is a testament to the power of public-private collaboration. This document is the result of the tireless efforts of the Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences (NIBS) with support from the NEHRP agencies: the Federal Emergency Management Agency (FEMA), the National Institute of Standards and Technology (NIST), the U.S. National Science Foundation (NSF), and the USGS. We extend our deepest gratitude to the hundreds of volunteer subject matter experts—structural and geotechnical engineers and researchers, seismologists, building officials, and others—who dedicated countless hours to the Provisions Update Committee, its specialized Issue Teams, and the Functional Recovery Task Committee as well as the associated Resilient Design Coalition. Lastly, this document is only possible with the substantial support of NIBS and the vision of its leaders, staff, and members of its various boards of directors.

The adoption of these recommendations into the American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI) standard ASCE/SEI 7: *Minimum Design Loads and Associated Criteria for*

Buildings and Other Structures remains a vital bridge between scientific discovery and public safety. On behalf of NEHRP, the BSSC offers this 2026 edition as an essential resource for engineers, codes and standards developers, and policymakers, unified by our shared commitment to building a safer, more resilient nation.

*Program Coordination Working Group
National Earthquake Hazards Reduction Program*

PREFACE AND ACKNOWLEDGEMENTS

The 2026 NEHRP Recommended Seismic Provisions demonstrate the significant impact of the mission that Congress entrusted to the National Institute of Building Sciences (NIBS). This work brings together experts from the public and private sectors of our nation to address critical challenges to our built environment. By advancing seismic safety, resilience, and building performance, the Provisions demonstrate how NIBS turns technical excellence, innovative insights, collaboration, consensus, and building science into findings, recommendations, and solutions that help safeguard people, communities, and the places where we live and work.

Through the Building Seismic Safety Council (BSSC), a longstanding and prominent NIBS council established in 1979, engineers, researchers, code and standards professionals, government representatives, and other experts from across the building community come together as a national forum for addressing the technical, regulatory, social, and economic issues associated with seismic safety. This work has advanced seismic design practices and provided the building community with key resources to protect individuals and communities from earthquake hazards.

The 2026 edition represents five decades of leadership in seismic safety. We are honored to partner with the NEHRP Agencies. In this edition, we took up the opportunity to commit our resources and our funding to finalize and publish the 2026 NEHRP Provisions, ensuring this vital work completes this cycle and continues into the future.

This publication would not be possible without the expertise and dedication of the many individuals and organizations that partner with and participate directly with NIBS and through the NIBS Building Seismic Safety Council. We are grateful to the 2026 Provisions Update Committee, BSSC member organizations, Issue Teams, technical experts, and contributors who brought their knowledge, insights, and perspectives to this effort. We also recognize the longstanding contributions of the federal agencies participating in the National Earthquake Hazards Reduction Program to advancing earthquake risk reduction nationwide.

The 2026 NEHRP Recommended Seismic Provisions represents America's active and continuing commitment to a safer and more resilient built environment that benefits lives, livelihoods, and communities across the nation.

George Guszcza
President and CEO

The 2026 *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (*NEHRP Provisions*) marks the 11th edition of this landmark publication since the creation of the National Earthquake Hazards Reduction Program (NEHRP) in 1977. Over its history, the *NEHRP Provisions* has become one of the nation's most influential technical resources for advancing earthquake-resistant design and construction. It provides a forum through which research findings, lessons from practice, and emerging technologies can be evaluated and translated into recommendations for national standards and model building codes.

The National Institute of Building Sciences (NIBS) and the NIBS Building Seismic Safety Council (BSSC) are proud to present the 2026 edition. Like earlier editions, the 2026 *NEHRP Provisions* introduces significant recommended changes and advancements intended to improve the seismic performance of new buildings and other structures. The recommendations build upon ASCE/SEI 7-22 and address a broad range of subjects, including seismic hazard characterization and mapping, site response, liquefaction, surface fault rupture, soil-structure interaction, foundations, diaphragm design, nonlinear analysis, new structural systems, and post-earthquake functional recovery.

The 2026 edition consists of three coordinated parts. Part 1 presents recommended modifications, additions, and exceptions to the seismic requirements of ASCE/SEI 7-22. Part 2 provides the commentary that explains the technical basis for the recommended changes and offers broader educational guidance. Part 3 presents resource papers on new developments, technical issues, and subjects requiring additional study or future standards development. Together, these materials support the continued improvement of seismic design standards and serve as an educational resource for practicing professionals, researchers, educators, students, standards-development organizations, and building officials.

Among the important advancements in this edition are updated national seismic hazard models and ground-motion procedures; revised seismic design category provisions; expanded requirements addressing liquefaction and fault displacement; improvements to diaphragm, foundation, soil-structure interaction, and nonlinear analysis provisions; and recommendations for new structural systems and technologies. The 2026 edition also introduces Appendix A, *Seismic Design Criteria and Other Requirements for Functional Recovery*. This appendix provides an optional framework for designing structures to support the recovery of their basic intended functions within defined periods following an earthquake.

Development of the 2026 *NEHRP Provisions* was conducted through BSSC's consensus process. The Provisions Update Committee (PUC), composed of nationally recognized experts representing the breadth of seismic design disciplines, directed the development of the recommendations and conducted the review, comment-resolution, and approval process. Thirteen Issue Teams examined specific technical subjects, while the Functional Recovery Task Committee and its five topic subcommittees developed the first functional recovery provisions. There were four additional ad hoc task group efforts that focused on specific technical topics outside of the Issue Team efforts. Professional Organizations representing a broad cross-section of the building community participated in the consensus process, and BSSC provided final oversight.

The development of this edition also reflects an important institutional commitment. The Federal Emergency Management Agency (FEMA) supported the *NEHRP Provisions* program for nearly five decades and provided support during much of the 2026 development cycle. Following the conclusion of FEMA’s contract in 2025, the National Institute of Building Sciences committed its own resources to self-fund the completion of the remaining development activities and publish the 2026 edition. This commitment ensured that the extensive work of the committees, Issue Teams, task groups, proposal proponents, reviewers, and volunteers could be brought to completion and made available to the building community.

BSSC wishes to express its sincere appreciation to the federal agency partners of NEHRP: the Federal Emergency Management Agency (FEMA), the National Institute of Standards and Technology (NIST), the National Science Foundation (NSF), and the U.S. Geological Survey (USGS). Their longstanding support of earthquake research, hazard assessment, knowledge implementation, and seismic safety has contributed significantly to the continued advancement of the *NEHRP Provisions*. BSSC also extends its gratitude to NIBS for its commitment to completing and publishing this edition.

Most importantly, BSSC offers its heartfelt thanks to the many professionals who contributed their time, expertise, and judgment to the 2026 update. This publication would not have been possible without the dedication of:

- The voting and corresponding members of the BSSC Provisions Update Committee, especially PUC and Vice Chair Bob Pekelnicky;
- The members and leaders of the thirteen PUC Issue Teams and the ad hoc task groups;
- The voting members of the Functional Recovery Task Committee and the members of its five topic subcommittees;
- Proposal proponents, technical contributors, resource-paper authors, reviewers, and commenters;
- The representatives of the Professional Organizations that participated in the consensus process;
- The members of BSSC and its Executive Committee;
- The representatives and technical experts from FEMA, NIST, NSF, USGS, and other participating organizations; and
- The NIBS staff, technical editors, project managers, and publication team whose work brought the final documents together.

The participation of these individuals and organizations reflects the collaborative, rigorous, and nationally representative process that has long distinguished BSSC. Their collective efforts help translate scientific knowledge, engineering research, professional experience, and lessons learned

into practical recommendations for improving the seismic safety and resilience of the nation's buildings and communities.

With so many individuals contributing to an undertaking of this scope, it is difficult to recognize every participant individually without inadvertently omitting someone whose contribution was essential. A complete listing of the committees, teams, organizations, authors, and volunteers participating in the 2026 cycle is included in the Recognition of Volunteers section of this publication.

The NIBS BSSC extends its sincere appreciation to everyone who contributed to the development, review, completion, and publication of the 2026 *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures*. We are proud to introduce this edition and to continue the longstanding role of the *NEHRP Provisions* in advancing seismic design practice and protecting lives, property, and communities.

John D. Hooper
Vice Chairperson, Building Seismic Safety Council, National Institute of Building Sciences
Chairperson, Provisions Update Committee - 2026

Guide to Using the 2026 NEHRP Provisions

Part 1 of the Provisions provides recommended changes to the seismic requirements of ASCE 7-22, Chapters 11 to 22 and Appendix A. For a given chapter, only those sections with recommended modifications or additions are shown. Therefore, Part 1, Provisions should be used side-by-side with ASCE 7-22 to grasp the full context of each chapter.

Part 2 of the Provisions provides commentary for the recommended changes in Part 1. It is comprised of the new commentary to each proposed change contained in Part 1. The commentary of ASCE/SEI 7-22 for unchanged sections is not reproduced. Therefore, Part 2, Commentary should be used side-by-side with the ASCE/SEI 7-22 Commentary.

Part 3 provides resource papers adopted in the 2026 Provisions cycle. These resource papers are self-contained and not necessarily directly associated with a Part 1 provision. They are not written in standards language format.

The table in the Introduction provides a summary of Part 1 topics along with reference sections in Parts 1 and 2, and any relevant resource paper in Part 3.

These Provisions provide recommended changes to ASCE 7-22. Adopted language in ASCE 7-28 for any recommendation may differ because of deliberations occurring within the ASCE development process. Reference ASCE 7-28 for the latest version to be adopted by a model building code.

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INTRODUCTION

The 2026 edition of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (NEHRP Provisions)* is a consensus-based technical resource document that can be used by model codes and standards organizations, guidelines organizations and the public as the foundation for improved seismic design. The *NEHRP Provisions* have been an essential resource in the advancement of improved seismic design and construction practices. Changes contained in Parts 1 and 2 of the 2026 *NEHRP Provisions* are expected to be considered for adoption by ASCE/SEI 7-2028 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, which will be later considered for adoption by the International Building Code (IBC) 2030.

The *NEHRP Recommended Seismic Provisions* is a major product of the National Earthquake Hazards Reduction Program (NEHRP), and results from a convergence of the efforts of the four NEHRP agencies: Federal Emergency Management Agency (FEMA), National Institute of Standards and Technology (NIST), National Science Foundation (NSF) and U.S. Geological Survey (USGS). It is developed based on extensive results and findings from research projects, problem-focused studies, and post-earthquake investigation reports conducted by various professional organizations, research institutes, universities, material industries, and the four NEHRP agencies.

What is Included in the *NEHRP Provisions*

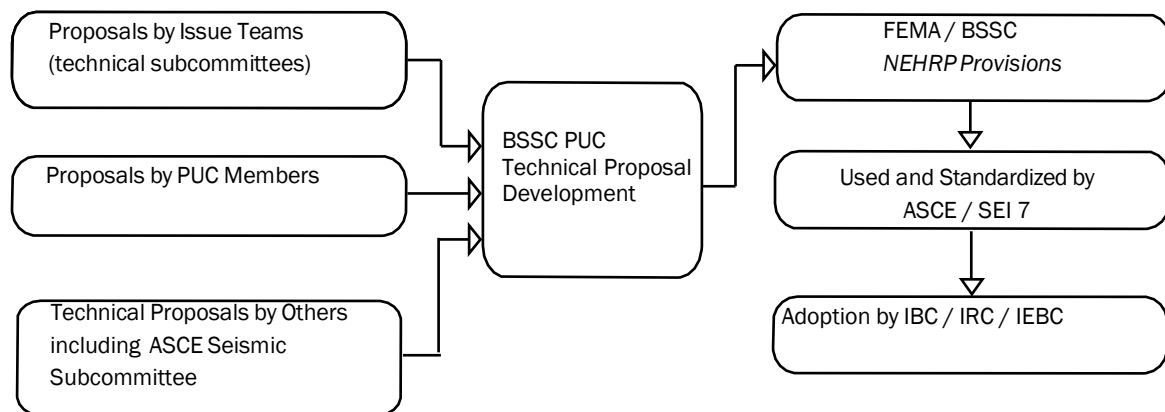
The *NEHRP Provisions* have focused on the adoption of new/significant technologies, which are brought forth as changes to the most recent edition of ASCE/SEI 7, adopted by the Building Seismic Safety Council (BSSC) Provisions Update Committee (PUC) as the reference standard. Consistent with the 2009, 2015, and 2020 editions, the 2026 *NEHRP Provisions* include two volumes:

Volume 1: Part 1, Provisions and Part 2, Commentary: Part 1, *Provisions* provides recommended changes to the seismic requirements of ASCE/SEI 7-22, Chapters 11 to 22 and Appendix A. For a given chapter, only those sections with recommended modifications or additions are shown. Therefore, Part 1, *Provisions* should be used side-by-side with ASCE 7-22 in order to grasp the full context of each chapter. Part 2 *Commentary* provides commentary for the recommended changes in Part 1. It is comprised of the new commentary to each proposed change contained in Part 1. The commentary of ASCE/SEI 7-22 for unchanged sections is not reproduced. Therefore, Part 2, *Commentary* should be used side-by-side with the ASCE/SEI 7-22 *Commentary*.

Volume 2: Part 3, Resource Papers: Part 3, *Resource Papers* introduces new concepts and procedures for experimental use by the design community, researchers, and standards development and code development organizations. These resource papers are self-contained and not necessarily directly associated with Part 1, *Provisions*. They are not written in standards language format. Feedback from these users is encouraged.

How the 2026 NEHRP Provisions are developed

Consistent with the approach used in previous editions, the 2026 *NEHRP Provisions* are developed through a BSSC consensus process conducted by the PUC and BSSC Professional Organizations. In consideration of balancing geographical and design practices, providing expertise in a broad range of subject areas, focusing on key areas of code improvement, and collaborating with national standards and codes, 28 individual experts were selected to serve as the voting members of PUC in the 2026 NEHRP Provisions update cycle. The PUC, with input from the earthquake engineering community, identified technical issues considered most critical for improvement of U.S. seismic design practice, and formed thirteen Issue Teams for developing change proposals to the ASCE/SEI 7 standard, plus a Functional Recovery Task Committee with five topic subcommittees, similar to the Project 17 effort of the 2020 cycle, and four ad hoc task groups. The *NEHRP Provisions* are developed and backed by a broad consensus process. Proposals are developed, vetted, and approved within each of the Issue Teams, then all proposals are officially balloted by the PUC, with all comments and responses reviewed and resolved. Following approval by the BSSC Board of Direction, the proposals are balloted by BSSC Professional Organizations, after which all their comments and responses are reviewed and resolved by the PUC. The process is briefly illustrated by the chart below.



The Role of the NEHRP Provisions in the U.S. Seismic Code Development Process

The Federal Emergency Management Agency (FEMA) supported the NEHRP Provisions program for nearly five decades and provided support during much of the 2026 development cycle. Following the conclusion of FEMA's contract in 2025, NIBS committed its own resources to complete the remaining development activities and publish the 2026 edition.

Over 180 subject matter experts and FEMA, NIST, and USGS representatives contributed to the 2026 *NEHRP Provisions* development, and 37 BSSC Professional organizations participated in the proposal vetting process. A total of 87 technical change proposals were developed and deliberated, with 52 receiving consensus approval and incorporated into the 2026 *NEHRP Provisions* publication. The project participants are presented in the Appendix, and the approved change proposals are summarized in next section.

What is New in the 2026 *NEHRP Provisions*

The 2026 *NEHRP Provisions* have adopted the national standard ASCE/SEI 7-22 Chapter 11-23, including any associated supplements, as its reference standard. Volume I, Parts 1 and 2 consist of changes by the 2026 *NEHRP Provisions* to the ASCE/SEI 7-22 Chapter 11-23 and their commentaries. Topics of the Parts 1 and 2 approved changes are summarized in the table below, along with their relevant ASCE/SEI 7-22 section numbers and commentary section numbers.

Topic of Change Proposals	Brief Summary of the Changes	Related or New Sections of ASCE/SEI 7-22	Related Commentary in ASCE/SEI 7-22
Intent of the NEHRP Provisions	Revises the Intent of the Provisions and its commentary, including the performance objectives for structure collapse, nonstructural damage, hazardous materials, preservation of egress, functionality of Risk Category IV facilities, and repair costs.	INTENT* 1.1	INTENT* 2.1
Adoption of ASCE/SEI 7-22 Chapters 11-23 and Associated Supplements	ASCE/SEI 7-22 is adopted by the PUC as the reference standard and proposals for technical changes are made relative to specific sections of the standard.	All sections of Chapters 11-23 in ASCE/SEI 7-22 without exception	All sections of C11-C23 in ASCE/SEI 7-22
Multi-Period Design Response Spectrum Extrapolation	Corrects the extrapolation of multi-period design response spectral accelerations to periods longer than 10 s (Item 3) and adds the case where T_L is less than 10 s (Item 4).	Section 11.4.5.1	None (no commentary required)
CEUS Site Class DE and E Spectral Acceleration Floor	Revises the design and site-specific spectral acceleration requirements for Site Classes DE and E in the Central and Eastern United States.	Sections 11.4.3, 11.4.5, 11.8.3 and 21.2.3	C11.4.3, C11.4.5 and C21.2.3
Justification Commentary for 0.2 Sec S_{DS} Cutoff	Adds commentary on the basis for the 0.2-second S_{DS} cutoff.		C11.4.5, C13.3.1 and C21.4
Clarification of SDC D, E and F Ground Motions	Clarifies the conceptual basis and revises the values of the ground motion spectral response acceleration parameters S_{DS} (Table 11.6-1) and $SD1$ (Table 11.6-2) corresponding to Seismic Design Category D and Seismic Design Category E (and F).	Section 11.6	C11.6 and Table C11.6-X

Topic of Change Proposals	Brief Summary of the Changes	Related or New Sections of ASCE/SEI 7-22	Related Commentary in ASCE/SEI 7-22
Seismic Design Category Maps	Adds Seismic Design Category maps for Risk Category I, II and III structures and for Risk Category IV structures, and revises Sections 11.6 and 12.14.1.1 and the commentary.	Section 11.6, Figures 11.6-1a to 11.6-1d and 11.6-2, Section 12.14.1.1	C11.6
Liquefaction-Targeted Peak Ground Acceleration	Establishes a new design parameter for liquefaction hazard analysis, the liquefaction-targeted Maximum Considered Earthquake (MCE_L) peak ground acceleration (PGA_L), to be used in liquefaction hazard assessments in place of the MCE_G peak ground acceleration (PGA_M).	Sections 11.2, 11.3, 11.4.7, 11.8.3, 12.13.9, 15.4.10.1 and 21.5	C11.4, C11.4.7, C11.8.3, C12.13.9, C15.4.10, C21.5 and C22
Identification of Seismic Fault Rupture Risk	Adds definitions and specific requirements for the identification of surface fault rupture hazard and the determination of fault displacement.	Sections 11.2, 11.8.1, 11.8.2 and 11.8.4, Figure 11.8-X	C11.8.1
Kinematic and Inertial Loading on Embedded Walls	Adds requirements for seismic pressures on below-grade components and the design of below-grade elements.	Sections 11.8.3, 12.7.5 and 15.6.1	C11.8.3, C12.7.5 and C15.6.1
Base of Structure	Revises the definition of the base of structure and the modeling and equivalent lateral force requirements for its determination, including sites with sloping grade, basements and liquefiable soils.	Sections 11.2, 12.7.3, 12.7.3.1 and 12.8.3	C11.2, C12.7.3, C12.7.3.1 and C12.8.3
Elastic Seismic Design	Adds uncategorized structural systems to Table 12.2-1 and a new Section 12.2.1.3 for seismic design for elastic response.	Table 12.2-1, Section 12.2.1.3	C12.2.1.3

Topic of Change Proposals	Brief Summary of the Changes	Related or New Sections of ASCE/SEI 7-22	Related Commentary in ASCE/SEI 7-22
<i>MCE_R</i> Ground Motions for Collapse Evaluation of Alternate Structural Systems	Revises the <i>MCE_R</i> ground motions required by Section 12.2.1.1 for FEMA P-695 collapse evaluation of alternate structural systems.	Section 12.2.1.1	C12.2.1.1
Design Coefficients and Factors for Steel Damped Special Moment Frame (DSMF)	Introduces the steel damped special moment frame as a permitted structural system in Table 12.2-1 and adds a new Section 14.1.12 with its design requirements.	Table 12.2-1, Sections 12.3.4.1, 14.1.2.2.1, 14.1.2.2.2 and 14.1.12	C14.1.12
Post-Tensioned Timber Rocking Walls	Introduces post-tensioned timber rocking walls as a new seismic force-resisting system in Table 12.2-1, with design and detailing requirements in a new Section 14.5.2.	Table 12.2-1, Section 14.5.2	C12.2.1 and C14.5.2
Single-Story Steel Ordinary Moment Frames - Added Option	Adds an option for single-story steel ordinary moment frames in Section 12.2.5.6.1.	Section 12.2.5.6.1	C12.2.5.6.1
Diaphragm Flexibility Calculation Clarification	Clarifies the calculation used to idealize a diaphragm as flexible, including the definition of the average drift of adjoining vertical elements.	Sections 11.3 and 12.3.1.3	C12.3.1.3
Vertical Ground Motion in Chapter 12 Commentary	Adds commentary on the two alternatives for the vertical seismic load effect, E_v .		C12.4.2

Topic of Change Proposals	Brief Summary of the Changes	Related or New Sections of ASCE/SEI 7-22	Related Commentary in ASCE/SEI 7-22
When Foundation Soil Flexibility Must Be Included in the Analysis Model	Clarifies when foundation soil flexibility is to be included in the analysis model and provides alternative design provisions when soil flexibility is omitted, with additional commentary on when it may be prudent to include it.	Section 12.7.1	C12.7.1
Revision to Diaphragm Design Forces	Reorganizes Section 12.10 into the non-system-based (12.10.1), system-based (12.10.2) and one-story (12.10.3) diaphragm design methods.	Sections 12.2.3.2, 12.3.3.5, 12.3.4.1 and 12.10, Table 12.10-1	C12.2.3.2, C12.3.3.5, C12.8.6, C12.10 and C14.1.5
Simplification of Alternative Diaphragm Design Provisions	Adds an alternative formula for the design acceleration coefficient used to determine diaphragm forces and extends the single linear interpolation of diaphragm forces over the height of the building from two-story to five-story buildings.	Section 12.10.2.2, Figure 12.10-2	C12.10.2.2
Amplification of Diaphragm Deflections	Revises commentary on the inclusion of diaphragm deformation in the Design Earthquake Displacement.		C12.8.6
Backstay Effect and Diaphragm Force Amplification	Adds commentary on the backstay effect and diaphragm force amplification.		C12.10.1.1
Diaphragm Force Amplification	Revises the collector requirements of the system-based diaphragm design method.	Section 12.10.2.4	C12.10.2.4

Topic of Change Proposals	Brief Summary of the Changes	Related or New Sections of ASCE/SEI 7-22	Related Commentary in ASCE/SEI 7-22
Including Rod-Braced Diaphragms in Alternative Diaphragm Design Provisions	Adds rod-braced diaphragm systems to Table 12.10-1 and adds Sections 14.1.10 and 14.1.11 with requirements for rod-braced diaphragms, including special seismic detailing.	Table 12.10-1, Sections 14.1.2.2.3, 14.1.2.2.4, 14.1.10 and 14.1.11	C12.10.2.5 and C14.1.11
Combination of Diaphragm Design Factors, R_s	Clarifies the design of combinations of diaphragm systems.	Section 12.10.2.5, Table 12.10-1	C12.10.2.5
Cross-Laminated Timber Diaphragm R_s Factor	Adds diaphragm design force reduction factors, R_s , for cross-laminated timber diaphragms.	Section 12.10.2.5, Table 12.10-1	C12.10.2.5
Allowable Drift Limits	Replaces Table 12.12-1 with a simpler version and substantially revises the commentary on drift and deformation.	Table 12.12-1	C12.12
Fault Offset Design Requirements	Adds requirements for foundations and structures located at sites subject to surface fault displacement.	Section 12.13.10, Table 12.13.10-1	C12.13.10

Topic of Change Proposals	Brief Summary of the Changes	Related or New Sections of ASCE/SEI 7-22	Related Commentary in ASCE/SEI 7-22
Chapter 16 without Chapter 12	Revises Chapter 16 so that a structure designed by nonlinear response history analysis does not need to conform to all Chapter 12 requirements.	Sections 11.1.3, 16.1.1, 16.1.2, 16.2.3.1, 16.3.3, 16.3.4, 16.3.7, 16.3.8, 16.3.10, 16.4.1.2, 16.4.1.5, 16.4.1.6 and 16.4.2	C16.1.1, C16.1.2, C16.3.1, C16.3.3, C16.3.4, C16.3.7, C16.3.8, C16.3.10, C16.4.1.2 to C16.4.1.6 and C16.4.2
Accidental Torsion in Nonlinear Response History Analysis	Updates the requirements on when accidental torsion needs to be included when using nonlinear response history analysis.	Section 16.3.5	C16.3.5
Damping Requirements for NLRHA	Expands the requirements and commentary on modeling equivalent viscous damping.	Section 16.3.6	C16.3.6
Unacceptable Response - Force-Controlled Elements Clarification	Clarifies the requirements for unacceptable response evaluation of force-controlled elements.	Sections 16.4.1.1 and 16.4.2.1	None (no commentary required)
Valid Range of Modeling of Deformation-Controlled Actions	Requires consideration of the potential loss of gravity support where the valid range of modeling of deformation-controlled actions is extended, and adds references to ANSI/AISC 341 and ANSI/AISC 342.	Section 16.4.2.2	C16.4.2.2
Foundation Soil Flexibility in Nonlinear Response History Analysis	Addresses when foundation soil flexibility must be included in the nonlinear analysis model.	Section 16.1.4	C16.3.9

Topic of Change Proposals	Brief Summary of the Changes	Related or New Sections of ASCE/SEI 7-22	Related Commentary in ASCE/SEI 7-22
Collapse Probabilities for Seismically Isolated Structures	Adds commentary clarifying the achieved probability of collapse for seismically isolated structures designed in accordance with Chapter 17.		C17.1
Updates to Determination of Kinematic Effects	Requires the reductions to the response spectrum due to site-specific effects and kinematic effects to be quantified separately for the nonlinear response history procedure.	Sections 19.2 and 19.4	C19.2 and C19.4
Updates to Section 21.1 on Site-Specific Site Response	Rewrites Section 21.1 requirements for site-specific site response analysis.	Section 21.1	C21.1
Allowable Ground Motion Reduction from Section 21.1 Site Response	Changes the allowable levels of reduction relative to the USGS Seismic Design Geodatabase depending on the method of analysis used to compute site response in Section 21.1.	Sections 21.2.3 and 21.5	C21.2, C21.2.3 and C21.5
MCEG PGAM Deterministic Capping	Replaces the deterministic capping of the MCEG peak ground acceleration with an increase in the uniform-hazard probability of exceedance from 2% to 3% in 50 years.	Section 21.5	C21.2 and C21.5
2023 USGS National Seismic Hazard Model for the Conterminous U.S.	Updates the Chapter 22 ground motion maps and commentary for the conterminous United States based on the 2023 USGS National Seismic Hazard Model and updates the earthquake-induced collapse risk in a 50-year period for structures to 1.5%.	Chapter 22	C22

Topic of Change Proposals	Brief Summary of the Changes	Related or New Sections of ASCE/SEI 7-22	Related Commentary in ASCE/SEI 7-22
2023 USGS National Seismic Hazard Model for Alaska and Hawaii	Updates the Chapter 22 ground motion maps and commentary for Alaska and Hawaii based on the latest USGS National Seismic Hazard Models for these regions and updates the earthquake-induced collapse risk in a 50-year period for structures to 1.5%.	Chapter 22	C22
2025 USGS National Seismic Hazard Model for Puerto Rico and the U.S. Virgin Islands	Updates the Chapter 22 ground motion maps and commentary for Puerto Rico and the U.S. Virgin Islands based on the 2025 USGS National Seismic Hazard Model and updates the earth-quake-induced collapse risk in a 50-year period for structures to 1.5%.	Chapter 22	C22
Long-Period Transition Period (T_L) Maps	Updates the procedure used to develop the T_L maps and replaces the T_L maps for the conterminous United States, Hawaii, Alaska, and Puerto Rico and the U.S. Virgin Islands.	Figures 22-14 to 22-17	C11.4.5.1 and C11.4.5.2
Appendix A: Seismic Design Criteria and Other Requirements for Functional Recovery	Creates a new Appendix A containing seismic design criteria and other requirements for functional recovery.	Appendix A	Commentary to Appendix A

* The INTENT describes the expected seismic performance that is judged to be inherent in the seismic requirements of NEHRP Provisions. These sections are not contained in ASCE/SEI 7.

Volume II Part 3 of the 2026 *NEHRP Provisions* is a collection of resource papers that introduce new procedures or provisions not currently contained in the referenced standards for consideration and for trial use by the design community, researchers, and standards development and code development organizations. Part 3 also represents Issue Team efforts on substantive proposals for topics that require further consideration by the seismic design community and additional research before being considered for Parts 1 and 2 provisions. Part 3 consists of the following Resource Papers:

- Structural Stainless Steel Seismic Force-Resisting Systems
- Diaphragm Acceleration Comparison Between 12.10.1, 12.10.3 and 13.3: Observations and Future Studies
- Design Ground Motions of Chapter 22
- Two-Stage Analysis: Current Use, Design Issues, and Recommendations For Future Work
- A Design Process to Nurture Creativity and Innovation in Seismic System Design

As part of its efforts to regularly update the *NEHRP Recommended Seismic Provisions*, the BSSC also works with its PUC, Professional Organizations, and general membership to identify and recommend issues to be addressed and research needed to advance the state of the art of earthquake-resistant design and to serve as the basis for future refinement of the Provisions. This future issues and research needs report will be published separately by NIBS and BSSC.

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2026 NEHRP RECOMMENDED SEISMIC PROVISIONS FOR NEW BUILDINGS AND OTHER STRUCTURES: INTENT

This chapter on the Intent of the 2026 Provisions describes the expected seismic performance that is judged to be inherent in the seismic requirements in Parts 1 and 2.

1.1 INTENT

The *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* presents the minimum recommended requirements necessary for the design and construction of new buildings and other structures to resist earthquake ground motions throughout the United States. The objectives of these *Provisions* are to provide reasonable assurance of seismic performance that will:

1. Avoid serious injury and life loss due to
 - a. Structure collapse
 - b. Failure of nonstructural components or systems
 - c. Release of hazardous materials
2. Preserve means of egress
3. Avoid loss of essential functions in Risk Category IV facilities, and
4. Reduce structural and nonstructural repair costs where practicable.

These performance objectives do not all have the same likelihood of being achieved. Additional detail on the objectives is provided in sections 1.1.1 through 1.1.6.

The degree to which these objectives can be achieved depends on several factors, including structural framing type, building configuration, structural and nonstructural materials and details, and overall quality of design and construction. In addition, large uncertainties as to the intensity and duration of shaking and the possibility of unfavorable response of a small subset of buildings or other structures may prevent full realization of these objectives.

1.1.1 Structure Collapse

For objective 1.a the *Provisions* target performance such that the probability of collapse of a significant portion or all of an ordinary use (Risk Category II¹) structure due to earthquake ground shaking does not exceed 10% given the occurrence of risk-targeted Maximum Considered Earthquake (MCE_R) shaking. For Risk Category II structures, this conceptually results in a 1.5% risk of earthquake-induced collapse in a 50-year period for structures having an assumed representative collapse fragility which is based on a detailed ductile structure. The collapse fragility is a mathematical function representing the probability that a structure will collapse at different intensities of ground motion, developed as described in the FEMA P-695 report. This 1.5% risk target is the basis for determining the MCE_R ground motion spectra. At any site, MCE_R shaking is a very rare event and, as the name implies, is the maximum earthquake motion that is considered by these *Provisions* for the design and evaluation of structures. Originally designated Maximum Considered Earthquake Motion (MCE), the quantitative definition of MCE_R shaking parameters, and the procedures for determining them have changed over time. Importantly, the actual design criteria of the *Provisions*, including base shear and other strength criteria, as well as design drift have always been stated in terms of Design Earthquake (DE) level shaking. Since the 1997 *Provisions*, the DE has been defined as 2/3rds of MCE_R level shaking.

Since the mid-1970s, building codes have recognized that some structures, such as schools and hospitals, are more important than others, due to the number or types of occupants or functions performed within the facility, and have sought to reduce the extent of damage such facilities would experience, given strong shaking. This was primarily done through assignment of an Occupancy Importance Factor, *I*, which increased the required strength of the structures, depending on occupancy, by as much as 50%. For some structures, such as schools, the primary intent was to provide a safer environment than afforded the general public, while for other structures, such as industrial facilities, the intent was to minimize the potential for catastrophic release of hazardous materials. For still others, such as hospitals, the primary intent was to reduce the risk of function loss, so that these essential buildings could continue to operate in the post-earthquake environment. These and other performance goals are addressed in later sections.

Regardless of the intent of the importance factor for different buildings, designing for higher strength, the effect of the importance factor value specified for structures assigned to Risk Categories III and IV, is to reduce the amount of damage the structure will sustain at a given intensity of motion, and thereby, also reduce the probability of collapse. ASCE 7, Table 1.3.2 specifies the target collapse reliabilities for structures assigned to the different risk categories. The 10% conditional probability of collapse, given MCER shaking shown for Risk Category II structures is an inherent goal of these *Provisions*. The reduced probabilities stated in ASCE 7 (5% for Risk Category III, 2.5% for Risk Category IV) are the estimated consequence of effectively designing these structures for higher seismic design forces. Depending on the extent that drift, rather than

¹ Where the Risk Category is defined in Section 1.5 of ASCE/SEI 7-16

strength, controls a structure's collapse capacity, and also the intensity of MCER shaking, probabilities may not be universally attained. It is important to recognize that the intent of the Importance Factors was not to design for more intense ground motions, but rather to reduce damage at design earthquake levels, by increasing the required design strength, and thereby decreasing the ductility demand.

The *Provisions* employ a system of Seismic Design Categories to apply various requirements for more rigorous design methods, construction details, and limitations on materials and systems. The category depends on the MCE_R ground motion at the specific site and the Risk Category of the structure. In prior versions of the *Provisions*, the determination of the Seismic Design Category at a specific site also depended on Design Earthquake shaking adjusted for site class at a specific site except for the very highest ground motion locations which were based on the ground motion assuming a rock site. However, starting with the 2026 *Provisions*, the Seismic Design Category (SDC) was based on default ground conditions at the site. The goal of changing the procedure to determine SDC was to cause the SDC to be the same in a given localized geographical area thus resulting in uniform design and construction practice. This uniformity in practice would be expected to occur by having consistent structural and nonstructural component detailing requirements for the area regardless of soil site class.

There are six Seismic Design Categories that are identified as Seismic Design Category A, B, C, D, E, and F. Seismic Design Category A is the lowest category. No seismic design requirements are applied for Category A. It is defined as those sites where the MCE_R ground motion is less than half that associated with structural damage in historical earthquakes, regardless of Risk Category. Seismic Design Category maps are provided in the *Provisions* and the Seismic Design Category for an individual site location is identified in the USGS Seismic Design Geodatabase based on the Design Earthquake ground motion parameters at that site and the Risk Category assigned to the structure that is being designed.

1.1.2 Nonstructural Damage

For objective 1.b, the *Provisions* require design and construction to prevent structural failures (e.g., failure of anchorage or bracing) of nonstructural components or systems, where such failures would endanger life. Although the nuclear industry has developed procedures that enable computation of the reliability of individual nonstructural components mounted in specific structures, no sound generally accepted procedures for characterizing the generic reliability of nonstructural components yet exist. As a result, the provisions for anchorage and bracing of nonstructural components are based on simplified characterization of the shaking transmitted to components mounted in structures, and historic precedent as to appropriate design force levels, benchmarked by observation of damage in past earthquakes. The criterion is based on less severe and more frequent ground shaking than used for protection against structure collapse. Unlike the criteria for design of structures, the criteria are intended to prevent failure of select nonstructural components under Design Earthquake (DE) shaking.

For components that pose a life safety threat due to their weight and position, the fundamental requirement is to maintain the position of the component through anchorage, bracing, and strength. Observations of damage to some unbraced and unanchored components in past earthquakes suggest that life-threatening damage is unlikely under moderate ground motions, while other components such as parapets and other appendages still pose a significant risk. Thus, the scope of components covered is substantially less in the seismic design categories where the ground shaking demand is moderate. Through the use of a component importance factor to require reduced inelastic demand and greater displacement capacity, greater, albeit unquantified assurance of performance is provided given the DE ground motion for components that are necessary for life safety immediately following a strong earthquake, such as fire suppression systems and egress stairways. In addition to requirements for bracing and anchorage, equipment assigned to the high component importance factor must be qualified through testing, experience data, or analysis to assure continuous operation when subject to the DE ground motion. Performance of nonstructural components is also influenced by the requirements for a minimum lateral stiffness (drift limits) for structural systems and requirements that nonstructural systems accommodate the anticipated structural drift; the stiffness requirement is more restrictive for higher Risk Categories.

1.1.3 Hazardous Materials

For objective 1.c, the *Provisions* require that structures and nonstructural components that contain hazardous material be designed and built to provide greater assurance that they will not release unacceptable quantities of hazardous materials during the DE by reducing the inelastic demand. For buildings, nonbuilding structures, and components, the expected performance target is adjusted for the Risk Category just as it is for the collapse objective (1.1.1). For Risk Categories III and IV, the objective is to provide a likelihood of major release of hazardous materials that is very low at the DE ground motion and thus low at the MCE_R ground motion. For nonstructural components, the amount of inelastic behavior permitted at strong ground motions is adjusted with the component importance factor. For nonbuilding structures, the protection from major releases may include secondary containment.

1.1.4 Preservation of Egress

For Objective 2, the *Provisions* intend that stairs be designed and built to be functional following the DE ground motion. The component importance factor is intended to provide a low likelihood that stairs lose support due to seismic displacements.

1.1.5 Functionality of Risk Category IV Facilities

For Objective 3 the *Provisions* intend for Risk Category IV structures to have a higher, but not explicitly quantified, probability of avoiding earthquake-induced loss of essential functionality when subjected to the design earthquake (DE) shaking intensity. To achieve functionality, structural damage must be limited to a level that would permit immediate building reoccupancy. In addition, nonstructural systems required for life safety protection, such as fire protection systems and egress

stairs, as well as components supporting the essential function of the facility, such as garage doors in a fire station, must be operational. The definition of essential functions is left to the determination of the owner or operator of the facility, the governing building code, or the authority having jurisdiction. The *Provisions* include some requirements to increase the likelihood that the function be maintained at the DE ground motion for nonstructural components and systems required to protect life safety and for nonstructural components and systems required for essential function of Risk Category IV facilities. To help achieve these goals, permissible story drifts are reduced to control damage to nonstructural components connected to multiple floor levels. Nonstructural system performance is also enhanced by strengthening the anchorage and bracing requirements for components necessary for life-safety purposes and for essential functionality of Risk Category IV facilities, and by requiring that such equipment and associated systems be shown to be functional after being shaken. The intent is that essential functionality will usually be maintained, or restored in a short amount of time, at ground motions comparable to the motion used for design of nonstructural elements (the DE ground motion).

Given the state of knowledge for predicting such performance for prior versions of the *Provisions*, the probability of meeting that intent was not specified. Even for this version of the *Provisions*, the probability of Risk Category IV structures meeting this intended performance varies and remains unspecified. However, this edition of the *Provisions* now includes a new Appendix A: Seismic Design Criteria and Other Requirements for Functional Recovery for use when functional recovery performance in terms of target time for recovery of basic intended function is desired for a given structure. Appendix A contains design criteria and other requirements for achieving post-earthquake functional recovery performance with stated risk and reliability expectations for different specified target recovery times, relative to a different earthquake's shaking intensity.

For Risk Category IV buildings designed and constructed to meet the base requirements of the *Provisions* (without implementing Appendix A), the following qualitative characteristics define the intended performance objective when subjected to the design earthquake shaking intensity (although the probability of achieving this performance will vary and is not specified):

1. Remains safe for continued occupancy.
2. Equipment required for the essential functions of the building functions after the event.
3. Contents required for the essential functions of the facility have not been damaged.
4. Nonessential equipment and contents may sustain damage, provided that the damage does not compromise the essential functions of the building.
5. The building envelope maintains integrity where required to preserve essential functions.
6. Piping carrying nontoxic substances have only minor leakage.

7. Toxic and highly toxic substances are not released in a quantity harmful to occupants unless controlled through secondary containment.
8. Egress is maintained.

For Risk Category IV nonbuilding structures designed and constructed to meet the base requirements of the *Provisions* (without implementing Appendix A), the following qualitative characteristics define the intended performance objective when subjected to the design earthquake shaking intensity (although the probability of achieving this performance will vary and is not specified):

1. Release of contents essential to the function of the facility is prevented.
2. Equipment required for essential function of the nonbuilding structure functions after the event.
3. Contents required to fulfill the essential functions of the facility have not been damaged.
4. Nonessential equipment and contents may sustain damage, provided that the damage does not compromise the essential functions of the nonbuilding structure.
5. Piping carrying nontoxic substances have only minor leakage.
6. Toxic and highly toxic substances are not released in a quantity harmful to occupants unless controlled through secondary containment.
7. Access required for essential functions of the nonbuilding structure is preserved after the event.
8. Egress from the nonbuilding structure is maintained.

When specific functional recovery performance objectives (e.g., time for recovery of basic intended function) are desired, including for Risk Category IV structures, use of the Appendix A provisions should be considered. See Appendix A and its commentary for additional discussion of the intent and background of the functional recovery provisions and performance objectives.

1.1.6 Repair Costs

Objective 4 is primarily aimed at those nonstructural elements for which seismic anchorage and bracing are both low cost and effective in reducing economic losses in ground motions that are smaller and more frequent than the motions used for life safety. There are also provisions in various material design standards that aim to provide additional resistance for certain structural failure states that are not particularly threatening to life but are very expensive to repair.

2.1 COMMENTARY TO THE INTENT

The primary intent of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* is to prevent, for ordinary buildings and structures, serious injury and life loss caused by damage from earthquake ground shaking and ground failure. Most earthquake injuries and deaths are caused by structural collapse; therefore, the major thrust of the Provisions is to prevent collapse for very rare, intense ground motion, termed the risk targeted maximum considered earthquake (MCE_R) ground motion. Additional objectives to preserve means of egress, maintain essential functionality of Risk Category IV facilities following major earthquakes, and to reduce damage costs, where practicable, are addressed as corollaries to the primary intent.

The *Provisions* requirements are not intended to prevent damage due to landslides (such as those that occurred in Anchorage, Alaska) or tsunami (such as occurred in Hilo, Hawaii, the Indian Ocean, and Japan). They provide only for required resistance to earthquake ground shaking and movements due to liquefaction without significant slides, subsidence, or faulting in the immediate vicinity of the structure. In most cases, practical engineering solutions are available to resist other potential earthquake hazards, but they must be developed on a case-by-case basis. The *Provisions* do require geotechnical investigations for sites where such instabilities are possible, and the geotechnical reports must recommend appropriate mitigation.

Although the *Provisions* set the minimum performance goals described in Section 1.1, earthquake performance of buildings and other structures is highly variable. The characteristics of the shaking itself are highly uncertain and even different ground motion records defined to qualify as maximum considered earthquake ground motions for the same target spectrum can result in significantly different responses. Additional uncertainty is created by the wide variety of systems and configurations allowed under the regulations as well as by the various interpretations and implementation practices of individual designers. Thus, a small percentage of buildings designed to the requirements of the *Provisions* may not meet the performance intent when exposed to earthquake ground motions. The commentary of the *Tentative Provisions for the Development of Seismic Regulations for Buildings* (Applied Technology Council, 1978), upon which the first edition of the *NEHRP Recommended Provisions* (1985) was based, suggested a less than 1% chance of collapse in a 50-year period for a building designed using the tentative requirements. More recent studies (e.g., *Quantification of Building Seismic Performance Factors*, FEMA P-695, 2009) suggest a 10% chance of collapse with shaking at the maximum considered earthquake level, which is roughly equivalent to the 1978 estimations.

In the future it is possible that the risk targeting concepts implemented for the structural collapse objective may be applied to other objectives, using methods such as described in *Tentative Framework for Advanced Seismic Design Criteria for New Building* (NIST, 2012). More knowledge of seismic performance of constructed systems is needed to accomplish this.

2.1.1 Structure Collapse

Preservation of life safety, through avoidance of serious structural damage and collapse has been the primary intent of U.S. seismic code provisions since the first building code adoption of earthquake design criteria in the early 20th century. Throughout much of the 20th century, commentary to the building codes stated that the provisions were intended to minimize the potential for life-threatening structural damage in earthquakes producing shaking intensities comparable to the most severe shaking historically experienced in different regions of the U.S. The commentary also recognized that more intense earthquake shaking could occur, and that there were no guarantee structures designed in accordance with the code would withstand such shaking.

With the publication of the ATC 3-06² document in the late 1970s, the profession began to accept that the level of shaking for which life safety protection was provided in the building code corresponded to shaking having a 10% chance of exceedance in 50 years, or a 475-year return period. The structural performance level associated with this level of ground shaking came to be called the Life Safety level. Life Safety performance was generally characterized as a damage state in which reduction of both structural stiffness and strength occurred where occupancy may not be advisable until repair or stabilization work was performed and also where repair might not be practical. Commentary recognized that more severe shaking could occur, and speculated, but did not guarantee, that while well-designed structures subjected to such shaking might experience damage threatening to a few occupants, overall building collapse could be avoided in most cases. Some project design criteria began to refer to such shaking as the Maximum Capable Earthquake. The California Building Code³ explicitly recognized this higher shaking level, terming it the Upper Limit Shaking level, and assigned it an exceedance probability of 5% in 50 years (975- year return period).

The NEHRP Provisions first formally adopted a low, but undefined, probability of collapse in the event of Maximum Considered Earthquake (MCE) shaking as a primary objective in the 1997 edition. The *1997 Provisions* generally defined MCE shaking as ground motion having a 2% probability of exceedance in 50 years (2,475-year mean recurrence). This interval was selected as it was then believed such long return periods were necessary to capture a repeat of ground shaking intensities that had historically occurred in the central and eastern U.S., such as the 1811-1812 New Madrid events. However, this low exceedance probability resulted in ground motions that seemed impractically high in the western U.S., particularly at sites within a few kilometers of major active faults such as the San Andreas and San Jacinto faults in California. Therefore, for such sites,

² Applied Technology Council. Tentative Recommended Lateral Force Criteria and Commentary, Report No. ATC 3-06, 1979, National Science Foundation

² California Building Standards Commission, California Building Code, Title 24, 1979, International Conference of Building Officials.

³ Applied Technology Council. Quantification of Building Seismic Performance Factors, 2009, Federal Emergency Management Agency

it was decided to cap the MCE, or later MCE_R ground shaking intensities to 1.5 times the median ground motion for a characteristic earthquake on the nearby fault. This characteristic earthquake was determined based on documented fault length, slip rate, and recurrence models for the fault. This limitation on MCE shaking, known as the Deterministic Cap, applied when the 2% in 50-year ground motions exceeded a Deterministic Lower Limit defined as 150% of the 1994 Zone 4 UBC ground motion.

The 2009 *Provisions* changed the procedures for determining the MCE ground motion based on technology developed in the *FEMA P695*⁴ report. In place of MCE shaking, the 2009 *Provisions* defined risk-targeted Maximum Considered Ground shaking (MCE_R) based on a presumed risk target of 10% probability of collapse given the occurrence of the MCE_R . MCE_R shaking was in turn defined as that shaking that resulted in a calculated 1% in 50-year collapse probability, which was obtained when the ground motion probabilistic site hazard was mathematically integrated with an assumed fragility function having a 10% probability of collapse at MCE_R shaking and a logarithmic standard deviation of 0.6. This logarithmic standard deviation represents uncertainty in the intensity at which predicted collapse will occur, resulting from record-to-record variability in ground motion, uncertainty in analytical modeling, and other factors in the design and construction process. The value of 0.6 is based on the *FEMA P-695* work and studies of typical detailed ductile structures, including special moment frame, special shear wall, and steel special concentrically braced frame structures. This resulted in varying return periods for MCE_R motion, around the nation. Generally, MCE_R return periods varied from approximately 1,000 years to 2,000 years, depending on regional seismicity. MCE_R intensities near major active faults in the western U.S. remained very high, and so the Deterministic Cap was maintained. However, rather than taking the Deterministic Cap as 150% of the median motion for a characteristic earthquake, it was defined as the 84th percentile motion for such an earthquake. To simplify MCE_R mapping, the USGS assumed that the 84th percentile motion was equal to 180% of the median motion for a characteristic earthquake. In the Deterministic Capped areas, the risk target goal of achieving a 10% collapse probability given the occurrence of the MCE was still the governing goal, however the absolute probability of collapse could be significantly higher than an absolute probability goal of 1% in 50 years. As a result, absolute risk targets were different in the very high seismic areas, as compared to the remaining areas. In addition, both probabilistic and deterministic MCE_R values were redefined as the intensity associated with the maximum direction of shaking (Rot-D100), recognizing that real ground motions have a polarity, with shaking producing stronger response in some directions than others.

In the 2026 *Provisions*, the procedures for determining MCE_R shaking intensities changed again, defining the target collapse risk at 1.5% in 50 years, rather than 1%. This generally reduced MCE_R shaking intensities in most of the United States. Changing the risk target to 1.5% made it possible to eliminate the deterministic caps on ground motions; however, as a result of removal of the Deterministic Cap, MCE_R motions in some of these previously capped areas did increase relative

to that required in earlier editions of the *Provisions*. It should be noted that since the governing risk target is 10% probability, given the occurrence of the MCE_R changing how the MCE_R is determined does not change that goal. Further information on determination of MCE_R shaking intensity may be found in Commentary to Chapter 22.

Although a basic intent of design is to provide a low probability of earthquake-induced collapse in the event of MCE_R shaking, rather than defining required design strength as a function of MCE_R shaking, these *Provisions* formulate required strength in terms of Design Earthquake shaking, taken as 2/3rds of the intensity of MCE_R shaking. This approach was adopted by the 1997 *Provisions*, based on a belief that well-designed structures had the capability of resisting motions 150% more intense than the design level without significant risk of collapse, and thus perform at a Life Safety level. In addition, in many regions in the western U.S., 2/3rds of the 1997 MCE motions approximated those specified as the basis for required strength in earlier editions of the provisions and contemporary building codes.

The 1.5% in 50-year risk target is roughly an order of magnitude higher than the chance of structural failure targeted by the ASCE 7 standard under other loading conditions (e.g., Dead, Live, Snow, Wind, and combinations of these), in large part because the cost of providing seismic protection is substantial in high-hazard locations while the likelihood a building will experience design-level or more intense shaking is low, even in the most seismically active regions. In early building codes, performance goals that admitted to the possibility of substantial structural damage under design shaking were accepted as an economic tradeoff, and in recognition of observations that many structures had survived strong earthquakes in California and other seismically active regions, without collapse, or endangerment of life safety. These tradeoffs were made without solid understanding of the real risk or how to calculate it. Later, when the FEMA P-695 methodology made it possible to compute collapse risk, the selected target was back-calculated for structures designed to the then-current provisions.

The target collapse probabilities should be regarded as notional. Observation of damage from major earthquakes in the U.S. and abroad, over the past 150 years, suggests that these calculated probabilities are much higher than the fragility of real structures, even those designed to outdated building codes. There are many reasons for this. Perhaps the most significant reason is that *FEMA P695* methodology intentionally neglects the substantial reserve strength and stiffness provided by structural elements that are not part of the intended seismic force-resisting system. This is because the building code does not regulate how much reserve strength must be provided by nonparticipating elements, and as a result, some structures could be designed with only the seismic force-resisting system present. Another significant factor is that most studies implementing *FEMA P695* methodology assume the structure is ‘pinned’ at the soil-structure interface; computed damage is somewhat mitigated when soil-structure interaction is explicitly considered and the soil-structure foundation interface may not be capable of transmitting very large ground motions into the structure. In addition the procedure accounts for many uncertainties including the difficulty of modeling the response of structures subject to extreme damage and very large displacements; the apparent random variability of ground motions and their effects on structures; errors induced by translating laboratory test results on isolated elements into models of holistic structures; and even

the errors engineers and contractors are likely to introduce into the constructed structure, when implementing the code. Regardless, the risk targets and the procedure for calculating them are a useful index but should not be regarded as more than that.

The *Provisions* also seek to protect against local failure that does not result in global collapse but could result in injury risk to a few persons. Local failure is prevented by prescriptive provisions in Chapter 12, which require structural design for deformation compatibility for members that are not part of the seismic force-resisting system to prevent local failures, and which prohibit the use of some structural systems that have a poor performance history in high seismic design categories.

The ground motion level below which seismic design is not required is established at a conservatively low level in part to recognize the lower confidence of knowledge of seismic hazards in such areas, but also to address this discrepancy in risk under other loadings in an approximate fashion. In other words, given the variation in ground shaking hazard with probability of exceedance in the pertinent range of probabilities, the risk of collapse due to seismic action should be well under the 1.5% in 50 years target near the transition in hazard level from Seismic Design Category A to B. The transition in risk of collapse to the target of 1.5% in 50 years where the MCE_R motions are higher is not yet well understood.

2.1.2 Nonstructural Damage

Falling exterior walls and cladding and falling interior ceilings, light fixtures, pipes, equipment, and other nonstructural components have historically caused many deaths and injuries, as well as loss of function. The *Provisions* seek to minimize this risk using requirements for anchoring and bracing nonstructural components. In the future it may be possible to target this objective to a specific risk, but at this time the level of protection is set at an undefined high reliability for Design Earthquake shaking, which is traditionally the level used to design nonstructural components.

The demand on nonstructural components depends on the strength and stiffness of the supporting structure, the intensity of the ground shaking, and varies with the amount of yielding and damage in the structural response to the ground shaking. Design force levels in Chapter 13 are based on simplified, approximate consideration of these effects.

The component importance factor is used to reduce the probability of failure of nonstructural components or systems that create a risk to life stemming from loss of their function immediately following the earthquake, such as the failure of the fire suppression system, rather than risks posed by the component from its weight and position. The uncertainty in performance would be similar to the uncertainty in structural collapse, because the overall uncertainty is dominated by variations in ground motion and dynamic response of both the structure and the component, therefore the risk of failure of such components is likely reduced but the degree cannot be stated at this time.

2.1.3 Hazardous Materials

Hazardous materials can be released by structural and nonstructural failure; however, such failures can occur short of collapse. The expectation is that the probability of catastrophic release of such materials across a facility boundary would be similar to the probability of structure collapse for ordinary structures, although more study will be needed to validate that any target is indeed met by the recommended provisions. Release within a facility where relatively fewer lives are at risk would be less rare, although no specific target exists at this time. There is a lack of data, especially regarding the performance of nonstructural systems under strong ground shaking that makes quantification of the objective impossible without further study. Refer to the commentary for Section 1.5.3 of ASCE/SEI 7-22 for the quantitative definition of toxic, highly toxic, and explosive categories of hazardous materials.

2.1.4 Preservation of Egress

In the 2015 *Provisions*, preservation of egress was identified as a distinct objective. At this time, the specific requirements are focused on deformation compatibility of stairs and ramps.

Consideration was given to requiring that doors be designed to sustain anticipated structural drifts. However, consensus could not be obtained on this approach, in part due to the lack of substantive history demonstrating that jammed doors were a substantial life safety hazard.

2.1.5 Functionality of Risk Category IV Facilities

This section describes qualitatively the intended performance of Risk Category IV structures designed to the *Provisions* with respect to preservation of essential functions. The *Provisions* intend for Risk Category IV structures to have a higher, yet unquantified, probability of resuming their essential functions following the design earthquake. The qualifier of “essential” on functions and functionality is present because it is recognized that following design earthquake level shaking the nonessential functions of the facility may be impaired, but that the most important functions the facility performs can still be accomplished. Moderate damage to components and contents, not essential to the function of the facility may occur but must not compromise the functionality of essential components. Cleanup and minor repairs, which can be carried out while the facility is operating, may be required.

It is important to realize that functionality does not imply an absence of damage, or even function as it would be under ordinary circumstances before an earthquake. Experience has shown that extensive workaround solutions are made to respond to damage, even in essential facilities so that some level of function is maintained; however, such solutions are not the goal of the *Provisions*. Some analytical studies (ATC, 2018) have shown that the probability of an unsafe placard and damage likelihood is not uniform for all structural systems, and some with potentially unacceptable results. The functionality objective for Risk Category IV will often control the structural design over the collapse objective. The performance of essential structures in past earthquakes indicates that the increase in the importance factor and more stringent story drift limits, in combination with strict regulation of design, testing, and inspection, reduces structural damage in moderate shaking.

Experience data shows that some nonstructural components will remain functional if they stay in position, but other components will require testing to show that they will function following strong shaking.

For nonstructural components and systems, the emphasis to date has been on the seismic qualification of individual components and analysis of individual systems. However, the nonstructural systems of many buildings are, in reality, complex networks that can be shut down by a single failure. For example, a break in a pressurized pipe can flood a critical area of the facility, or if not quickly isolated, all of a building, forcing it to close, and failure of the anchorage (or internal workings) of a battery, day tank, fuel lines, muffler, or main engine can shut down an emergency generator. Therefore, the special regulations for seismic protection of nonstructural systems represent a rational approach to achieving performance appropriate for the various occupancies, but experience data to confirm their adequacy are lacking.

To increase the likelihood that function be maintained at the DE ground motion for nonstructural components and systems required to function for life-safety purposes and for components and systems required for the continued operation of Risk Category IV facilities, the *Provisions* includes some further design, detailing, and testing considerations for such components and systems. Permissible story drifts are reduced in the *Provisions* to control damage to structural components and nonstructural components connected to multiple floor levels. The anchorage and bracing requirements are enhanced for nonstructural systems and components necessary for life-safety or for functionality of Risk Category IV facilities. Equipment and associated systems required for life-safety or essential functions are required to be shown to be functional after being shaken through direct testing or detailed analysis. The expectation is that functionality will usually be maintained at ground motions comparable to the motion used for design of nonstructural elements (the DE ground motion). However, the probability of meeting that expectation is not specified and may not be consistent for all structural and nonstructural systems and configurations.

The intent dictates a high probability of preventing loss of function but does not explicitly state a reliability target. Observations from past earthquakes have shown that the performance of many structural systems, particularly those used in nonbuilding structures, will meet or exceed this goal. However, it may be difficult to demonstrate so analytically. Many nonbuilding structures designed to modern codes performed well in the Chile Earthquake of 2010 (Soules, Bachman, Silva 2016). Ground supported tanks and pressure vessels did not release their contents even though the measured ground motions were in excess of the design level event. Steel and concrete stacks and a recently completed bar mill building were completely undamaged during the Chile event.

Ultimately, the probability of Risk Category IV structures meeting the functionality performance intent of the base provisions is still variable and unspecified. However, much work has been done since the 2020 edition of the *Provisions* (and even prior to that edition) to develop design and construction practices that consider post-earthquake functional recovery (recovery of basic intended function). The development and advancement of performance-based assessment methods, like FEMA P-58, has allowed earthquake performance of structures to be quantified in terms of probability of safety, repair cost, and repair time. More recently, a new methodology

(based on, but an extension of, the FEMA P-58 methodology) has been developed with the intent to quantify performance in terms of probability of functional recovery time. Based on these advanced methods, this version of the *Provisions* now includes a first-generation approach to consider functional recovery performance in Appendix A: Seismic Design Criteria and Other Requirements for Functional Recovery. See Appendix A and its commentary for additional discussion of the background, intent, and implementation of the functional recovery provisions and performance objectives. Even with the inclusion of Appendix A, more work is expected in the future so that functional recovery provisions can be expanded and updated to include consideration of additional structural systems, additional nonstructural components and their systems, and additional seismic design categories.

2.1.6 Repair Costs

The requirements for anchoring and bracing of nonstructural components and systems coupled with reasonable limitations on differential movement between floors (i.e., story drift limits) may serve to control damage that may be costly to repair or that would result in lengthy building closures, particularly for moderate shaking levels. This level of economic protection will vary across different types of structural and nonstructural systems, and no specific target has been established, nor is there a consensus among stakeholders as to the appropriate levels of protection. Nonstructural designs for story drift that focus on limiting damage to the component or system rather than only preventing catastrophic failure are much more effective at reducing economic losses.

Stricter story drift limits can further limit damage to components connected to more than one floor (e.g., walls, cladding and stairways) but, at the same time, can create higher acceleration levels in the building that could increase damage to nonstructural components braced or anchored to a single floor (e.g., ceilings, light fixtures, and pipes). Achieving an optimum balance between the cost and performance of the structural system and the effect of structural stiffness on performance of the nonstructural systems is not accomplished using the prescriptive rules of a building code, particularly given the variety of structural systems used in the United States.

Examples of provisions with a primary focus of damage control, rather than life safety, include bracing of lightweight ceiling systems, limitations on punching shear in concrete flat slabs (in the design standard for concrete structures), and limitations on interstory drift for masonry walls.

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REFERENCE DOCUMENT

Design for seismic resistance of structural elements including foundation elements and nonstructural components shall conform to the requirements of ASCE/SEI 7-22, *Minimum Design Loads for Buildings and Other Structures*, including any associated supplements (referred to hereinafter as ASCE/SEI 7-22), as modified herein.

2026 NEHRP RECOMMENDED SEISMIC PROVISIONS FOR NEW BUILDINGS AND OTHER STRUCTURES: PART 1, PROVISIONS

MODIFICATIONS TO ASCE/SEI 7-22, CHAPTERS 11-22 AND APPENDIX A

Part 1, Provisions provides recommended changes to the seismic requirements of ASCE/SEI 7-22, Chapters 11 to 23 and Appendix A. For a given chapter, only those sections with recommended modifications or additions are shown. Therefore, Part 1, Provisions should be used side-by-side with ASCE/SEI 7-22 to grasp the full context of each chapter.

CHAPTER 11, SEISMIC DESIGN CRITERIA (Modifications)

SECTION 11.1 GENERAL

Revise Section 11.1.3 as follows:

11.1.3 Applicability Structures and their nonstructural components shall be designed and constructed in accordance with the requirements of the following chapters based on the type of structure or component:

Buildings: Chapter 12 or Chapter 16

Nonbuilding structures: Chapter 15

Nonstructural components: Chapter 13

Seismically isolated structures: Chapter 17

Structures with damping systems: Chapter 18

SECTION 11.2 DEFINITIONS

Add and revise the following definitions to Section 11.2:

The following definitions apply only to the seismic provisions of Chapters 11 through 22 of this standard.

BASE: The level at which the horizontal seismic ground motions are imparted to the structure. See section 12.7.3.1.

Fault: A geologic fault is a shear or zone of closely associated shears across which earth materials on one side have been displaced with respect to those on the other side because of tectonic forces. Shears caused by landslides or other gravity-driven surficial failures are not included in this definition.

Fault trace: The line formed by the intersection of a fault and the surface of the earth.

Fault zone: An area that envelopes the fault traces associated with a given fault.

Known Active Surface Fault Zone: A fault zone that has documented surface fault rupture within the Holocene identified by the AHJ.

MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION: The most severe earthquake effects considered by this standard, more specifically defined in the following three terms:

Liquefaction-Targeted Maximum Considered Earthquake (MCE_L) Peak Ground Acceleration:

The most severe earthquake effects considered by this standard determined for the geometric mean peak ground acceleration and with adjustment for targeted liquefaction hazard level. The MCE_L peak ground acceleration (PGA_L) is used in this standard for evaluation of liquefaction triggering and liquefaction-related consequences. In this standard, general procedures for determining PGA_L are provided in Section 11.8.3; site specific procedures are provided in Section 21.5.

Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration: The most severe earthquake effects considered by this standard determined for geometric mean peak ground acceleration and without adjustment for targeted risk. The MCE_G peak ground acceleration adjusted for site effects (PGA_M) is used in this standard for evaluation of geotechnical issues not related to seismic liquefaction, such as seismic slope stability and evaluation of permanent ground deformation not caused by soil liquefaction. In this standard, general procedures for determining PGA_M are provided in Section 11.8.3; site-specific procedures are provided in Section 21.5.

Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Acceleration:

The most severe earthquake effects considered by this standard determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk. In this standard, general procedures for determining the MCE_R ground motion values are provided in Section 11.4.4; site-specific procedures are provided in Sections 21.1 and 21.2.

Maximum Considered Fault Displacement (MCFD): Maximum differential, horizontal and/or vertical displacement (i.e., offset) within the plan envelope of the structure from a single fault trace that must be considered in the design of the structure.

Structural Height: The vertical distance from the base to the highest level of the seismic force-resisting system of the structure. For pitched or sloped roofs, the structural height is from the base to the average height of the roof.

Surface fault rupture: Relative displacement across a fault at the surface of the earth.

USGS SEISMIC DESIGN GEODATABASE: A US Geological Survey (USGS) database of geocoded values of seismic design parameters S_S , S_1 , S_{MS} , S_{M1} , PGA_L , and PGA_M and geocoded sets of multi-period 5%-damped risk-targeted maximum considered earthquake (MCE_R) response spectra.

SECTION 11.3 SYMBOLS

Add and revise the following symbols:

C_L = site-specific liquefaction coefficient at peak ground acceleration (Section 21.5.4)

PGA_G = Lower-bound limit on deterministic maximum considered earthquake geometric mean peak ground acceleration (Table 21.2-1)

PGA_L = Mapped MCE_L peak ground acceleration as defined in Section 11.8.3

PGA_M = Mapped MCE_G peak ground acceleration as defined in Section 11.8.3

Δ_{ADVE} = Average drift of adjoining vertical elements of the seismic force-resisting system over the story below the diaphragm under consideration, in. (mm)

SECTION 11.4 SEISMIC GROUND MOTION VALUES

Revise Sections 11.4.3 and 11.4.5 as follows:

11.4.3 Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration

Parameters Risk-targeted maximum considered earthquake (MCE_R) spectral response acceleration parameters S_S , S_1 , S_{MS} , and S_{M1} shall be obtained from the USGS Seismic Design Geodatabase for the applicable site class. For sites classified as Site Class DE or E in the conterminous United States, east of -105 degrees longitude, S_{MS} , and S_{M1} shall not be taken as less than the values for Site Class D.

EXCEPTION: Where a site-specific ground motion analysis is performed in accordance with Section 11.4.7, risk-targeted maximum considered earthquake (MCE_R) spectral response acceleration parameters S_{MS} and S_{M1} shall be determined in accordance with Section 21.4 and risk-targeted maximum considered earthquake (MCE_R) spectral response acceleration parameters S_S and S_1 shall be either (1) determined from the site-specific MCE_R response spectrum calculated in accordance with the requirements of Section 21.2.3 assuming Site Class BC site condition or (2) obtained from the USGS Seismic Design Geodatabase.

11.4.4 Design Spectral Acceleration Parameters Design earthquake spectral response acceleration parameters at short periods, S_{DS} , and at 1-s periods, S_{D1} , shall be determined from Equations (11.4-1) and (11.4-2), respectively. Where the simplified alternative design procedure of Section 12.14 is used, the value of S_{DS} shall be determined in accordance with Section 12.14.8.1, and the value for S_{D1} need not be determined.

$$S_{DS} = \frac{2}{3} S_{MS} \quad (11.4-1)$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad (11.4-2)$$

where

S_{MS} = MCE_R , 5%-damped, spectral response acceleration parameter at short periods adjusted for site effects as determined in accordance with Section 11.4.3, and

S_{M1} = MCE_R , 5%-damped, spectral response acceleration parameter at a period of 1 s adjusted for site effects as determined in accordance with Section 11.4.3.

11.4.5 Design Response Spectrum Where a design response spectrum is required by this standard, the design response spectrum shall be determined in accordance with the requirements of Section 11.4.5.1.

EXCEPTIONS:

1. Where a site-specific ground motion analysis is performed in accordance with Section 11.4.7, the design response spectrum shall be determined in accordance with Section 21.3.
2. Where values of the multi-period 5%-damped MCE_R response spectrum are not available from the USGS Seismic Design Geodatabase, the design response spectrum shall be permitted to be determined in accordance with Section 11.4.5.2.

11.4.5.1 Multi-Period Design Response Spectrum The multi-period design response spectrum shall be developed as follows:

1. At discrete values of period, T , equal to 0.0 s, 0.01 s, 0.02 s, 0.03 s, 0.05 s, 0.075 s, 0.1 s, 0.15 s, 0.2 s, 0.25 s, 0.3 s, 0.4 s, 0.5 s, 0.75 s, 1.0 s, 1.5 s, 2.0 s, 3.0 s, 4.0 s, 5.0 s, 7.5 s, and 10 s, the 5%-damped design spectral response acceleration parameter, S_a , shall be taken as 2/3 of the multi-period 5%-damped MCE_R response spectrum from the USGS Seismic Design Geodatabase for the applicable site class.
2. At each response period, T , less than 10 s and not equal to one of the discrete values of period, T , listed in Item 1 above, S_a , shall be determined by linear interpolation between values of S_a , of Item 1 above.
3. If $T_L \geq 10$ s, then at each response period, T , greater than 10 s, S_a , shall be taken as the value of S_a at the period of 10 s of Item 1 above, and multiplied by:

$10/T$, where the value of T is less than or equal to that of the long-period transition period, T_L , or

$10T_L/T^2$, where the value of T is greater than that of the long-period transition period, T_L .

4. If $T_L < 10$ s, then at each response period, T , greater than 10 s, S_a , shall be taken as the value of S_a at the period of 10 s of Item 1 above, and multiplied by $(10/T)^2$.
5. For sites classified as Site Class DE or E in the conterminous United States, east of -105 degrees longitude, S_a shall not be taken as less than the values for Site Class D.

11.4.5.2 Two-Period Design Response Spectrum The two-period design response spectrum shall be developed as indicated in Figure 11.4-1 and as follows:

1. For periods less than T_0 , the design spectral response acceleration parameter, S_a , shall be taken as given in Equation (11.4-3):

$$S_a = S_{DS} \left(0.4 + 0.6 \frac{T}{T_0} \right) \quad (11.4-3)$$

2. For periods greater than or equal to T_0 and less than T_S , the design spectral response acceleration parameter, S_a , shall be taken as equal to S_{DS} .
3. For periods greater than or equal to T_S and less than T_L , the design spectral response acceleration parameter, S_a , shall be taken as given in Equation (11.4-4):

$$S_a = \frac{S_{D1}}{T} \quad (11.4-4)$$

4. For periods greater than T_L , S_a shall be taken as given in Equation (11.4-5):

$$S_a = \frac{S_{D1} T_L}{T^2} \quad (11.4-5)$$

where

S_{DS} = Design spectral response acceleration parameter at short periods;

S_{D1} = Design spectral response acceleration parameter at a 1 s period;

T = Fundamental period of the structure, S;

$$T_0 = 0.2(S_{D1} / S_{DS});$$

$$T_S = S_{D1} / S_{DS}; \text{ and}$$

T_L = Long-period transition period(s) shown in Figures 22-14 through 22-17.

For sites classified as Site Class DE or E located in the conterminous United States, east of -105 degrees longitude, the values of S_{DS} and S_{D1} shall not be taken as less than the values of S_{DS} and S_{D1} determined for Site Class D.

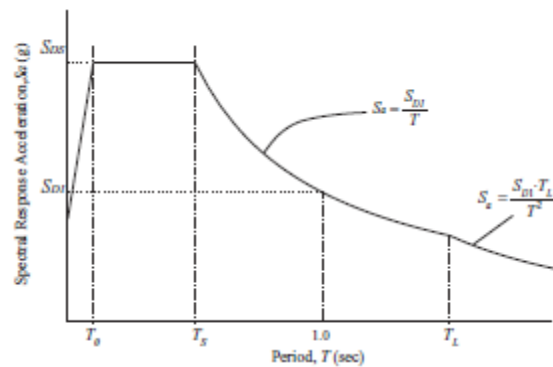


Figure 11.4-1. Two-period design response spectrum.

Revise Section 11.4.7 as follows:

11.4.7 Site-Specific Ground Motion Procedures A site response analysis shall be performed in accordance with Section 21.1 for structures on Site Class F sites, unless exempted in accordance with Section 20.3.1.

It shall be permitted to perform a site response analysis in accordance with Section 21.1 and/or a ground motion hazard analysis in accordance with Section 21.2 to determine ground motions for any structure.

When the procedures of either Section 21.1 or 21.2 are used, the design response spectrum shall be determined in accordance with Section 21.3, the design acceleration parameters shall be determined in accordance with Section 21.4, and, if required, the MCE_L peak ground acceleration parameter PGA_L and MCE_G peak ground acceleration parameter PGA_M shall be determined in accordance with Section 21.5.

SECTION 11.6 SEISMIC DESIGN CATEGORY

Revise Section 11.6 as follows:

11.6 SEISMIC DESIGN CATEGORY

Each Structure shall be assigned a Seismic Design Category in accordance with this section. The Seismic Design Category for structures classified as Risk Category I, II and III shall be determined from Figure 11.6-1. The Seismic Design Category for structures classified as Risk Category IV shall be determined from Figure 11.6-2.

Add Figure 11.6-1a, 11.6-1b, 11.6-1c and 11.6-1d: SDC Map for Risk Category I/II/III Structures

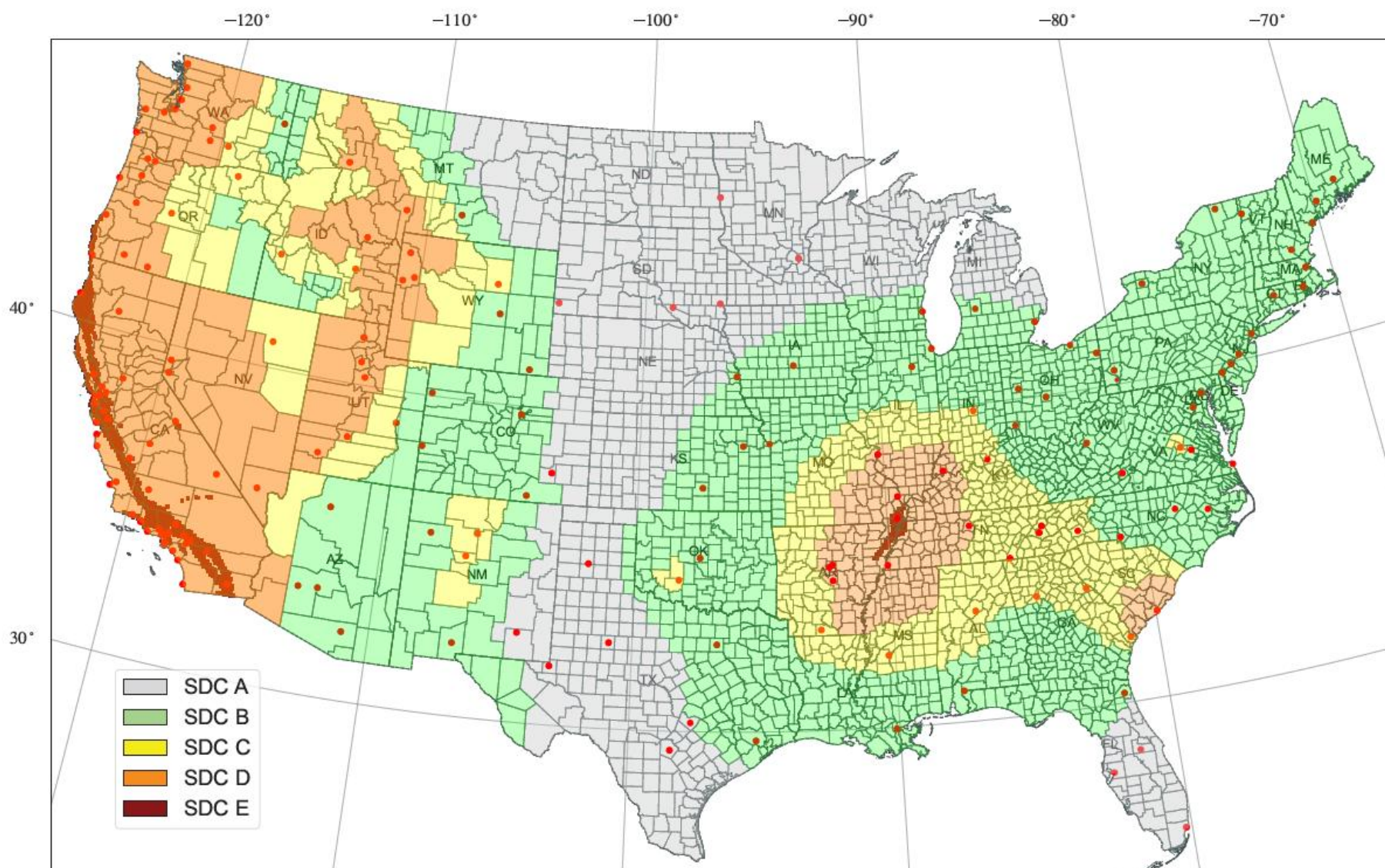


FIGURE 11.6-1a SDC MAP FOR RISK CATEGORY I, II AND III STRUCTURES

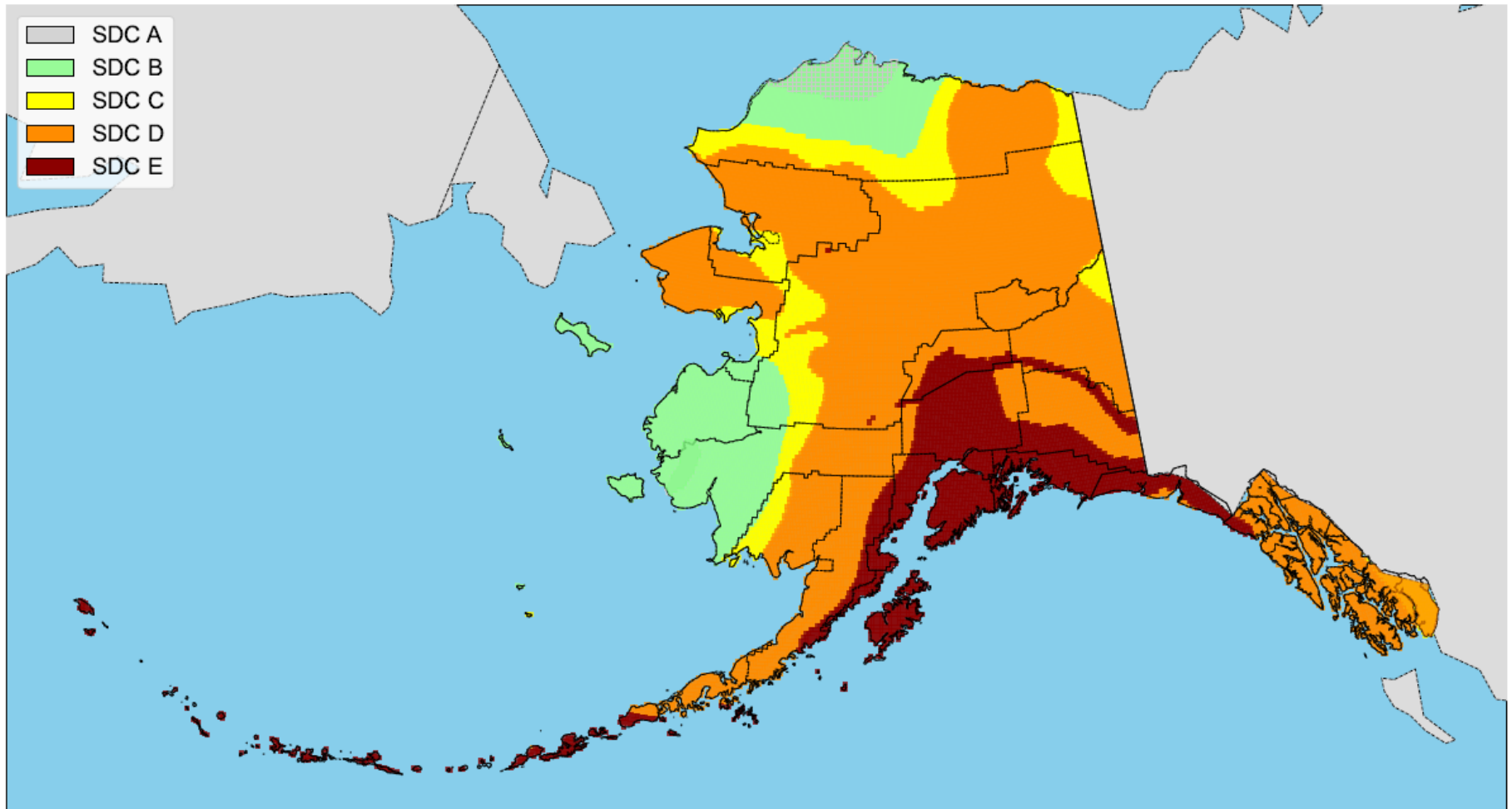


FIGURE 11.6-1b SDC MAP FOR RISK CATEGORY I, II AND III STRUCTURES

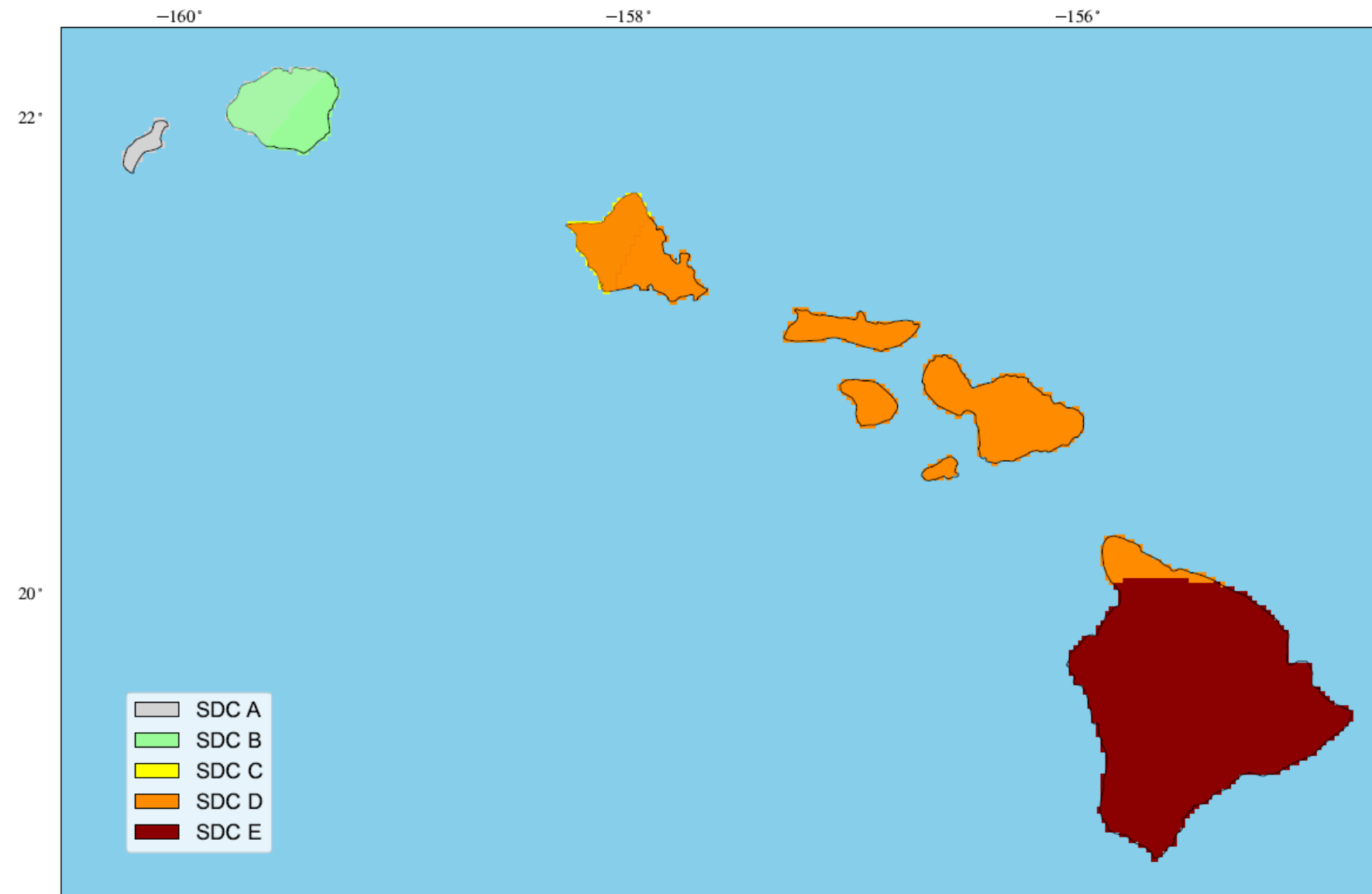


FIGURE 11.6-1c SDC MAP FOR RISK CATEGORY I, II AND III STRUCTURES

FIGURE 11.6-1d SDC MAP FOR RISK CATEGORY I, II AND III STRUCTURES

US TERRITORIES NOT SHOWN

PUERTO RICO – SDC D

US VIRGIN ISLANDS – SDC D

NORTHERN MARIANA ISLANDS – SDC C

GUAM – SDC D

AMERICAN SAMOA – SDC D

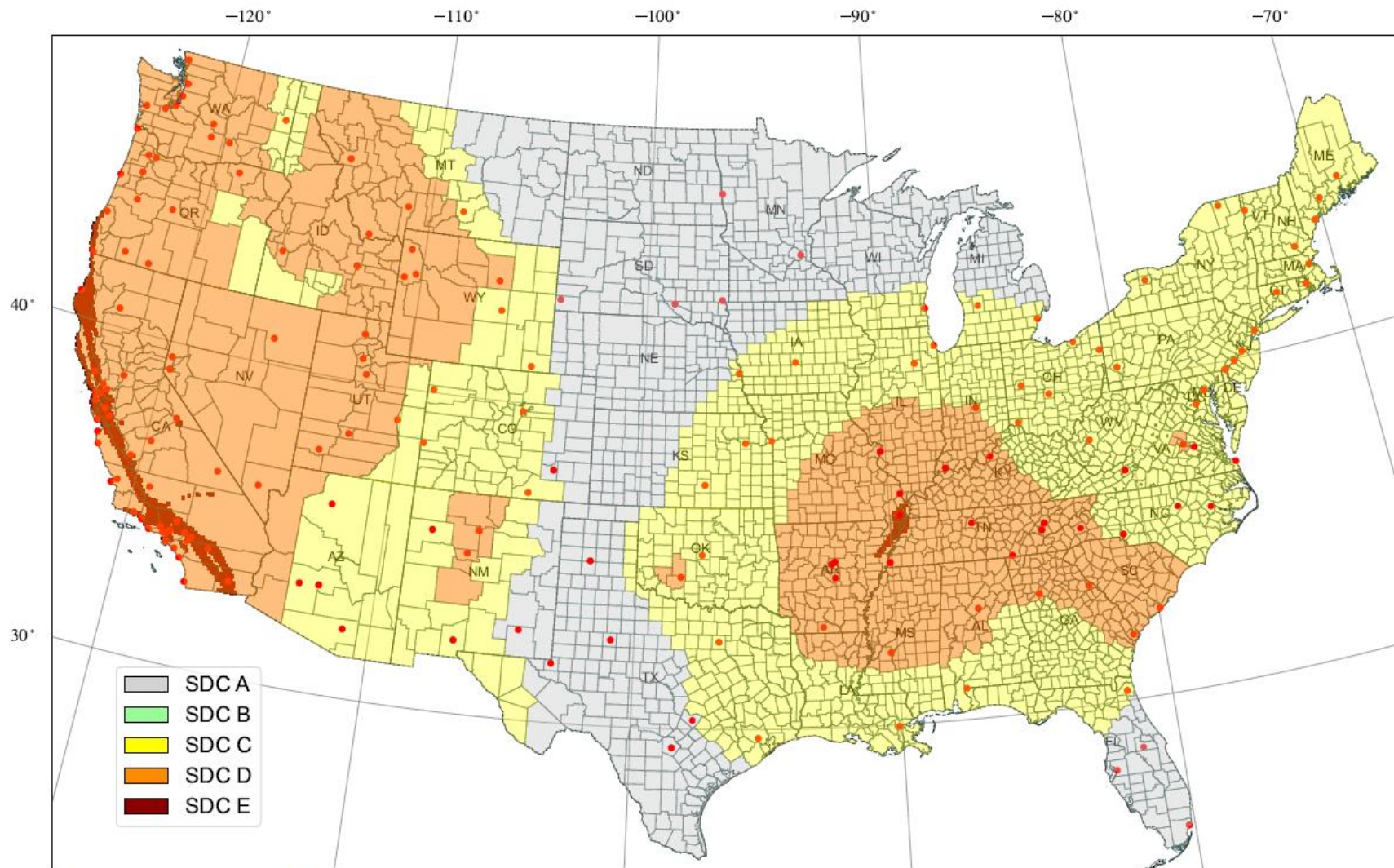


FIGURE 11.6-2a SDC MAP FOR RISK CATEGORY IV STRUCTURES

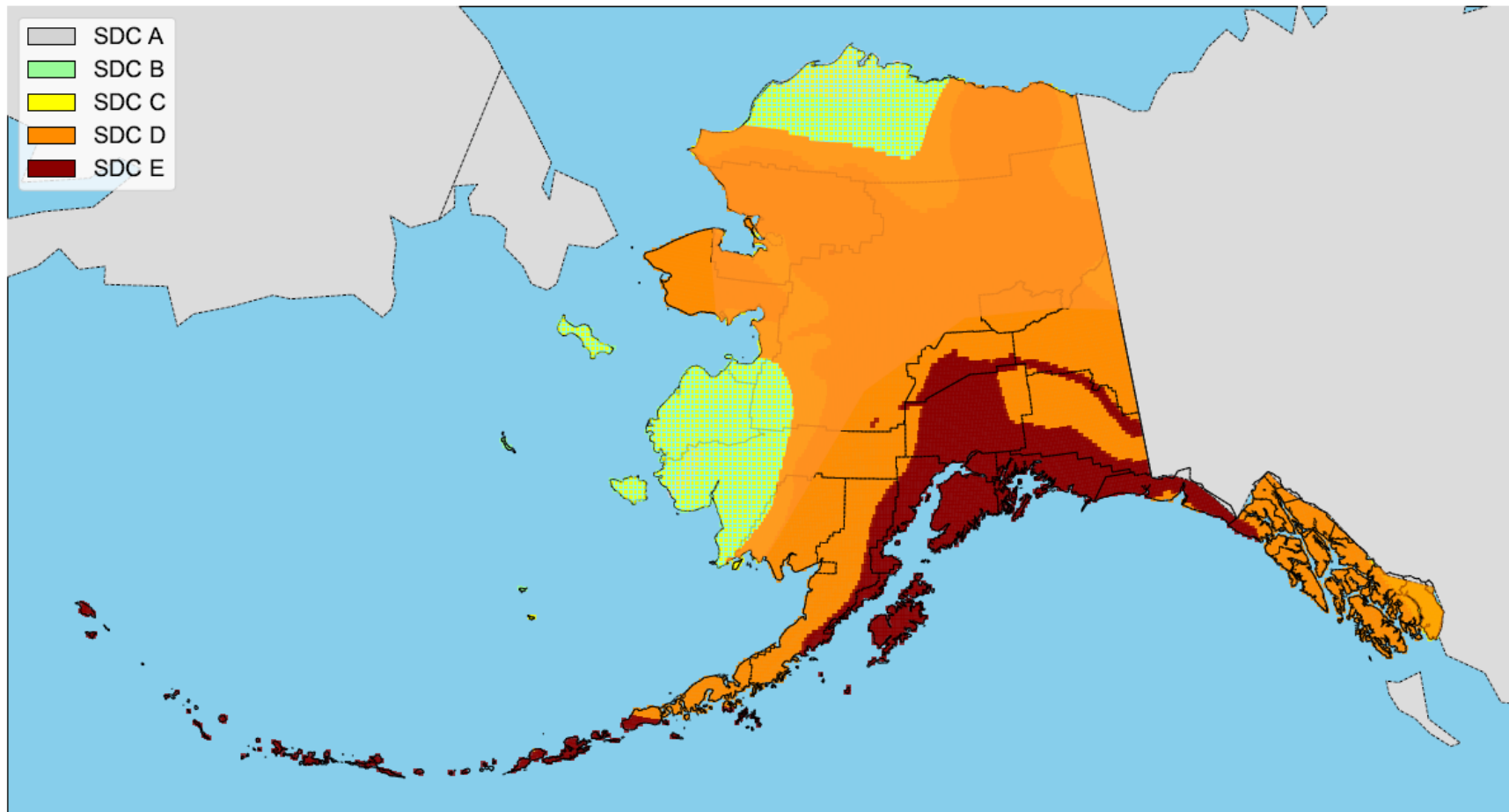


FIGURE 11.6-2b SDC MAP FOR RISK CATEGORY IV STRUCTURES

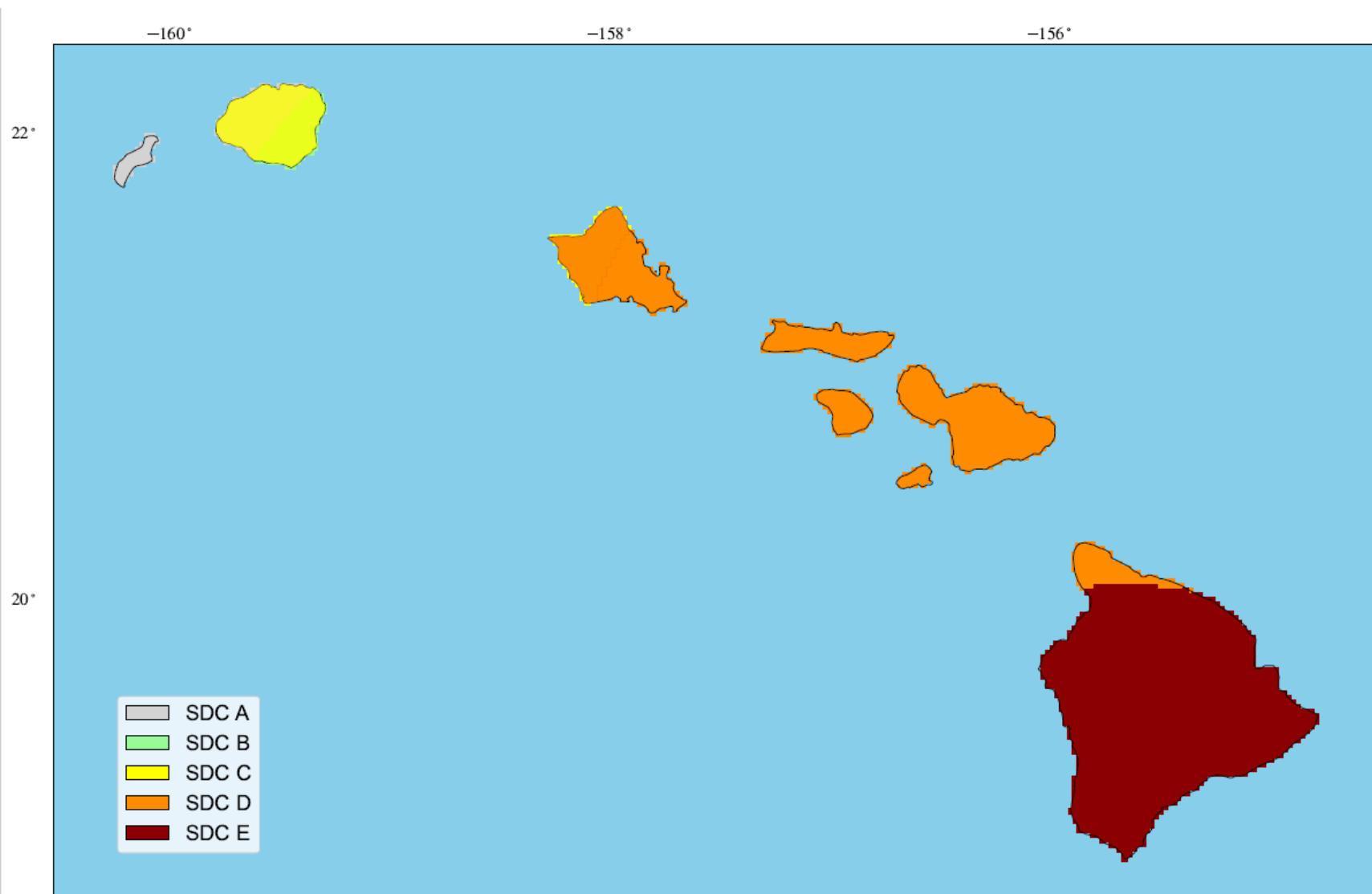


FIGURE 11.6-2c SDC MAP FOR RISK CATEGORY IV STRUCTURES

FIGURE 11.6-2d SDC MAP FOR RISK CATEGORY IV STRUCTURES

US TERRITORIES NOT SHOWN

PUERTO RICO – SDC D

US VIRGIN ISLANDS – SDC D

NORTHERN MARIANA ISLANDS – SDC C

GUAM – SDC D

AMERICAN SAMOA – SCD D

SECTION 11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

Revise Sections 11.8.1 and 11.8.2 as follows:

11.8.1 Site Limitation for Known Active Surface Fault Zones. For structures assigned to Seismic Design Category D, E, or F, the structure shall not be located within a Known Active Surface Fault Zone, unless the Maximum Considered Fault Displacement (MCFD) is determined in accordance with this section and the foundation design requirements of Section 12.13.10 are satisfied for the MCFD. The presence of a Known Active Surface Fault Zone need only be checked for structures assigned to Seismic Design Category D, E, or F using the default site conditions where the AHJ has specifically identified a Known Active Surface Fault Zone.

The MCFD shall be computed at the 84th-percentile if the fault slip rate is ≥ 0.9 mm/yr, or the 50th-percentile if the fault slip rate is ≤ 0.3 mm/yr. For intermediate fault slip rates between 0.3 mm/yr and 0.9 mm/yr, the value of MCFD shall be computed by linear interpolation. The 50th percentile is permitted to be used for any fault slip rate if the surface fault displacement is based upon a site geologic investigation. The fault rupture scenario used to evaluate displacements shall be selected by the Registered Design Professional (RDP) in consideration of the range of possible ruptures that could occur at the site and their frequencies of occurrence.

The geotechnical investigation report shall address surface fault rupture in accordance with Section 11.8.4. The portion of the geotechnical investigation report addressing surface fault rupture shall be subject to peer review where required by Section 12.13.10.3.

11.8.2 Geotechnical Investigation Report Requirements for

Seismic Design Categories C through F A geotechnical investigation report shall be provided for a structure assigned to Seismic Design Category C, D, E, or F in accordance with this section. An investigation shall be conducted, and a report shall be submitted that includes an evaluation of the following potential geologic and seismic hazards:

- (a) Slope instability,
- (b) Liquefaction,
- (c) Total and differential settlement, and
- (d) Surface displacement caused by seismically induced lateral spreading or lateral flow.

The report shall contain recommendations for foundation designs or other measures to mitigate the effects of the previously mentioned hazards.

Revise Section 11.8.3 as follows:

11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F The geotechnical investigation report for a structure assigned to Seismic Design Category D, E, or F shall include the following, as applicable:

1. The seismic lateral earth pressures on basement and retaining walls caused by design earthquake ground motions. For Risk Category III and IV structures, seismic lateral earth pressures shall be evaluated using design earthquake ground motions multiplied by 1.25 for Risk Category III and 1.5 for Risk Category IV, and the soil pressures resulting from these multiplied ground motions shall be used.
2. The potential for liquefaction evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the MCE_L peak ground acceleration. Peak ground acceleration shall be determined based on either (1) a site-specific study as specified in Section 11.4.7 and Section 21.5.4 or (2) the value of the MCE_L peak ground acceleration parameter PGA_L from the USGS Seismic Design Geodatabase for the applicable site class.
3. The potential for non-liquefaction-related soil hazards, such as seismically-induced permanent ground displacement not caused by liquefaction, evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the MCE_G peak ground acceleration. Peak ground acceleration shall be determined based on either (1) a site-specific study taking into account soil amplification effects as specified in Section 11.4.7 or (2) the value of the MCE_G peak ground acceleration parameter PGA_M from the USGS Seismic Design Geodatabase for the applicable site class, except for sites classified as Site Class DE or E located in the conterminous United States, east of -105 degrees longitude, where the values of PGA_M shall not be taken as less than the values of PGA_M determined for Site Class D.
4. Assessment of potential consequences of liquefaction, seismically-induced permanent ground displacement, and soil strength loss, including, but not limited to, estimation of total and differential settlement, lateral soil movement, lateral soil loads on foundations, reduction in foundation soil-bearing capacity and lateral soil reaction, soil downdrag, and reduction in axial and lateral soil reaction for pile foundations, increases in soil lateral pressures on retaining walls, and flotation of buried structures. Assessment of liquefaction-related consequences shall be evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the MCE_L peak ground acceleration, while non-liquefaction-related consequences shall be evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the MCE_G peak ground acceleration.
5. Discussion of mitigation measures such as, but not limited to the selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate

anticipated displacements and forces, ground stabilization, or any combination of these measures, and how they shall be considered in the design of the structure.

Add Section 11.8.4 as follows:

11.8.4 Geotechnical Investigation Report Requirements for Structures in Known Active Surface Fault Zones. For structures assigned to Seismic Design Categories D, E, or F using the default site conditions with foundations located within a Known Active Surface Fault Zone, the geotechnical investigation report shall provide the following information about the MCFD:

1. Whether principal fault traces exist at the structure and have likely been active within the Holocene. If principal faults are located within the plan envelope of the structure, their estimated location shall be provided. If no principal fault trace is located at the structure, whether and where distributed deformation is expected to occur at the site shall be provided.
2. The estimated slip rate of the fault traces onsite
3. The fault type (i.e., strike-slip, normal, reverse, or oblique)
4. The anticipated width and distribution of surface fault rupture (e.g., abruptness)
5. The orientation of ground movement (e.g., dip, strike, rake, and vertical and horizontal components of slip)
6. The portion of slip that is aseismic (i.e., fault creep)
7. The Maximum Considered Fault Displacement

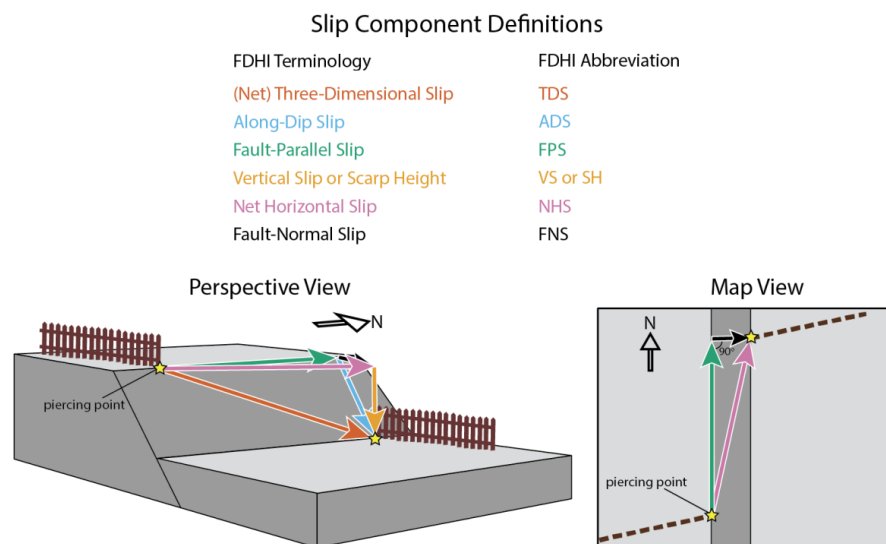


Figure 11.8-X. Fault displacement definitions (credit FDHI, 2024).

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CHAPTER 12, SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES (Modifications)

SECTION 12.2 STRUCTURAL SYSTEM SELECTION

Revise Table 12.2-1 by adding the following rows:

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n , Limits (ft) ^d				
					Seismic Design Category				
					B	C	D ^e	E ^e	F ^f
A. BEARING WALL SYSTEMS									
22. Post-tensioned timber rocking walls	14.5	8	2	8	NL	NL	180	180	100
B. BUILDING FRAME SYSTEMS									
29. Post-tensioned timber rocking walls	14.5	8	2	8	NL	NL	180	180	100
...									
C. MOMENT-RESISTING FRAME SYSTEMS									
...									
12. Cold-formed steel – special bolted moment frame	14.1	3 ½	3	3 ½	35	35	35	35	35
13. Steel Damped Special Moment Frame	14.1	8	3.0	4.5	300	300	300	300	300

Seismic Force-Resisting System	Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_o^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n , Limits (ft) ^d				
					Seismic Design Category				
					B	C	D ^e	E ^e	F ^f
I. UNCATEGORIZED STRUCTURAL SYSTEMS	12.1 and 12.2.1.3	0.5	1	0.5	65 ^z	65 ^z	65 ^z	65 ^z	65 ^z

^d NL = Not Limited...

^z An increase in structural height to 160 ft is permitted for Linear Dynamic Analysis.

Revise Section 12.2.1.1 as follows:

12.2.1.1 Alternative Structural Systems Use of seismic force-resisting systems not contained in Table 12.2-1 shall be permitted contingent on submittal to and approval by the Authority Having Jurisdiction and independent structural design review of an accompanying set of design criteria and substantiating analytical and test data. The design criteria shall specify any limitations on system use, including Seismic Design Category and height; required procedures for designing the system's components and connections; required detailing; and the values of the response modification coefficient, R ; overstrength factor, Ω_0 ; and deflection amplification factor, C_d . The submitted data shall establish the system's nonlinear dynamic characteristics and demonstrate that the design criteria result in a probability of collapse conditioned on the occurrence of MCE_R shaking not greater than 10% for Risk Category II structures. The conditional probability of collapse shall be determined based on a nonlinear analytical evaluation of the system and shall account for sources of uncertainty in the quality of the design criteria, modeling fidelity, laboratory test data, and ground motions. Structural design review shall conform to the criteria of Section 16.5. The conditional probability of collapse shall be calculated using MCE_R ground motions as defined in either Section 12.2.1.1.1 for generic applications or Section 12.1.1.1.2 for a project-specific application of the alternative structural system.

12.2.1.1.1 Generic Applications For generic applications of the alternative structural system, the conditional probability of collapse shall be calculated using MCE_R ground motions corresponding to 1.5 times the strongest intensity of shaking defined by Table 11.6-1 and Table 11.6-2.

EXCEPTION: MCE_R ground motions corresponding to 1.5 times the strongest intensity of shaking defined by Table 11.6-1 and Table 11.6-2 for Seismic Design Category C shall be permitted for calculation of the conditional probability of collapse of an alternative structural system that is designated as Not Permitted for Seismic Design Category D, E or F.

12.2.1.1.2 Project-Specific Application For a project-specific application of the alternative structural system, the conditional probability of collapse shall be calculated using the MCE_R ground motions determined in accordance with Section 11.4 for the site of the project-specific application.

Add Section 12.2.1.3 as follows:

12.2.1.3 Uncategorized Structural Systems

The use of the Seismic Force-Resisting System named Uncategorized Structural Systems shall be permitted when it complies with the following requirements:

1. The design coefficients (R , Ω_0 , and C_d) for uncategorized structural systems contained in Table 12.2-1 shall apply.
2. For foundation design:

- (a) The foundation is permitted to be proportioned for the global stability limit states of sliding and overturning using R equal to 2.0. Sliding resistance shall be permitted to include both soil-to-foundation friction and passive soil resistance.
 - (b) The strength of the foundation elements of reinforced concrete satisfying Section 14.2 or structural steel satisfying Section 14.1 shall be permitted to be designed for forces derived from R equal to 2.0. Section 14.1.3 of ACI 318 shall be modified to include a structure assigned to Seismic Design Category B or C.
 - (c) The connection of the SFRS to the foundation shall resist the forces from the superstructure using R tabulated in Table 12.2-1.
 - (d) Where building foundations are not interconnected by a system capable of enforcing equal lateral displacements, the evaluation of the structure for drift-limit compliance, deformation compatibility, and stability shall include the potential differential lateral movement of the foundations. Additional rotation and displacement resulting from uplift or rocking shall be considered in the evaluation of the structure for drift-limit compliance, deformation compatibility, and stability.
- Exception: The additional displacement due to differential lateral movement and vertical/rotation movement need not be considered if the structure remains stable when evaluated with R equal to 1.0.
- 3. The redundancy factor, ρ , shall be taken equal to 1.0.
 - 4. Section 12.10.2 System-Based Diaphragm Design Method shall be permitted with R_s not taken greater than 1.0.
 - 5. Structural walls shall be designed for a minimum force F_p no less than W_p times F_{px}/w_{px} where F_{px} and w_{px} are the controlling values from Section 12.10, except Equation (12.10-3) shall not apply. For structural walls attached to flexible diaphragms, the minimum force shall be increased by the amplification factor, k_a , from Section 12.11.2.1.
 - 6. Nonstructural components shall be evaluated using the larger of:
 - (a) Chapter 13 requirements with R_μ of 1.0 in lieu of Equation (13.3-6).
 - (b) A minimum force F_p no less than W_p times $[(F_{px}/w_{px})(C_{AR}/R_{po})]$ where F_{px} and w_{px} are the controlling values from Section 12.10, except Equation (12.10-3) shall not apply.

Revise Section 12.2.3.2 as follows:

12.2.3.2 Two-Stage Analysis Procedure for Vertical Combinations of Systems. A two-stage equivalent lateral force procedure is permitted to be used for structures that have a flexible upper portion above a rigid lower portion, provided the design of the structure complies with all of the following.

- (g) Where Horizontal Irregularity Type 4 or Vertical Irregularity Type 3 exists at the transition from the upper to the lower portion, the reactions from the upper portion shall be amplified in accordance with Sections 12.3.3.4, 12.10.1.1, and 12.10.2.3, in addition to amplification required by Item (d).

Revise Section 12.2.5.6.1 as follows:

12.2.5.6.1 Seismic Design Category D or E

- (a) Single-story steel ordinary moment frames in structures assigned to Seismic Design Category D or E are permitted up to a structural height, h_n , of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 lb/ft² (0.96 kN/m²). In addition, the dead load of the exterior walls more than 35 ft (10.6 m) above the base tributary to the moment frames shall not exceed 20 lb/ft² (0.96 kN/m²).

EXCEPTION:

Single-story structures with steel ordinary moment frames whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery, or their associated processes shall be permitted to be of unlimited height where the sum of the dead and equipment loads supported by and tributary to the roof does not exceed 20 lb/ft² (0.96 kN/m²). In addition, the dead load of the exterior wall system, including exterior columns more than 35 ft (10.6 m) above the base, shall not exceed 20 lb/ft² (0.96 kN/m²). For determining compliance with the exterior wall or roof load limits, the weight of equipment or machinery, including cranes, not self-supporting for all loads shall be assumed to be fully tributary to the area of the adjacent exterior wall or roof not to exceed 600 ft² (55.8 m²), regardless of its height above the base of the structure.

- (b) Steel ordinary moment frames in structures assigned to Seismic Design Category D or E not meeting the limitations set forth in Section 12.2.5.6.1.a are permitted within light-frame construction up to a structural height, h_n , of 35 ft (10.6 m) where neither the roof dead load nor the dead load of any floor above the base supported by and tributary to the moment frames exceeds 35 lb/ft² (1.68 kN/m²). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 lb/ft² (0.96 kN/m²).

- (c) Single-story steel ordinary moment frames in structures assigned to Seismic Design Category D or E not meeting the limitations set forth in Section 12.2.5.6.1(a) are permitted up to a structural height, h_n , of 45 ft (10.6 m) where:

- (1) neither the roof dead load nor the dead load of any floor above the base supported by and tributary to the moment frames exceeds 30 lb/ft² (1.44 kN/m²),
- (2) the dead load for the exterior walls shall not exceed 120 lb/ft² (5.76 kN/m²), and
- (3) the flange and web are proportioned such that $h/b_f \leq 6$ and $A_w/A_f \leq 2.5$ are satisfied,

where, h = web depth; b_f = flange width; A_w = web area; A_f = flange area

SECTION 12.3 DIAPHRAGM FLEXIBILITY, CONFIGURATION IRREGULARITIES, AND REDUNDANCY

Revise Section 12.3.1.3 as follows:

12.3.1.3 Calculated Flexible Diaphragm Condition

Diaphragms not satisfying the conditions of Sections 12.3.1.1 or 12.3.1.2 are permitted to be idealized as flexible provided

$$\frac{\delta_{MDD}}{\Delta_{ADVE}} > 2 \quad (12.3-1)$$

where δ_{MDD} and Δ_{ADVE} are as shown in Figure 12.3-1. The loading used in this calculation shall be prescribed in Section 12.3.3 with the distribution of force, F_x , to the diaphragm accounting for the position and distribution of the masses supported. Δ_{ADVE} shall be computed under tributary lateral loading equivalent to that used in the computation of δ_{MDD} in Equation 12.3-1. δ_{MDD} and Δ_{ADVE} are computed without the use of deflection amplification factors, C_d or $C_{d-diaph}$.

Revise Section 12.3.3.5 as follows:

12.3.3.5 Increase in Forces caused by Irregularities for Seismic Design Categories D through F For structures assigned to Seismic Design Category D, E, or F and having a horizontal structural irregularity of Type 1, 2, 3, or 4 in Table 12.3-1 or a vertical structural irregularity of Type 3 in Table 12.3-2, the design forces determined from Section 12.10.1.1 shall be increased 25% at each diaphragm level where the irregularity occurs for the following elements of the seismic force-resisting system.

1. Connections of diaphragms to vertical elements and to collectors, and
2. Collectors and their connections, including connections to vertical elements of the seismic force-resisting system.

EXCEPTION: Forces calculated using the amplified seismic collector forces of Section 12.10.1.2, or diaphragm transfer forces required to be designed using the seismic load effects including overstrength of Section 12.4.3 need not be further increased in accordance with this section.

Revise Section 12.3.4.1 as follows:

12.3.4.1 Conditions Where Value of p is 1.0 The value of p is permitted to equal 1.0 for the following:

1. Structures assigned to Seismic Design Category B or C;

2. Drift calculation and P-delta effects;
3. Design of nonstructural components;
4. Design of nonbuilding structures that are not like buildings;
5. Design of collector elements, splices, and their connections for which the seismic load effects including overstrength of Section 12.4.3 are used;
6. Design of members or connections where the seismic load effects, including overstrength of Section 12.4.3, are required for design;
7. Diaphragm seismic design forces determined using F_{px} forces in accordance with Equation (12.10-1), including the limits imposed by Equations (12.10-2) and (12.10-3). This is not applicable to diaphragm design forces based on F_x forces;
8. Diaphragm seismic design forces determined using F_{px} forces in accordance with Equation (12.10-4), including the limits imposed by Equation (12.10-5). This is not applicable to diaphragm design forces based on F_x forces;
9. Diaphragm seismic design forces determined in accordance with Section 12.10.3;
10. Structures with damping systems designed in accordance with Chapter 18 or Section 14.1.12; and
11. Design of structural walls for out-of-plane forces, including their anchorage.

SECTION 12.7 MODELING CRITERIA

Revise Section 12.7.1 as follows:

12.7.1 Foundation Modeling For purposes of determining seismic force and deformation demands on the building structure and its foundations, it is permitted to consider the structure to be fixed at the base subject to limitations in this section. Where flexibility associated with the soil supporting the foundations is considered, it shall be in accordance with Section 12.13.3 or Chapter 19.

Buildings in Seismic Design Category D through F on site class CD through F shall consider soil flexibility as required by Section 12.7.1.1.

12.7.1.1 Conditions where Lateral Soil Flexibility is Required to be Modeled

Lateral soil flexibility, which models foundation translational resistance, shall be included in the analysis model where either of the following apply:

1. The foundation elements resisting seismic base shear consist of a mix of deep and shallow foundation elements, where each foundation type resists more than 20% of the base shear in either of the two orthogonal directions.
2. The calculated lateral displacement of any pile cap resisting seismic base shear under the design seismic forces in a given direction is more than two times the minimum calculated lateral displacement at any other pile cap resisting seismic base shear in the same direction.

Exceptions:

Where all diaphragms are permitted to be idealized as flexible in accordance with Section 12.3.1.1, lateral soil flexibility need not be modeled.

Revise Section 12.7.3 and add Section 12.7.3.1 as follows:

12.7.3 Structural Modeling A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or P-delta effects. The model(s) shall include all above grade levels and any below grade levels. The model shall include the stiffness and strength of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass above the base and stiffness throughout the structure.

Exception(s):

- 1) Structures assigned to Seismic Design Category B need not include levels below the base in their analysis model.
- 2) Structures assigned to Seismic Design Category C need not include levels below the base in their analysis model unless liquefiable soils are present within the depth of the basement levels.

In addition, the model shall comply with the following:

1. Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections.
2. For steel moment frame systems, the contribution of panel zone deformations to displacement and drift shall be included.

Structures that have horizontal structural irregularity Type 1, 4, or 5 of Table 12.3-1 shall be analyzed using a 3D representation. Where a 3D model is used, a minimum of three degrees of freedom consisting of translation in two orthogonal plan directions and rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms have not been classified as rigid or flexible in accordance with Section 12.3.1, the model shall include representation of the diaphragm's stiffness characteristics and, when dynamic analysis is performed, sufficient degrees of

freedom as are required to account for the participation of the diaphragm in the structure's dynamic response. When modal response spectrum or response history analysis is performed, a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis at each level of the structure shall be used.

EXCEPTION: Analysis using a 3D representation is not required for structures with flexible diaphragms that have Type 4 horizontal structural irregularities.

12.7.3.1 Base of Structure The base of structure shall be taken at an elevation no higher than the interior diaphragm nearest the lowest exterior ground surface. For sloping sites, the base of structure shall be taken no higher than the diaphragm at the side with the lowest adjacent grade.

Exception: For sites with liquefiable soils, the base of structure shall not be taken as higher than the slab elevation below the liquefiable soils regardless of adjacent grade. Where liquefiable soils extend below the structure the base of structure shall be taken as the lowest slab elevation.

Add Section 12.7.5 as follows:

12.7.5 Design of below-grade elements For structures assigned to Seismic Design Category D through F, all earth retaining elements shall be designed for the seismic lateral earth pressures as specified by the geotechnical investigation report. Elements are permitted to be designed for seismic lateral earth pressures as either yielding or nonyielding walls based on their support conditions. Cantilevered walls shall be assumed to be yielding walls.

Where the seismic earth pressures produce a net force on the structure, the Seismic Force Resisting System shall be designed for the seismic lateral earth pressures in combination with the lateral seismic forces from the structure as shown in Equation 12.7-1.

$$E = (F_i^2 + F_{SE}^2)^{0.5} \quad \text{EQ 12.7-1}$$

Where

F_{SE} = Forces on element from seismic lateral earth pressures

F_i = Forces on element from the lateral seismic forces from the structure per section 12.8.3

SECTION 12.8 EQUIVALENT LATERAL FORCE PROCEDURE

Revise Section 12.8.3 as follows:

12.8.3 Vertical Distribution of Seismic Forces The lateral seismic force, F_x (kip or kN), induced at any level above the base shall be determined from the following equations:

$$F_x = C_{vx} V \quad (12.8-12)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (12.8-13)$$

where

C_{vx} = Vertical distribution factor;

V = Total design lateral force or shear at the base of the structure [kip (kN)];

$w_i w_x$ = Portion of the total effective seismic weight of the structure (W) located or assigned to level i or x ;

h_i, h_x = Height [ft (m)] from the base to level i or x ; and

k = Exponent related to the structure period as follows:

For structures that have a period of 0.5 s or less, $k = 1$;

for structures that have a period of 2.5 s or more, $k = 2$; and

for structures that have a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

SECTION 12.10 DIAPHRAGMS, CHORDS, AND COLLECTORS

Revise Section 12.10 as follows:

The provisions of this section shall apply to the seismic design of diaphragms. Provisions applicable to diaphragms shall also be applicable to chords, collectors, collector connections to vertical elements, and similar parts and portions of the diaphragm system.

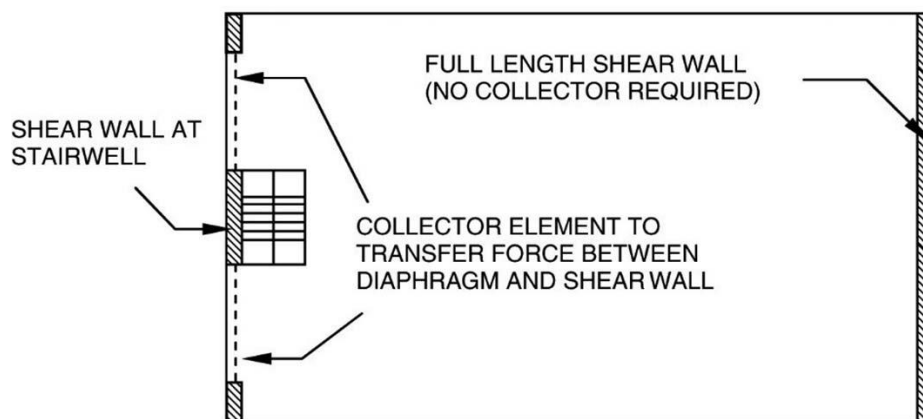
Seismic design of diaphragms shall be in accordance with one of the following methods, within the noted limitations:

1. The non-system-based diaphragm design method of Section 12.10.1, permitted for all diaphragms except in structures assigned to Seismic Design Category C, D, E, or F as follows:
 - a. Prohibited for precast concrete diaphragms.

- b. Prohibited for bare steel deck diaphragms with welded connections between the steel deck and any supporting members.
- 2. The system-based diaphragm design method of Section 12.10.2, permitted for all structures except as follows:
 - a. Prohibited for diaphragm systems not listed in and conforming to Table 12.10-1.
- 3. The one-story diaphragm design method of Section 12.10.3, permitted for one-story structures with flexible diaphragms and rigid vertical elements conforming to the limitations of Section 12.10.3.1.

Collector elements (Figure 12.10-1) shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces.

At diaphragm discontinuities, such as openings and reentrant corners, the design shall ensure that the dissipation or transfer of edge (chord) forces, combined with other forces in the diaphragm, is within the capacity of the diaphragm.



4. Figure 12.10-1. Collectors.

Revise Sections 12.10.1 and 12.10.2 as follows:

12.10.1 Non-System-Based Diaphragm Design Method Diaphragms shall be designed for both the shear and the bending stresses resulting from design forces.

12.10.1.1 Diaphragm Design Forces

Floor and roof diaphragms shall be designed to resist in-plane seismic design forces, F_x , determined in accordance with 12.8.3 but the design forces shall not be less than F_{px} forces determined in accordance with the following:

$$F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px} \quad (12.10-1)$$

Where

F_{px} = Diaphragm design force at level x ;

F_i = Design force applied to level i ;

w_i = Weight tributary to level i ; and

w_{px} = Weight tributary to the diaphragm at level x .

The force determined from Equation (12.10-1) shall not be less than

$$F_{px} = 0.2S_{DS}I_e w_{px} \quad (12.10-2)$$

The force determined from Equation (12.10-1) need not exceed

$$F_{px} = 0.4S_{DS}I_e w_{px} \quad (12.10-3)$$

In addition to inertial forces, diaphragms shall be designed to resist transfer forces resisted by the diaphragm between vertical seismic force-resisting elements. Transfer forces associated with Horizontal Irregularity Type 4 shall be increased by the overstrength factor of Section 12.4.3 before being added to the diaphragm inertial forces. For structures that have horizontal or vertical structural irregularities of the types indicated in Section 12.3.3.5, the requirements of that section shall also apply.

EXCEPTION: One- and two-family dwellings of light-frame construction shall be permitted to use $\Omega_0=1.0$.

12.10.1.2 Collector Elements for Structures Assigned to Seismic Design Categories C through F

In structures assigned to Seismic Design Category C, D, E, or F, collector elements (Figure 12.10-1) and their connections, including connections to vertical elements, shall be designed to resist the maximum of the following:

(a) Forces calculated using the seismic load effects including overstrength of Section 12.4.3 with seismic forces determined by the equivalent lateral force procedure of Section 12.8 or the modal response spectrum analysis procedure of Section 12.9.1;

(b) Forces calculated using the seismic load effects including overstrength of Section 12.4.3 with seismic forces determined by Equation (12.10-1); and

(c) Forces calculated using the load combinations of Section 2.3.6 with seismic forces determined by Equation (12.10-2).

Transfer forces as described in Section 12.10.1.1 shall be considered.

EXCEPTION: In structures or portions thereof braced entirely by wood light-frame shear walls, collector elements and their connections, including connections to vertical elements, need only be designed to resist forces using the load combinations of Section 2.3.6 with seismic forces determined in accordance with Section 12.10.1.1.

Renumber Section 12.10.3 as Section 12.10.2 and revise as follows:

12.10.2 System-Based Diaphragm Design Method Diaphragms, including chords and collectors, shall be designed using the provisions in Sections 12.10.2.1 through 12.10.2.5 and the following:

1. Footnote *b* to Table 12.2-1 shall not apply.
2. Section 12.3.3.5 shall not apply.
3. See Section 12.3.4.1 Item 8 for conditions where ρ is permitted to be set to 1.0.

12.10.2.1 Design Diaphragms, including chords, collectors, and their connections to the vertical elements, shall be designed in two orthogonal directions to resist the sum of the in-plane design seismic forces determined in Sections 12.10.2.2 and 12.10.2.3.

12.10.2.2 Diaphragm Inertial Seismic Design Forces Diaphragms shall be designed to resist inertial in-plane seismic design forces, F_x , determined from the structural analysis but the design forces shall not be taken less than F_{px} forces, as determined below.

$$F_{px} = \frac{C_{px}}{R_s} w_{px} \quad (12.10-4)$$

The force F_{px} determined from Equation (12.10-4) shall not be taken as less than:

$$F_{px} = 0.2 S_{DS} I_e w_{px} \quad (12.10-5)$$

The design acceleration coefficient, C_{px} , shall be determined as illustrated in Figure 12.10-2(a) for buildings with $N \leq 5$, or as illustrated in Figure 12.10-2(b) for buildings with $N \geq 3$. Buildings with $3 \leq N \leq 5$ are permitted to calculate C_{px} with either Figure 12.10-2(a) or Figure 12.10-2(b).

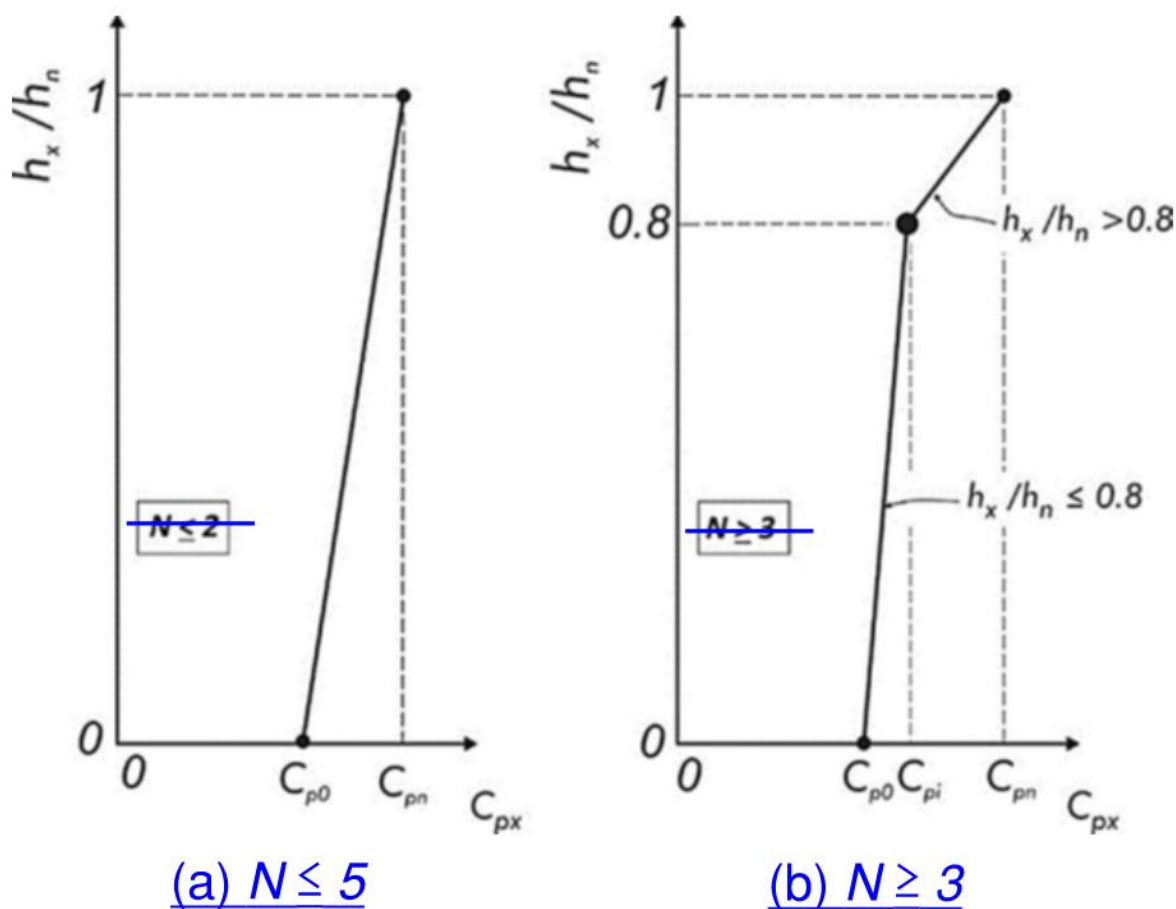


Figure 12.10-2. Design acceleration coefficient, C_{px} , determination.

12.10.2.2.1 Design Acceleration Coefficients C_{p0} , C_{pi} , and C_{pn} Design acceleration coefficient

C_{p0} shall be calculated by Equation (12.10-6):

$$C_{p0} = 0.4S_{DS}I_e \quad (12.10-6)$$

Design acceleration coefficient C_{pn} shall be calculated by Equation (12.10-7):

$$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_0C_s)^2 + (\Gamma_{m2}C_{s2})^2} \geq C_{pi} \quad (12.10-7)$$

Alternatively, for buildings with $N \leq 5$ and $T \leq T_s$, the design acceleration coefficient C_{pn} is permitted to be calculated by Equation (12.10-8):

$$C_{pn} = C_s \sqrt{\left[\left(1.5 - \frac{1}{2N} \right) \Omega_0 \right]^2 + [(0.14 - 0.14N)R]^2} \geq C_{po} \quad (12.10-8)$$

Design acceleration coefficient, C_{pi} , shall be the greater of the values given by Equations (12.10-9) and (12.10-10):

$$C_{pi} = C_{po} \quad (12.10-9)$$

$$C_{pi} = 0.9 \Gamma_{m1} \Omega_0 C_s \quad (12.10-10)$$

where Ω_0 is the overstrength factor given in Table 12.2-1; C_s is determined in accordance with Section 12.8 or 12.9; and C_{s2} shall be the smallest of the values calculated from Equations (12.10-11), (12.10-12), and (12.10-13).

$$C_{s2} = (0.15N + 0.25) I_e S_{DS} \quad (12.10-11)$$

$$C_{s2} = I_e S_{DS} \quad (12.10-12)$$

$$\text{For } N \geq 2 \quad C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} \quad (12.10-13a)$$

$$\text{For } N = 1 \quad C_{s2} = 0 \quad (12.10-13b)$$

The modal contribution factors Γ_{m1} and Γ_{m2} in Equation (12.10-7) shall be calculated from Equations (12.10-14) and (12.10-15).

$$\Gamma_{m1} = 1 + \frac{z_s}{2} \left(1 - \frac{1}{N} \right) \quad (12.10-14)$$

and

$$\Gamma_{m2} = 0.9 z_s \left(1 - \frac{1}{N} \right)^2 \quad (12.10-15)$$

Where the mode shape factor, z_s , is to be taken as 0.3 for buildings designed with buckling-restrained braced frame systems defined in Table 12.2-1; 0.7 for buildings designed with moment-resisting frame systems defined in Table 12.2-1; 0.85 for buildings designed with dual systems defined in Table 12.2-1 with special or intermediate moment frames capable of resisting at least 25% of the prescribed seismic forces; and 1.0 for buildings designed with all other seismic force-resisting systems.

12.10.2.3 Diaphragm Transfer Forces In addition to the inertial seismic design forces of Section 12.10.2.2, diaphragms shall be designed for transfer forces acting through the diaphragm between vertical seismic force-resisting elements. Diaphragm transfer forces shall be derived from the F_x forces used in design of the vertical elements.

Diaphragm transfer forces associated with Horizontal Irregularity Type 4 shall be increased by the overstrength factor of Section 12.4.3 before being added to the diaphragm inertial forces.

EXCEPTION: One- and two-family dwellings of light-frame construction shall be permitted to use $\Omega_0=1.0$.

12.10.2.4 Collectors — Seismic Design Categories C through F

In structures assigned to Seismic Design Category C, D, E, or F, collectors and their connections, including connections to vertical elements, shall be designed to resist the diaphragm inertial forces from Section 12.10.2.2 plus the design transfer forces from Section 12.10.2.3, each amplified by the greater of R_s or 1.5.

EXCEPTIONS:

1. Any transfer force required by Section 12.10.2.3 to be increased by the overstrength factor of Section 12.4.3 need not be further amplified as required by this section.
2. For moment frame and braced frame systems, collector forces need not exceed the lateral strength of the corresponding frame line below the collector, considering only the moment frames or braced frames. In addition, diaphragm design forces need not exceed the forces corresponding to the collector forces so determined.
3. In structures or portions thereof braced entirely by light-frame shear walls, collector elements and their connections, including connections to vertical elements, need only be designed to resist the diaphragm seismic design forces, without amplification as required by this section.
4. Capacity-limited collector forces, as described in Section 12.4.3.2, shall be permitted where they can be demonstrated to be lower than otherwise required by this section.

12.10.2.5 Diaphragm Design Force Reduction Factor

The diaphragm design force reduction factor, R_s , shall be determined in accordance with Table 12.10-1. The value of R_s used for design shall not be greater than the least value of R_s for any diaphragm system at the level under consideration.

EXCEPTION: Where multiple diaphragm systems occur at a single level, the value of R_s used for design is permitted to be the R_s for the diaphragm system for each diaphragm span, where it can be demonstrated that the diaphragms are able to deflect independently of each other under a given seismic loading direction.

Table 12.10-1. Diaphragm Design Force Reduction Factor, R_s .

Diaphragm System		R_s
1. Cast-in-place concrete designed in accordance with ACI 318	Shear controlled	1.5
	Flexure controlled	2.0
2. Precast concrete designed in accordance with ACI 318	Elastic design option	0.7
	Basic design option	1.0
	Reduced design option	1.4
3. Wood sheathed designed in accordance with Section 14.5 and AWC SDPWS	-	3.0
4. Cross-laminated Timber diaphragm designed in accordance with Section 14.5 and AWC SDPWS	-	2.5
5. Bare steel deck diaphragm designed in accordance with Section 14.1.5	With special seismic detailing	2.5
	Other	1.0
6. Concrete-filled steel deck diaphragm designed in accordance with Section 14.1.6	-	2.0
7. Rod-braced diaphragm designed in accordance with Section 14.1.10	With special seismic detailing	2.5
	Other	1.0

Renumber Section 12.10.4 as Section 12.10.3 and revise as follows:

12.10.3 One-Story Diaphragm Design Method for Structures with Flexible Diaphragms and Rigid Vertical Elements

Where permitted by Section 12.10 and subject to the limitations of Section 12.10.3.1, diaphragm design forces, including design forces for chords, collectors, and their in-plane connections to vertical elements, shall be determined in accordance with Section 12.10.3.2.

12.10.3.1 Limitations

Diaphragms in one-story structures are permitted to be designed in accordance with Section 12.10.3 provided all of the following limitations are satisfied.

1. All portions of the diaphragm shall be designed using the provisions of this section in both orthogonal directions.
2. The diaphragm shall consist of either (a) a wood structural panel diaphragm designed in accordance with AWC SDPWS and fastened to wood framing members or wood nailers with sheathing nailing in accordance with the AWC SDPWS Section 4.2 nominal shear capacity tables, or (b) a bare (untopped) steel deck diaphragm meeting the requirements of AISI S400 and AISI S310.
3. Toppings of concrete or similar materials that affect diaphragm strength or stiffness shall not be placed over the wood structural panel or bare steel deck diaphragm.
4. The diaphragm shall not contain horizontal structural irregularities, as specified in Table 12.3-1, except that Horizontal Structural Irregularity Type 2 is permitted.
5. The diaphragm shall be rectangular in shape or shall be divisible into rectangular segments for purpose of seismic design, with vertical elements of the seismic force-resisting system or collectors provided at each end of each rectangular segment span.
6. The vertical elements of the seismic force-resisting system shall be limited to one or more of the following: concrete shear walls, precast concrete shear walls, masonry shear walls, steel concentrically braced frames, steel and concrete composite braced frames, or steel and concrete composite shear walls.
7. The vertical elements of the seismic force-resisting system shall be designed in accordance with Section 12.8, except that they shall be permitted to be designed using the two-stage analysis procedure of Section 12.2.3.4, where applicable.

12.10.3.2 Design

Diaphragms, including chords, collectors, and their connections to vertical elements, shall be designed in two orthogonal directions to resist the in-plane design seismic forces determined in accordance with this section. Multi-span diaphragms and diaphragms that are not rectangular in shape shall be divided into rectangular segments for purposes of design in accordance with this section, with lateral support provided at each end of each diaphragm segment span by a vertical element or collector element.

12.10.3.2.1 Seismic Design Forces The diaphragm seismic design force, F_{px} , shall be determined in accordance with Equation (12.10-16).

$$F_{px} = C_{s-\text{diaph}} (w_{px}) \quad (12.10-16)$$

where w_{px} = The effective seismic weight tributary to the diaphragm, and

$C_{s-\text{diaph}}$ is calculated as

$$C_{s\text{-diaph}} = \frac{S_{DS}}{\frac{R_{\text{diaph}}}{I_e}} \quad (12.10-17a)$$

and need not be greater than

$$C_{s\text{-diaph}} = \frac{S_{D1}}{T_{\text{diaph}} \left(\frac{R_{\text{diaph}}}{I_e} \right)} \quad (12.10-17b)$$

where

S_{DS} = Design spectral response parameter in the short period range as determined from Section 11.4.4 or 11.4.7;

R_{diaph} = 4.5 for wood structural panel diaphragms, 4.5 for bare steel deck diaphragms that meet the special seismic detailing requirements of AISI S400, and 1.5 for all other bare steel deck diaphragms;

I_e = Importance Factor determined in accordance with Section 11.5.1; and

$T_{\text{diaph}} = 0.002L_{\text{diaph}}$ (.00656 L_{diaph} for L_{diaph} in m.) for wood structural panel diaphragms, or $0.001L_{\text{diaph}}$ (.00328 L_{diaph} for L_{diaph} in m.) for profiled steel deck panel diaphragms, determined for each rectangular segment of the diaphragm in each orthogonal direction [seconds].

12.10.3.2.2 Diaphragm Shears

Diaphragm design shears shall be computed for each diaphragm segment in accordance with Section 12.10.3.2.1. Where the diaphragm segment span, L_{diaph} , is less than 100 ft (30.5 m), the diaphragm design shear, from loading perpendicular to the span, shall be the diaphragm shear calculated using the F_{px} forces of Section 12.10.3.2.1 multiplied by 1.5. Where the diaphragm segment span, L_{diaph} , is greater than or equal to 100 ft (30.5 m), the diaphragm design shear shall be amplified to 1.5 times the shear calculated using the F_{px} forces of Section 12.10.3.2.1 over an amplified shear boundary zone having a minimum width of 10% of the diaphragm segment span, L_{diaph} . The amplified shear boundary zone shall be provided at each supporting end of the diaphragm segment span under consideration.

12.10.3.2.3 Diaphragm Chords

Diaphragm chords shall be provided at each edge of each diaphragm segment to resist tension and compression forces resulting from diaphragm moments. Diaphragm chord forces shall be computed using the F_{px} forces of Section 12.10.3.2.1

12.10.3.2.4 Collector Elements and Their Connections

Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the vertical elements of the seismic force-resisting system. Collector element forces shall be computed using the F_{px} forces of Section 12.10.3.2.1 Collectors and their connections to vertical elements of the seismic force-resisting system in structures assigned to Seismic Design Categories C through F shall be designed to resist the forces calculated using the seismic load effects including overstrength factor of Section 12.4.3, with diaphragm overstrength factor, $\Omega_{0\text{-diaph}}$, taken equal to 2; however, $\Omega_{0\text{-diaph}}$ need not exceed R_{diaph} . This need not be combined with the shear amplification of 1.5 specified in Section 12.10.3.2.2.

12.10.3.2.5 Diaphragm Deflection

Where required by this standard, the deflection amplification factor, $C_{d\text{-diaph}}$, for diaphragm deflection shall be taken as one of the following:

$C_{d\text{-diaph}} = 3.0$ for wood structural panel diaphragms,

$= 3.0$ for bare steel deck diaphragms that meet the special seismic detailing requirements of AISI S400, and

$= 1.5$ for all other bare steel deck diaphragms.

Diaphragm deflections shall be calculated using the seismic design forces of Section 12.10.3.2.1.

12.10.3.2.6 Modifications to Diaphragm Seismic Design

For structures in which the diaphragm design forces are determined in accordance with Section 12.10.3, footnote *b* to Table 12.2-1 shall not apply, and Section 12.3.3.5 shall not apply.

SECTION 12.12 DRIFT AND DEFORMATION

Revise Section 12.12 as follows:

12.12 Drift and Deformation

12.12.1 Story Drift Limit The design story drift, Δ , as determined in Section 12.8.6, 12.9.1, or 12.9.2 shall not exceed the allowable story drift, Δ_a , as obtained from Table 12.12-1 for any story.

12.12.1.1 Moment Frames in Structures Assigned to Seismic Design Categories D through F For seismic force-resisting systems solely comprising moment frames in structures assigned to Seismic Design Category D, E, or F, the design story drift, Δ , shall not exceed Δ_a / ρ for any story. ρ shall be determined in accordance with Section 12.3.4.2.

Table 12.12-1. Allowable Story Drift, Δ_a .

Risk Category	Δ_a
I or II	$0.020h_{sx}^{a, b}$
III	$0.015h_{sx}^b$
IV	$0.010h_{sx}^b$

^aThe allowable story drift, Δ_a , is permitted to be increased to $0.025h_{sx}$ for structures that are four stories or less above the base as defined in Section 11.2 where drift sensitive structural elements and architectural nonstructural components, including interior walls, partitions, and ceilings, accommodate the drifts associated with the Design Earthquake Displacements.

^b h_{sx} is the story height below level x.

Notes: For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.

SECTION 12.13 FOUNDATION DESIGN

Revise Section 12.13.9 as follows:

12.13.9 Requirements for Foundations Subject to Seismically-Induced Soil Displacement or Strength Loss Where the geotechnical investigation report required in Section 11.8 identifies the potential for liquefaction, seismically-induced permanent ground displacement, or soil strength loss in the applicable earthquake motions as defined in Section 11.8.3, structures shall be designed to accommodate the effects in accordance with the requirements of Sections 12.13.9.1 through 12.13.9.3. Such structures shall also be designed to resist the seismic load effects of Section 12.4.1, presuming these effects do not occur.

EXCEPTION: Structures on shallow foundations need not be designed for the requirements of this section where the geotechnical investigation report indicates a negligible potential for the occurrence of permanent horizontal ground displacement, no bearing capacity loss, no lateral resistance loss, and differential settlements of site soils or improved site soils do not exceed one-fourth of the differential settlement threshold specified in Table 12.13-3.

Where the geotechnical investigation report indicates the potential for flow failure or seismically-induced long runout landslides, the provisions of Section 12.13.9 are not applicable and the condition shall be mitigated.

12.13.9.1 Foundation Design Foundations shall be designed to support gravity and design earthquake loads, as indicated in the basic load combinations of Section 12.4.1, using the reduced soil bearing capacity, as indicated in the geotechnical investigation report, considering the effects of

liquefaction, seismically-induced permanent ground displacement, and soil strength loss caused by the applicable earthquake motions as defined in Section 11.8.3. It shall be permitted to include the mitigating effects of any planned improvements for the site.

12.13.9.2 Shallow Foundations Building structures shall be permitted to be supported on shallow foundations provided that the foundations are designed and detailed in accordance with Section 12.13.9.2.1 and the conditions provided in items (a) and (b) of Section 12.13.9.2 are met.

(a) The geotechnical investigation report indicates that seismically-induced permanent horizontal ground displacement does not exceed the value in Table 12.13-2.

(b) The geotechnical investigation report indicates that the differential settlement over a defined length, L , does not exceed the differential settlement threshold specified in Table 12.13-3.

Table 12.13-2. Upper Limit on Permanent Horizontal Ground Displacement for Shallow Foundations.

Risk Category	I or II	III	IV
Limit, in. (mm)	18 (455)	12 (305)	4 (100)

EXCEPTION: It is permitted to exceed the differential settlement threshold specified in Table 12.13-3 for shallow foundations, provided the foundation and superstructure are designed to accommodate seismically-induced differential settlements without loss of the ability to support gravity loads in addition to the requirements of Section 12.13.9.2.1. For structures assigned to Risk Category II or III, residual strength of members and connections shall not be less than 67% of the undamaged nominal strength, considering the nonlinear behavior of the structure or, alternatively, demands on all members and connections shall not exceed the element's nominal strength when subjected to differential settlements. For structures assigned to Risk Category IV, demands on all members and connections shall not exceed the element's nominal strength when subjected to differential settlements.

Table 12.13-3. Differential Settlement Threshold.

Structure Type	Risk Category		
	I or II	III	IV
Single-story structures with concrete or masonry wall systems	$0.0075L$	$0.005L$	$0.002L$
Other single-story structures	$0.015L$	$0.010L$	$0.002L$
Multistory structures with concrete or masonry wall systems	$0.005L$	$0.003L$	$0.002L$
Other multistory structures	$0.010L$	$0.006L$	$0.002L$

12.13.9.2.1 Shallow Foundation Design Shallow foundations shall satisfy the design and detailing requirements of this section as required.

Foundation Ties

Individual footings shall be interconnected by ties in accordance with Section 12.13.8.2 and the additional requirements of this section. Reinforced concrete sections shall be detailed in accordance with Sections 18.13.4 of ACI 318. Where the geotechnical investigation report indicates that seismically-induced permanent horizontal ground displacement exceeding 3 in. (76.2 mm) will occur in the applicable earthquake motions as defined in Section 11.8.3, both of the following requirements shall be met.

1. Ties between individual footings on the same column or wall line shall, in lieu of the force requirements of Section 12.13.8.2, have a design strength in tension and compression at least equal to the design tie force, F_{tie} , as indicated in Equation (12.13-1). These effects shall be combined with the load effects from design earthquake lateral loads.

$$F_{tie} = 0.5\mu P_u \quad (12.13-1)$$

where μ is the coefficient of friction between the bottom of the footing and the soil, as indicated in the geotechnical report, or taken as 0.5 in the absence of other information; and P_u is the total of the supported gravity loads of all footings along the same column or wall line, determined in accordance with load combination 6 in Section 2.3.6.

2. Individual footings shall be integral with or connected to a reinforced concrete slab-on-ground, at least 5 in. (127.0 cm) thick and reinforced in each horizontal direction with a minimum reinforcing ratio of 0.0025. Alternately, individual footings shall be integral with or connected to a posttensioned concrete slab on ground designed according to PTI DC10.5 with a minimum effective compression after losses of 100 psi (690 kPa). For sites with expansive soils, movements from both expansive soils and seismically-induced permanent ground displacement need not be considered concurrently. For purposes of this section, concrete slab on ground need not satisfy Section 18.6.4 of ACI 318.

EXCEPTION: A system of diagonal reinforced concrete ties is permitted to be used if the system of ties provides equivalent lateral shear strength and stiffness to a slab on ground as defined above.

Mat Foundations

Mat foundations shall be designed to accommodate the expected vertical differential settlements indicated in the geotechnical investigation report, considering any increased loads induced by differential settlements of adjacent columns. The flexural demands caused by seismically-induced differential settlement need not be considered if the mat is detailed in accordance with the

requirements of Section 18.13.3.1 of ACI 318. Mat foundations shall have longitudinal reinforcement in both directions at the top and bottom.

12.13.9.3 Deep Foundations Deep foundations shall be designed to support vertical loads as indicated in the basic load combinations of Section 12.4.1, in combination with the moments and shears caused by lateral deformation of deep foundation elements in response to lateral inertial loads. Axial capacity of the deep foundation and lateral soil resistance shall be reduced to account for the effects of liquefaction and reduced soil strength. Deep foundations shall satisfy the design and detailing requirements of Sections 12.13.9.3.1 through 12.13.9.3.5.

12.13.9.3.1 Downdrag Design of piles shall incorporate the effects of downdrag caused by seismically-induced settlement. For geotechnical design, the downdrag load shall be determined as the downward skin friction on the pile within and above the settling zone(s). The net geotechnical ultimate capacity of the pile shall be the ultimate geotechnical capacity of the pile below the settling zone(s) reduced by the downdrag load. For structural design, the downdrag load induced by seismic settlement shall be treated and factored as a seismic load, though it need not be considered concurrently with axial loads resulting from inertial response of the structure, determined according to Section 12.4.1.

EXCEPTION: In lieu of the procedure defined in this section, rational analysis is permitted to be used to determine the downdrag load and the net geotechnical ultimate capacity of the pile if the seismically-induced foundation differential settlement is indicated in the geotechnical investigation report and one of the following requirements is met:

1. The seismically-induced foundation differential settlement does not exceed one-fourth of the differential settlement threshold in Table 12.13-3.
2. The seismically-induced foundation differential settlement does not exceed the differential settlement threshold in Table 12.13-3 and does not exceed $0.0075L$; foundation ties comply with the additional detailing requirements of Section 12.13.9.2.1; and mat foundations comply with the requirements of Section 12.13.9.2.1.
3. The seismically-induced foundation differential settlement complies with the design requirements of Section 12.13.9.2(b); foundation ties comply with the additional detailing requirements of Section 12.13.9.2.1; and mat foundations comply with the requirements of Section 12.13.9.2.1.

12.13.9.3.2 Lateral Resistance Passive pressure and friction mobilized against walls, pile caps, and grade beams, when reduced for the effects of liquefaction and seismically-induced reduction in soil strength, shall be permitted to resist lateral inertial loads in combination with piles. Resistance provided by the combination of piles, passive pressure, and friction shall be determined based on compatible lateral deformations.

12.13.9.3.3 Concrete Deep Foundation Detailing Concrete piles, including cast-in-place and precast piles, shall be detailed to comply with Section 18.13.5.5 of ACI 318 from the top of the pile to a

depth exceeding that of the deepest liquefiable soil by at least 7 times the member cross-sectional dimension.

12.13.9.3.4 Permanent Horizontal Ground Displacement Where the geotechnical investigation report indicates a potential for seismically-induced permanent horizontal ground displacement will occur in the event of the applicable earthquake motions as defined in Section 11.8.3, pile design shall be based on a detailed analysis incorporating the expected lateral deformation, the depths over which the deformation is expected to occur, and the nonlinear behavior of the piles. Where nonlinear behavior of piles occurs caused by seismically-induced permanent horizontal ground displacement, the pile deformations shall not result in loss of the pile's ability to carry gravity loads, nor shall the deteriorated pile's lateral strength be less than 67% of the undamaged nominal strength. In addition, the following requirements shall be satisfied:

1. Structural steel H-piles shall satisfy the width-thickness limits for highly ductile H-pile members in ANSI/AISC 341.
2. Unfilled structural steel pipe piles shall satisfy the width-thickness limits for highly ductile round hollow structural steel (HSS) elements in ANSI/AISC 341.
3. Concrete piles shall be detailed to comply with Section 18.13.5.5 of ACI 318 from the top of the pile to a depth exceeding that of the deepest layer of soil prone to seismically-induced permanent horizontal ground displacement by at least 7 times the pile diameter. Nominal shear strength shall exceed the maximum forces that can be generated because of pile deformations determined in the detailed analysis.

Add Section 12.13.10 as follows:

12.13.10 Requirements for Foundations and Structures Within Known Active Surface Fault Zones

Structures in Known Active Surface Fault Zones as determined in Section 11.8.1 shall be designed in accordance with the requirements of Sections 12.13.10.1 and 12.13.10.2 to accommodate the effects of the Maximum Considered Fault Displacement (MCFD), based on the MCFD horizontal and vertical component values and orientation determined in accordance with Section 11.8.1.1 and shown in Figure 11.8-X.

12.13.10.1 Threshold Values of Fault Displacement Below Which Design for Surface Fault Displacement Is Not Required

Where required, a structure shall be designed to accommodate the effects of surface fault displacement in accordance with Section 12.13.10.2 if any of the MCFD vertical and horizontal component values exceed the threshold values shown in Table 12.13.10-1. For Table 12.13.10-1, the structure's foundation shall be categorized as Foundation Type B when ANY of the following conditions are present:

1. Deep foundations.
2. Foundation and/or integral retaining walls retain more than 4 feet of ground measured from the bottom of the footing to the ground surface.
3. Foundation lacks ties, as required in Section 12.13.8.2.
4. Foundation has an overall plan dimension in any direction larger than 200 ft.
5. Foundation has an overall plan dimension in any direction larger than 100 ft. with a reentrant corner where both plan projections of the foundation beyond the reentrant corner are greater than 20% of the plan dimension of the foundation in the given direction.

If none of these conditions are present, the foundation is permitted to be categorized as Foundation Type A.

Table 12.13.10-1 Threshold Values of MCFD Surface Fault Displacement Components

Risk Category	Horizontal Displacement Component (in),		Vertical Displacement Component (in)
	Foundation Type A	Foundation Type B	
I-II	4	1	1
III	3	1	1
IV	2	0.5	0.5

12.13.10.2 Designing for Surface Fault Displacement Demand

If any of the MCFD component values exceed the applicable threshold values of Table 12.13.10-1, structures whose foundations are located within a Known Active Surface Fault Zone as determined in Section 11.8.1 shall not be permitted unless the design is based on an analysis and evaluation meeting the following requirements.

1. Force and deformation demands imposed on the structure and its foundation by the surface fault displacement shall be evaluated in comparison to the mechanisms by which the structure can resist the force demands and accommodate the deformation demands. The evaluation of demands shall include forces that can be imposed from (a) retained ground, such as from passive pressure and (b) friction between the foundation and the ground. The evaluation shall account for the expected location, orientation, and affected width or area of surface fault displacement, with the design accounting for the least favorable conditions within the expected range of each assumption.
2. The evaluation of structural capacities shall consider the expected failure modes by which fault displacement could damage or collapse the structure. The foundation and superstructure shall be designed to accommodate the MCFD without loss of the ability to support gravity loads, with

nonlinear behavior permitted. For structures assigned to Risk Category II or III, residual strength of members and connections shall not be less than 67% of the undamaged nominal strength, considering the nonlinear behavior of the structure or, alternatively, demands on all members and connections shall not exceed the element's nominal strength when subjected to the MCFD. For structures assigned to Risk Category IV, demands on all members and connections shall not exceed the element's nominal strength when subjected to the MCFD

3. In addition, foundations shall be designed to support gravity and design earthquake loads, as indicated in the basic load combinations of Section 12.4, considering the case without fault displacement and the case with concurrent fault displacement demands per Section 11.8.1.1. Where design evaluations are for load combinations that include design earthquake ground motion loadings, the MCFD provided in the geotechnical investigation report are permitted to be divided by 1.5.
4. There shall be independent design review consistent with the requirements of Section 16.5, including a review of the evaluation and design per Sections 11.8.1 and 12.13.10 related to Engineering Geology, Geotechnical Engineering, and Structural Engineering and performance assessment. Aspects to be reviewed shall include the fault displacements, location, extent, and orientations, forces and pressures that could be delivered by the ground, mechanisms of potential movement between the ground and the structure, such as sliding, and associated forces, computations of structural capacity, potential failure modes of the structure, and assessments of structural performance.

SECTION 12.14 SIMPLIFIED ALTERNATIVE STRUCTURAL DESIGN CRITERIA FOR SIMPLE BEARING WALL OR BUILDING FRAME SYSTEMS

Revise Section 12.14.1.1 as follows:

12.14.1.1 Simplified Design Procedure The procedures of this section are permitted to be used in lieu of other analytical procedures in Chapter 12 for the analysis and design of simple buildings with bearing wall or building frame systems, subject to all of the limitations listed in this section. Where these procedures are used, the Seismic Design Category shall be determined using section 11.6. The simplified design procedure is permitted to be used if the following limitations are met.

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CHAPTER 14, MATERIAL-SPECIFIC SEISMIC DESIGN AND DETAILING REQUIREMENTS (Modifications)

Revise Sections 14.1.2.2.1 and 14.1.2.2.2 as follows:

14.1.2.2.1 Seismic Design Categories B and C. Structural steel structures assigned to Seismic Design Category B or C shall be of any construction permitted by the applicable reference documents in Section 14.1.1. Where a response modification coefficient, R , is used in accordance with Table 12.2-1 for the design of structural steel structures assigned to Seismic Design Category B or C, the structures shall be designed and detailed in accordance with the requirements of AISC 341. Steel Damped Special Moment Frames (DSMF), as designated in Table 12.2-1 Line C.13, shall be designed and detailed in accordance with the requirements of Section 14.1.12.

EXCEPTION: The response modification coefficient, R , designated for “steel systems not specifically detailed for seismic resistance, excluding cantilever column systems” in Table 12.2-1 shall be permitted for systems designed and detailed in accordance with AISC 360 and need not be designed and detailed in accordance with AISC 341.

14.1.2.2.2 Seismic Design Categories D through F. Structural steel structures assigned to Seismic Design Category D, E, or F shall be designed and detailed in accordance with AISC 341, except as permitted in Table 15.4-1. Steel Damped Special Moment Frames (DSMF), as designated in Table 12.2-1 Line C.13, shall be designed and detailed in accordance with the requirements of Section 14.1.12.

Add Sections 14.1.2.2.3 and 14.1.2.2.4 as follows:

14.1.2.2.3 Glossary Definitions for Rods

The following definitions apply:

Hillside washer. A hemispherical ductile iron casting that allows a threaded rod to be oriented at a range of angles relative to the longitudinal axis of the supporting member.

Turnbuckle. A device consisting of a link that is threaded at both ends that is used to adjust the amount of tension in a threaded rod.

14.1.2.2.4 Materials for Rods

Materials allowed for bars in AISC 341 Table A3.1 shall also be allowed for rods. In addition, ASTM F1554 Gr. 55 shall also be allowed for rods.

Add Sections 14.1.10 and 14.1.11 as follows:

14.1.10 Rod-Braced Diaphragms Elements of the rod-braced diaphragm shall be structural or cold-formed steel. Structural steel elements and connections in the rod-braced diaphragms shall be designed in accordance with the requirements of AISC 360. If used, cold-formed steel members and connections shall be designed in accordance with AISI S100. Where required by this standard, special seismic detailing requirements shall be in accordance with Section 14.1.11 rod-braced diaphragms with special seismic detailing.

14.1.11 Rod-Braced Diaphragms with Special Seismic Detailing

14.1.11.1 SCOPE

Rod-braced diaphragms with special seismic detailing shall be designed in conformance with this section.

14.1.11.2 BASIS OF DESIGN

A special rod-braced diaphragm designed in accordance with these provisions is expected to provide inelastic deformation capacity through yielding the unthreaded portion of the rods. An adequate load path to deliver the lateral loads to the rod-bracing is required.

14.1.11.3 ANALYSIS

There are no additional analysis requirements.

14.1.11.4 SYSTEM REQUIREMENTS

All of the following requirements shall apply:

- (a) The design tensile strength, ϕP_n (LRFD), and the allowable tensile strength, P_n/Ω (ASD), of the rod shall be based on the limit state of tensile yielding in the gross section of the unthreaded body of the rod as follows:

$$P_n = F_y A_g \quad (B5-1)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

Where:

F_y = Specified minimum yield stress of the rod

A_g = Gross area of the unthreaded rod

- (b) The material used for the rods shall meet one of the ASTM standards in AISC 341 Section A3 for round rods and have a nominal diameter not less than 0.375 in. (10mm) and not greater than 1.5 in. (38mm). Additionally, the rod material shall satisfy the following:

$$\frac{F_u}{F_y} \geq 1.1 \frac{A_g}{A_n} \quad (\text{B5-2})$$

Where:

F_u = Specified minimum tensile strength of the rod

A_n = Net tensile area of the rod

- (c) At locations where the rod bracing is attached to a beam, the beam shall be braced against lateral displacement at both flanges within 6 in. of the rod bracing connection, measured along the longitudinal axis of the member. Bracing shall be designed in accordance with AISC 360 Appendix 6.
- (d) The beam flange nearest to the rod connection shall satisfy the compact width-to-thickness limit in AISC 360 for a distance not less than one beam depth beyond the diagonal rod bracing connection in both directions. If the rod connection is located within the middle third of the beam depth, then both flanges shall satisfy the compact width-to-thickness limit in AISC 360 within this distance.
- (e) The beams, members perpendicular to the beams and their connections, turnbuckles, and connection of the rod brace to the beam shall be designed for the capacity-limited seismic load effect based on expected rod brace strengths. Beams and members perpendicular to the beams shall be designed for loads acting perpendicular to the roof plane resulting from rods being oriented at an angle relative to the roof plane, with a minimum out of plane load taken as the required bracing force from AISC 360 Appendix 6. The expected rod brace strength in tension shall be determined in accordance with the following:

$$P_{be} = R_y F_y A_g / \alpha_s \quad (\text{B5-4})$$

Where

R_y = Ratio of the expected yield stress of the rod to the specified minimum yield stress, F_y , of the rod

α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD.

Exception: These members and connections need not be designed for forces larger than those associated with 2.5 times the diaphragm inertial forces from Section 12.10.2.2 plus transfer forces per 12.10.2.3 as applicable.

- (f) For structures assigned to Seismic Design Categories D through F, diaphragm design forces in a given direction of loading shall be multiplied by 1.3 if the seismic force in that direction is resisted by a single opposing pair of rod braces at any section along the diaphragm span.
- (g) At the connection of the rod to the supporting member, either a hillside washer or clevis connection must be used. Where hillside washers are used, they shall be ASTM A536 material and have been shown by the manufacturer to develop the tensile strength of the rod. Hillside washers shall be mechanically attached to the supporting member web to prevent unseating during cyclic loading while allowing the bearing tab to engage the edge of the web hole. The washer-to-support connection shall be designed to prevent washer pull-through or local beam web failure at loads associated with the expected rod brace strength. [User note: A hillside washer is a hemispherical ductile iron casting with its flat surface placed against the beam web, a bearing tab that fits into the beam web slotted hole to restrain sliding along the length of the beam caused by rod tension, and a rounded edge that allows the rod and nut to accommodate a wide range of rod angles relative to the beam longitudinal axis. More guidance is provided in the commentary]

Add Section 14.1.12 as follows:

14.1.12 Seismic Requirements for Steel Damped Special Moment Frame Structures

14.1.12.1 GENERAL

14.1.12.1.1 Scope

Steel Damped Special Moment Frames (DSMFs) shall be designed in accordance with this section and, where explicitly stated, the specified requirements of Chapter 18. DSMFs consist of (i) Steel Special Moment Frames (SMFs) and (ii) supplemental damper frames (DFs) utilizing fluid viscous dampers.

14.1.12.1.2 System Description

This system consists of structural Steel Special Moment Frames conforming to the requirements of AISC 341 and Damper Frames designated as part of the Seismic Force Resisting System conforming with the requirements herein and AISC 341 Chapters A – General Requirements, B – General Design Requirements, D – General Member and Connection Design Requirements, I – Fabrication and Erection, and J – Quality Control and Quality Assurance. The Damper Frame contains fluid viscous dampers subject to the additional limitations of Section 14.1.12.1.3.

14.1.12.1.3 System Limitations The following system limitations shall apply:

1. Fluid viscous dampers properties shall meet the requirements of Section 14.1.12.7.3.
2. Floor diaphragms shall not qualify as flexible as defined in Section 12.3.1.1 or 12.3.1.3.
3. Buildings shall not have a Torsional Irregularity Ratio (TIR), as defined in Section 12.3.2.1.1, greater than 1.4
4. In each principal direction utilizing the DSMF system, the damping system shall have at least two damping devices in each story above the base, with at least one damping device on either side of the center of mass.
5. Fluid viscous damping devices shall not be located within the bays of the SMF.

14.1.12.1.4 Notation

The symbols listed below are to be used in addition to or as replacements for those in this Standard, AISC 360 and AISC 341.

A_1 = amplification factor for first story yielding

A_v = velocity amplification factor for higher modes and yielding

BF = base shear correction factor—removes the effect of the base shear scaling from the design earthquake damper displacement

$C_{ji(L)}$ = linear damper constant computed for the j^{th} damper in the i^{th} story

$C_{ji(NL)}$ = required nonlinear damper constant computed for the j^{th} damper in the i^{th} story

$C_{ji(NL)_{\text{spec}}}$ = specified nonlinear damping constant for the j^{th} damper in the i^{th} story

CD_i = center of damping location in the i^{th} story

$CF1_{ji}$ = column axial load reduction factor, option 1

$CF2_{ji}$ = column axial load reduction factor, option 2

d_{ji} = design earthquake level damper displacement measured from the maximum displacement stage analysis

$e_{FVD,i}$ = damping eccentricity, defined as the distance between center of mass and center of damping (CD_i) in the direction of interest

F_{ji} = Overstrength FVD force in the j^{th} damper in i^{th} story

$F_{px,FVD}$ = cumulative damper-induced diaphragm force at the i^{th} story

f_{ji} = design force in the j^{th} damper in the i^{th} story

$f_{MCE,ji}$ = MCE_R damper force in the j^{th} damper in the i^{th} story

$h_{s,i}$ = story height of story i

K_E = extender brace stiffness

$K_{EM,ji}$ = minimum extender brace stiffness for the j^{th} damper extender brace in the i^{th} story

K_{SM} = minimum allowable DF bay stiffness

k_i = linear stiffness of the i^{th} story in the principal direction of interest

L = plan dimension perpendicular to the direction of interest; or pin-to-pin length of the damper and brace assembly

$M_{ta,DSMF}$ = modified accidental torsion moment, with added torsion due to damping eccentricity

n = number of dampers in a DF bay

n_{ai} = number of consecutive columns taking damper forces above story i of interest

$n_{local,ai}$ = number of consecutive columns taking damper forces above story i of interest for which $CF2_{ji}$ is computed

n_i = number of dampers in story i

n_s = number of above grade DSMF stories

$P_{EFVD,ji}$ = reduced damper-induced column axial load for the j^{th} column in the i^{th} story

$P_{EFVD1,ji}$ = reduced damper-induced column axial load for the j^{th} column in the i^{th} story, option 1

$P_{EFVD2,ji}$ = reduced damper-induced column axial load for the j^{th} column in the i^{th} story, option 2

P_{ji} = damper-induced column axial load at story i and column line j , considering damper overstrength from maximum forces at all stories above

$P_{local,ji}$ = local damper-induced column axial load considering damper overstrength acting at story i and column line j

Q_{MD} = maximum displacement stage load effect

Q_{FVD} = maximum velocity stage load effect, i.e. from design earthquake FVD forces

R_v = factor for determining the upper bound value of the damping constant

$r_{FVD,i}$ = radius of gyration of damping at story i in the direction of interest

$s_{req,ji}$ = minimum required damper stroke capacity

$V_{FVD,i}$ = damping system story shear at story i , based on the design earthquake FVD load effect from maximum velocity stage analysis

$V_{MD,i}$ = earthquake story shear in story i for the maximum displacement stage analysis for strength design

V_i = seismic shear force acting between levels i and $(i - 1)$ based on the ELF

v_{ji} = damper velocity of the j^{th} damper in the i^{th} story for computing the target nonlinear damping constant

v_{ji}^* = design earthquake damper velocity for computing the design earthquake FVD force

x_{ji} = horizontal location of the j^{th} damper in the i^{th} story measured perpendicular to the direction of interest

α = velocity exponent, 0.4

β_v = target viscous damping ratio, 0.25

Δ_i = design story drift between levels i and $(i - 1)$ occurring simultaneously with V_i , computed from a static ELF analysis

θ_{ji} = angle of inclination of the j^{th} damper in the i^{th} story measured from horizontal

$\theta_{\max}$ = maximum allowable stability coefficient

λ = Parameter computed with equation 4-14 of MCEER report 00-0010, used for determining the nonlinear damping constant based on the linear damping constant. For the DSMF system, it is 3.582 corresponding to a velocity exponent, $\alpha = 0.4$.

$\lambda_{(\max)}$ = factor for maximum design properties of the specified damper

$\lambda_{(\min)}$ = factor for minimum design properties of the specified damper

τ_{ji} = ratio of the specified nonlinear damping constant to the required nonlinear damping constant for the j^{th} damper in the i^{th} story

φ_{ji} = angle of DF containing the j^{th} damper on i^{th} story measured from the principal direction (zero degrees when the DF aligns with the principal direction)

ψ_{ji} = angle of the j^{th} damper on i^{th} story measured from the principal direction of motion under consideration. For DFs aligned with the principal direction, ψ_{ji} is equal to the damper angle of inclination, θ_{ji}

Ω_d = overstrength factor on viscous damper stroke required

Ω_F = overstrength factor for damper forces

Ω_v = overstrength factor on design velocity for determining overstrength damper force demands

ω = angular frequency in radians per second

14.1.12.2 BASIS OF DESIGN

The special moment frames (SMFs) shall act as the primary lateral force-resisting system for both wind and seismic excitation, while the damper frame (DF) shall be included to dissipate seismic energy and reduce the overall response of the structure.

The SMF shall be designed in accordance with AISC 341.

The DF shall be designed separately from the SMF, although the two systems are permitted to have common elements. The damping devices are permitted to be located colinear or on different structural framing lines with respect to the SMF.

Elements common to the SMF and DF shall be designed for combined demands from both systems.

For the purposes of this section, the DF shall include the fluid viscous dampers, extender braces, connections, beams, columns, diaphragm, and foundations that transfer the damper forces laterally and vertically throughout the structure. Dampers shall be arranged in diagonal, chevron, or V-type configurations where the dampers are in-line with the inclined extender braces between work points within the DF. Dampers are also permitted to be arranged horizontally in a modified chevron or V-type configuration with extender braces acting as driver braces. Eccentricities in the damper line of action at the joints in the DF are permitted if the resulting member and connection forces are computed and the members and connections resist those forces as specified in Section 14.1.12.7 and if the eccentricities do not alter either the viscous energy dissipation in the DF or the expected inelastic deformation of the SMF.

The horizontal seismic load effect, E_h , for the DSMF system shall be the maximum displacement stage load effect (Q_{MD} , Section 14.1.12.3) and the maximum velocity stage load effect (Q_{FVD} , Section 14.1.12.5) combined using the equations of Section 14.1.12.9.

14.1.12.3 MAXIMUM DISPLACEMENT STAGE ANALYSIS

The maximum displacement stage load effect, Q_{MD} , shall be determined in accordance with Chapter 12 as modified in this section.

14.1.12.3.1 SMF Analysis

The required strength and stiffness of the SMF for the design earthquake (DE) shall be determined using Modal Response Spectrum Analysis (MRSA), using the 5% damped design spectrum without consideration of the supplemental damping provided by the DF, in accordance with Section 12.9.1, with the following modifications:

1. Section 12.8.7 – P-Delta Effects:

Equation 12.8-19 shall be modified as follows:

$$\theta_{\max} = 0.25 \quad (\text{Eqn. 14.1.12-1})$$

2. Section 12.8.4.2 – Accidental Torsion:

When the damping eccentricity, $e_{FVD,i}$, is greater than $0.02L$, the accidental torsion moment calculation shall be modified to include damping eccentricity:

$$M_{ta,DSMF} = 0.05V_{MD,i}L + 0.7e_{FVD,i}V_{FVD,i} \left(1 - \frac{r_{FVD,i}}{L}\right)^2 \quad (\text{Eqn. 14.1.12-2})$$

Where,

$V_{MD,i}$ = earthquake story shear in story i for the maximum displacement stage analysis of this section for strength design

L = plan dimension perpendicular to the direction of interest.

$e_{FVD,i}$ = damping eccentricity, defined as the distance between center of mass and center of linear damping (CD_i) in the direction of interest

$r_{FVD,i}$ = radius of gyration of linear damping at story i in the direction of interest

$V_{FVD,i}$ = damping system story shear at story i , based on the design earthquake fluid viscous damper (FVD) load effect (f_{ji} per Section 14.1.12.5.2) from maximum velocity stage analysis in Section 14.1.12.5.

Where all lines of damping in a given direction have identical configurations and dampers, the center of damping (CD_i) shall be the average of their locations. Otherwise, CD_i shall be computed in accordance with the following equation:

$$CD_i = \frac{\sum_{j=1}^{n_i} x_{ji} \tau_{ji} C_{ji(L)} \cos^2 \psi_{ji}}{\sum_{j=1}^{n_i} \tau_{ji} C_{ji(L)} \cos^2 \psi_{ji}} \quad (\text{Eqn. 14.1.12-3})$$

Where,

n_i = number of dampers in story i

x_{ji} = horizontal location of the j^{th} damper on the i^{th} story measured perpendicular to the direction of interest

τ_{ji} = the ratio of the specified nonlinear damping constant to the required nonlinear damping constant for j^{th} damper on i^{th} story, defined in Section 14.1.12.4.4

$C_{ji(L)}$ = linear damper constant computed for j^{th} damper on i^{th} story in Section 14.1.12.4.1.

ψ_{ji} = angle of the j^{th} damper on i^{th} story measured from the principal direction of motion under consideration. For DFs aligned with the principal direction, ψ_{ji} is equal to the damper angle of inclination measured from horizontal (θ_{ji}). For DFs not aligned with the principal direction:

$$\psi_{ji} = \cos^{-1}(\cos \theta_{ji} \cos \varphi_{ji})$$

Where,

θ_{ji} = angle of inclination of the j^{th} damper on i^{th} story measured from horizontal

φ_{ji} = angle of DF containing the j^{th} damper on i^{th} story measured from the principal direction (zero degrees when the DF aligns with the principal direction)

The radius of gyration of damping at the i^{th} story, $r_{FVD,i}$, shall be computed using only the dampers' contributions to damping in the direction of interest. For two identical lines of damping that are parallel to the direction of interest, this quantity shall be half the perpendicular distance between them. Otherwise, radius of gyration of damping at the i^{th} story, $r_{FVD,i}$, shall be determined by the following equation:

$$r_{FVD,i} = \sqrt{\frac{\sum_{j=1}^{n_i} [(x_{ji} - CD_i)^2 \tau_{ji} C_{ji(L)} \cos^2 \psi_{ji}]}{\sum_{j=1}^{n_i} [\tau_{ji} C_{ji(L)} \cos^2 \psi_{ji}]}} \quad (\text{Eqn. 14.1.12-4})$$

3. Section 12.9.1.4.1 – Scaling of Forces:

The combined response for the modal base shear (V_t) shall be scaled to 75% of the ELF base shear (V) computed in Section 12.8.

4. Section 12.9.1.4.2 – Scaling of Drifts:

The seismic response coefficient, C_s , from Equation 12.8-7 shall be replaced with Equation 14.1.12-5 for scaling of drifts. The 25% reduction that is used for strength checks shall not be permitted to be taken. This scaling shall apply for any design in which the combined modal base shear V_t is less than C_s from the following equation:

$$C_s = \frac{0.35S_{D1}}{\frac{R}{I_e}} \leq \frac{0.5S_1}{\frac{R}{I_e}} \quad (\text{Eqn. 14.1.12-5})$$

14.1.12.3.2 Diaphragms, chords and collectors

Diaphragm inertial forces for the maximum displacement stage analysis shall be determined in accordance with Section 12.10, with the following modification:

1. Limiting equations for diaphragm inertial forces in Sections 12.10.1.1 and 12.10.2.3 (Equations 12.10-2, 12.10-3, 12.10-5) shall be multiplied by 0.75.

Damper-induced diaphragm forces, $F_{px,FVD}$, shall be determined in accordance Equation 14.1.12-6 and shall be combined with diaphragm inertial forces using the load combinations provided in Section 14.1.12.9.

$$F_{px,FVD} = (0.5 - 0.01 * n_s) * \left[\sum_{j=1}^{n_i} (F_{ji}) + \sum_{j=1}^{n_{i-1}} (F_{j(i-1)}) \right] \quad (\text{Eqn. 14.1.12-6})$$

Where,

$F_{px,FVD}$ = cumulative damper-induced diaphragm force at the i^{th} story

n_s = number of above grade DSMF stories

n_i = total number of dampers in the direction of interest in the i^{th} story, above the diaphragm of interest

n_{i-1} = total number of dampers in the direction of interest in the $(i-1)^{\text{th}}$ story, below the diaphragm of interest

F_{ji} = overstrength force in the j^{th} damper in the i^{th} story, above the diaphragm of interest

$F_{j(i-1)}$ = overstrength force in the j^{th} damper in the $(i-1)^{\text{th}}$ story, below the diaphragm of interest

For diaphragms with dampers on one adjacent story only, including the top and bottom floors, the summation of overstrength damper forces, F_{ji} , from the adjacent story shall be combined with diaphragm inertial forces using the load combinations provided in Section 14.1.12.9.

Exception: If the SMF and DF are in line both above and below the diaphragm of interest, the damper-induced forces need not be considered for diaphragm shear and in-plane moment demands. In this case, overstrength damper forces shall be transferred directly through collectors to the SMF.

14.1.12.4 VISCOUS DAMPER CONSTANT FOR TARGET VISCOUS DAMPING

The DSMF system shall be tuned to provide 25% viscous damping at the DE intensity. To achieve this level of damping, the following procedures shall be followed to determine the damping constants, C , for each device at each level.

14.1.12.4.1 Target Linear Damper Constant

The following equation shall be used to determine the target linear damper constant for j^{th} damper in i^{th} story, $C_{ji(L)}$

$$C_{ji(L)} = \beta_v \frac{k_i}{\sum_{j=1}^{n_i} \cos^2 \varphi_{ji}} \frac{T_1}{\pi} \frac{1}{\cos^2 \theta_{ji}} \quad (\text{Eqn. 14.1.12-7})$$

Where,

$$\beta_v = 0.25$$

θ_{ji} = angle of inclination of the j^{th} damper on i^{th} story measured from horizontal

φ_{ji} = angle of DF containing the j^{th} damper on i^{th} story measured from the principal direction (zero degrees when the DF aligns with the principal direction)

T_1 = fundamental translational elastic period in the principal direction of interest computed from the structural model. Alternatively, T_1 is permitted to be computed using Equation 18.7-22.

k_i = i^{th} story linear stiffness in principal direction of interest, determined as:

$$k_i = (C_d V_i) / \Delta_i$$

Δ_i = inelastic design story drift between levels i and $(i - 1)$ occurring simultaneously with V_i , computed from a static ELF analysis (i.e. including C_d amplification)

V_i = seismic shear force acting between levels i and $(i - 1)$ based on ELF

n_i = number of dampers in story i

Exception: Alternative values of $C_{ji(L)}$ are permitted to be determined by satisfying Equation 14.1.12-8 for the dampers in story i that are used for damping in the principal direction of interest. If a damper is not aligned with a principal direction of interest, and its contribution to system level

damping is relied upon in both principal directions, $C_{ji(L)}$ shall be calculated for each principal direction per Equation 14.1.12-8 and the maximum value shall be used.

$$\sum_{j=1}^{n_i} C_{ji(L)} \cos^2 \psi_{ji} = \beta_v k_i \frac{T_1}{\pi} \quad (\text{Eqn. 14.1.12-8})$$

Where,

ψ_{ji} = angle of the j^{th} damper in i^{th} story measured from the principal direction of motion under consideration, determined as:

$$\psi_{ji} = \cos^{-1}(\cos \theta_{ji} \cos \phi_{ji})$$

14.1.12.4.2 Damper Velocity for Target Nonlinear Damping Constant

The individual FVD velocities, v_{ji} , shall be obtained from the following:

$$v_{ji} = \omega d_{ji} BF \quad (\text{Eqn. 14.1.12-9})$$

Where,

$$\omega = \frac{2\pi}{T_1}$$

d_{ji} = displacement of j^{th} damper in i^{th} story corresponding to the inelastic MRSA results

BF = base shear correction factor determined as:

$$BF = \min\left(1.0, \frac{V_t}{C_s W}\right)$$

Where,

V_t = modal base shear from Section 12.9.1.4.2

C_s = minimum base shear for drift checks in accordance with Section 14.1.12.3.1

14.1.12.4.3 Target Nonlinear Damping Constant

The following equation shall be used to determine the target nonlinear damper constant for j^{th} damper in i^{th} story, $C_{ji(NL)}$

$$C_{ji(NL)} = c_{ji(L)} \frac{\pi}{\lambda} (v_{ji})^{(1-\alpha)} \quad (\text{Eqn. 14.1.12-10})$$

Where,

$$\lambda = 3.582$$

α = velocity exponent, 0.4

14.1.12.4.4 Specified Nonlinear Damping Constant

When Eqn. 14.1.12-7 is used to determine the linear damping constant, the nonlinear damping constant $C_{ji(NL)spec}$ for each damper specified for installation shall satisfy the provisions of this section.

The ratio of the specified nonlinear damping constant to the target nonlinear damping constant (τ_{ji}) shall be greater than 0.9 and less than 1.3:

$$0.9 < \tau_{ji} < 1.3 \quad (\text{Eqn. 14.1.12-11})$$

Where,

$$\tau_{ji} = \frac{C_{ji(NL)spec}}{C_{ji(NL)}} \quad (\text{Eqn. 14.1.12-12})$$

When Eqn. 14.1.12-8 is used to determine linear damping constants such that the cumulative damping provided on a given floor in each principal direction is monitored, the following criterion shall be used:

$$0.9\beta_v k_i \frac{T_1}{\pi} < \sum_{j=1}^{n_i} C_{ji(L)} \cos^2 \psi_{ji} < 1.3\beta_v k_i \frac{T_1}{\pi} \quad (\text{Eqn. 14.1.12-13})$$

14.1.12.5 MAXIMUM VELOCITY STAGE ANALYSIS

Additional analyses shall be required to determine the FVD load effect, Q_{FVD} , based on peak FVD velocities created in the system by the DE ground motion. FVD load effects in elements of the DF shall be determined from an analysis in which all FVDs simultaneously resist forces, as determined in accordance with this section.

14.1.12.5.1 DE Damper Velocity

The pseudo-velocity calculated in Eqn. 14.1.12-9 shall be modified as follows to account for higher mode effects and nonlinear actions:

$$v_{ji}^* = A_v A_1 \omega d_{ji} \quad (\text{Eqn. 14.1.12-14})$$

Where,

A_v = velocity amplification factor for higher modes and yielding, determined as:

$$A_v = 1.1 + 0.1n_s \quad (\text{Eqn. 14.1.12-15})$$

With n_s = number of above grade DSMF stories.

A_1 = amplification factor for first story yielding, determined as:

$$A_1 = 0.85 \left(\frac{\frac{\Delta_2}{h_{s,2}}}{\frac{\Delta_1}{h_{s,1}}} \right) \geq 1.0 \quad (\text{at bottom DSMF story})(\text{Eqn. 14.1.12-16a})$$

$$A_1 = 1.0 \quad (\text{for all other stories}) \quad (\text{Eqn. 14.1.12-16b})$$

With,

Δ_i = inelastic story drift in story i from MRSA results

$h_{s,i}$ = story height of story i

14.1.12.5.2 FVD Forces at the Design Earthquake

Damper forces associated with the DE intensity shall be determined in accordance with the following:

$$f_{ji} = C_{ji(NL)spec} (v_{ji}^*)^\alpha \quad (\text{Eqn. 14.1.12-17})$$

Where,

$C_{ji(NL)spec}$ = specified nonlinear damping constant from Section 14.1.12.4.4 for the j^{th} damper in the i^{th} story

14.1.12.5.3 FVD Forces at MCE_R

The FVD forces at MCE_R shall be determined in accordance with the following:

$$f_{MCE,ji} = C_{ji(NL)spec} (1.5v_{ji}^*)^\alpha = 1.18f_{ji} \quad (\text{Eqn. 14.1.12-18})$$

14.1.12.5.4 Overstrength FVD Forces

Overstrength damper forces shall be determined in accordance with the following:

$$F_{ji} = R_v C_{ji(NL)spec} (\Omega_v v_{ji}^*)^\alpha = \Omega_v f_{ji} \quad (\text{Eqn. 14.1.12-19})$$

Where,

Ω_v = overstrength factor on design velocity for maximum considered demands

$$\Omega_v = 2.5$$

R_v = expected properties for damping (upper bound damping constant)

$$R_v = 1.15$$

If the final design utilizes dampers with properties exceeding normal testing limits, as permitted by Section 14.1.12.11.5, R_v shall be adjusted upwards in proportion to the accepted damping constant.

Ω_F = overstrength factor for damper forces, determined as:

$$\Omega_F = R_v (\Omega_v)^\alpha \quad (\text{Eqn. 14.1.12-20})$$

14.1.12.6 STEEL SPECIAL MOMENT FRAME (SMF) DESIGN REQUIREMENTS

14.1.12.6.1 Beams, Columns, and Connections

Beams, columns, and connections of the SMF shall meet the following design requirements:

1. The seismic load effect, E , shall be determined in accordance with Section 12.4 using the alternative horizontal load effect E_h as determined in Section 14.1.12.9.
2. SMF connections shall comply with provisions for Steel Special Moment Frames as specified in AISC 341.

14.1.12.7 DAMPER FRAME (DF) DESIGN REQUIREMENT

14.1.12.7.1 Beams and Columns

Beams and columns of the DF shall meet the following design requirements:

1. The seismic load effect, E , shall be determined in accordance with Section 12.4 using the alternative horizontal load effect E_h as determined in Section 14.1.12.9.
2. Beams shall satisfy the requirements of AISC 341, Section D1.1 for moderately ductile members.
3. Columns shall satisfy the requirements of AISC 341, Section D1.1 for highly ductile members.
4. For determining FVD load effects on the DF, beam-to-column connections are permitted to be treated as pinned for analysis.
5. Demand critical welds shall be designed to satisfy AISC 341, Section F4.6a.
6. The beam-to-column connections shall be designed to satisfy AISC 341, Section F4.6b, except the minimum rotation for option (a) shall be 0.050 radians.
7. Column splices shall be designed to satisfy AISC 341, Section F4.6d.

14.1.12.7.2 FVD Design Requirements

All FVDs shall be subject to prototype and production testing, as defined in Section 18.6. The test results and design report shall be approved by the registered design professional in accordance with the project specifications in Section 14.1.12.11. FVDs shall be specified in accordance with the following criteria:

1. Velocity Exponent, α : The velocity exponent, α , shall be 0.4.
2. Damper Force Capacity Requirement: FVDs shall be designed to resist the force at MCE_R developed by the specified damper, $f_{MCE,ji}$ (Section 14.1.12.5.3), and shall have a minimum factor of safety of 1.6 considering all applicable failure modes. This factor of safety is permitted to be demonstrated and approved through analytical means.
3. Minimum Required Damper Stroke, $s_{req,ji}$

The minimum required damper stroke, $s_{req,ji}$, shall be computed in accordance with the following:

$$s_{req,ji} = A_1 I_e \Omega_d d_{ji} \quad (\text{Eqn. 14.1.12-21})$$

Where,

Ω_d = overstrength factor on viscous damper stroke required, dependent upon number of stories, n_s , and seismic design category (SDC):

For building structures in SDC D and lower:

For $n_s \leq 4$	$\Omega_d = 3.5$
For $5 < n_s < 12$	$\Omega_d = 4.0 - 0.125n_s$
For $n_s \geq 12$	$\Omega_d = 2.5$

For building structures in SDC E and higher:

For $n_s \leq 8$	$\Omega_d = 3.5$
For $8 < n_s < 14$	$\Omega_d = 4.0 - 0.125n_s$
For $n_s \geq 14$	$\Omega_d = 2.75$

The minimum required damper stroke, $s_{req,ji}$, shall not be less than the stroke capacity required to accommodate a 3% interstory drift ratio.

14.1.12.7.3 Extender Brace Design Requirements

The required strength of extender braces and connecting elements between the extender brace and FVD shall be based on overstrength viscous damper demands as determined in accordance with Section 14.1.12.5.4. Nominal strength (P_n), strength reduction factor (ϕ), and detailing of the brace shall be determined in accordance with AISC 360 for axial members with pinned ends, and the cross-section shall qualify as non-slender for axial compression.

The stiffness of the extender brace for damper configuration with concentric connection shall be computed in accordance with the following:

$$K_E = \frac{A_g E}{L} \quad (\text{Eqn. 14.1.12-22})$$

Where,

E = modulus of elasticity for extender material

A_g = gross area of extender section

L = pin to pin length

The extender brace stiffness shall not be less than the following:

$$K_{EM,ji} = \frac{5f_{ji}}{d_{jiBF}} \quad (\text{Eqn. 14.1.12-23})$$

Where,

f_{ji} = DE damper force in accordance with Section 14.1.12.5.2

d_{jiBF} = DE damper displacement in accordance with Section 14.1.12.4.2, where,

$d_{jiBF} > 0.015h_i \cos\theta_{ji}$ shall be used as the minimum for extender brace stiffness calculations,

and h_i is the vertical distance between work points of the brace/damper assembly.

14.1.12.7.4 Stiffness for DF with Eccentric Connections

For the modified chevron or modified V with horizontal damper DF configuration, or for any DF configuration with connection eccentricities greater than half the beam depth, the lateral story stiffness of the combined brace, beams, and columns of the DF bay shall be computed assuming the columns are pinned at the top and bottom of the story and including the eccentricities of the connections. The minimum allowable DF bay stiffness shall be K_{SM} determined in accordance with the following:

$$K_{SM} = \frac{5 \sum_{j=1}^n f_j \cos \theta_j}{\sum_{j=1}^n d_j B F \cos \theta_j} \quad (\text{Eqn. 14.1.12-24})$$

where n is the number of dampers in the DF bay in the story being designed.

14.1.12.7.5 Connection Design Requirements

Required strength of connections for damping devices and extender braces to beams and columns shall be based on overstrength damper load effects as determined in Section 14.1.12.5.4. Design strengths shall be established in accordance with AISC 360.

Connections of dampers and braces shall be detailed to accommodate in-plane and out-of-plane drift of the SFRS.

14.1.12.8 OPTIONAL REDUCTION OF DAMPER-INDUCED COLUMN AXIAL LOADS FOR CONTINUOUSLY DAMPED BAYS

For column elements receiving damper-induced axial loads, the axial load corresponding to the Q_{FVD} loading is permitted to be reduced based on the number of continuously damped stories directly above the column of interest (n_{ai}). The resulting damper-induced column axial load for the j^{th} column in the i^{th} story ($P_{EFVD,ji}$) shall be given by the following relationships:

$$P_{EFVD,ji} = \max(P_{EFVD1,ji}, P_{EFVD2,ji}) \quad (\text{Eqn. 14.1.12-25})$$

$$P_{EFVD1,ji} = CF1_{ji} \cdot P_{ji} \quad (\text{Eqn. 14.1.12-26})$$

$$CF1_{ji} = 1/[1 + 0.06(n_{ai} - 3)] \quad 0.5 \leq CF1_{ji} \leq 1.0 \quad (\text{Eqn. 14.1.12-27})$$

Where,

P_{ji} = damper-induced column axial load considering damper overstrength at story i and column line j from maximum forces at all stories above

n_{ai} = number of consecutive columns taking damper induced axial forces of the same sign (i.e. all compression or all tension) above story i of interest, determined as:

$$P_{EFVD2,ji} = \sum_i^{n_s} CF2_{ji} P_{local,ji} \quad (\text{Eqn. 14.1.12-28})$$

$P_{local,ji}$ = local damper-induced column axial load considering damper overstrength acting at story i and column line j , determined as:

$$CF2_{ji} = 1 - [(n_{ai,local} - 2)0.15] \quad 0.25 \leq CF2_{ji} \leq 1.0 \quad (\text{Eqn. 14.1.12-29})$$

$n_{local,ai}$ = number of consecutive columns taking damper forces above story i for which $CF2_{ji}$ is computed

Exception: For the column under consideration, $P_{EFVD2,ji}$ is permitted to be taken as equal to $P_{EFVD1,ji}$, if there is no story above in which the story stiffness is less than 85% of the stiffness of the story below. The 85% limit shall not apply for the story supporting the roof.

14.1.12.9 HORIZONTAL SEISMIC LOAD EFFECT, E_h

For DF beams and columns, SMF elements, transfer elements, and all elements in the SFRS load path, including diaphragms, collectors, and foundations, the horizontal seismic load effect (E_h) shall be the maximum load effect resulting from Q_{MD} and Q_{FVD} as follows:

For portions of foundations that do not require overstrength, the horizontal seismic load effect, E_h , shall be determined in accordance with:

$$E_h = \max(Q_{MD}, Q_{FVD}, Q_{MD} \pm 0.7Q_{FVD}, 0.7Q_{MD} \pm Q_{FVD}) \quad (\text{Eqn. 14.1.12-30})$$

For other SFRS elements that do not require overstrength, the horizontal seismic load effect, E_h , shall be determined in accordance with:

$$E_h = \max(Q_{MD}, \Omega_F Q_{FVD}, Q_{MD} \pm 0.7\Omega_F Q_{FVD}, 0.7Q_{MD} \pm \Omega_F Q_{FVD}) \quad (\text{Eqn. 14.1.12-31})$$

For SFRS elements where overstrength is required, including collectors, the horizontal seismic load effect, E_h , shall be determined in accordance with:

$$E_{mh} = \max(\Omega_0 Q_{MD}, \Omega_F Q_{FVD}, \Omega_0 Q_{MD} \pm 0.7\Omega_F Q_{FVD}, 0.7\Omega_0 Q_{MD} \pm \Omega_F Q_{FVD}) \quad (\text{Eqn. 14.1.12-32})$$

14.1.12.10 ORTHOGONAL LOAD COMBINATIONS

Where the orthogonal combination of earthquake load effects requires simultaneous application of 30% of the orthogonal seismic load effect, the portion of the orthogonal load effect due to damper forces (Q_{FVD}) shall be 60% rather than 30%.

14.1.12.11 DAMPING DEVICES

14.1.12.11.1 Device Design, Construction and Installation

The design, construction, and installation of damping devices shall comply with the requirements of Section 18.2.4.1 for velocity-dependent devices.

14.1.12.11.2 Multiaxis Movement.

Connection points of damping devices shall comply with Section 18.2.4.2 for multi-axis movement.

14.1.12.11.3 Inspection and Periodic Maintenance

Requirements for inspection and periodic maintenance of damping devices shall comply with the requirements of Section 18.2.4.3.

14.1.12.11.4 Specified Design Properties

Specified design properties (damper constant and velocity exponent) for energy-dissipation devices shall be established from the design procedure. Damper performance within the allotted bounds of Section 14.1.12.11.5 shall be confirmed through either project-specific prototype test data or prior prototype tests on devices meeting the requirements of Section 18.6.2.3.

14.1.12.11.5 Maximum and Minimum Damper Properties

Maximum and minimum design properties for each device shall be determined in accordance with the following equations:

$$\text{Maximum Design Property} = \text{Specified Design Property} * \lambda_{(\max)} \quad (\text{Eqn. 14.1.12-33})$$

$$\text{Minimum Design Property} = \text{Specified Design Property} * \lambda_{(\min)} \quad (\text{Eqn. 14.1.12-34})$$

where $\lambda_{\max}$ is 1.15 and $\lambda_{\min}$ is 0.85.

Exception: At the discretion of the registered design professional and damper manufacturer, $\lambda_{\max}$ and $\lambda_{\min}$ are permitted to be taken greater than 1.15 and less than 0.85, respectively, for project conditions with expected temperature exposure greater than 120 degrees Fahrenheit (50 degrees Celsius) or less than 30 degrees Fahrenheit (0 degrees Celsius). In such cases, R_v used in Section 14.1.12.5.4 shall be equal to the alternative $\lambda_{\max}$ and Ω_F shall be adjusted accordingly.

14.1.12.12 TESTING

The force-velocity-displacement relationships and damping properties assumed as the specified design properties, $C_{(NL)_{\text{spec}}}$ and α , shall be confirmed by tests conducted in accordance with Section 18.6. Prototype testing is permitted to be waived in accordance with the requirements of Section 18.6.2.3.

Add Section 14.5.2 as follows:

14.5.2 Requirements for Post-tensioned Timber Rocking Walls

14.5.2.1 GENERAL

14.5.2.1.1 Scope

These provisions shall be used for the design and construction of all members and connections that are part of a Post-tensioned Timber Rocking Wall (PTRW) lateral force-resisting system (LFRS)

including timber panels, bounding columns (where present), post-tensioning components, energy dissipation components, and their connections. Design shall be performed using the Load and Resistance Factor Design (LRFD) procedure. The provisions provided herein shall be applied in combination with the requirements of the AWC SDPWS, the AWC NDS, this Standard, and the applicable building code.

14.5.2.1.2 Definitions and Abbreviations

Bounding Column. A column which serves as part of the PTRW lateral force-resisting system (LFRS) and which is connected to PTRW segments through energy dissipation components.

BRB. Buckling-restrained brace

Timber Panel. A single piece of structural composite lumber or cross-laminated timber.

PTRW. A PTRW includes all post-tensioned timber rocking wall (PTRW) segments in the same plane which are coupled together through energy dissipation components.

PTRW segment. A PTRW segment is the portion of a post-tensioned timber rocking wall (PTRW) lateral force-resisting system (LFRS) composed of timber panels connected together through horizontal and/or vertical splices.

UFP. U-shaped Flexural Plate

14.5.2.1.3 Notation

$A_{parallel}$ = area of vertical laminations of timber panel composed of cross-laminated timber, in².

C_1 = modification factor to relate displacements from linear analysis to expected inelastic displacements.

C_P = column stability factor.

D_{ufp} = diameter of UFP component measured center-to-center of thickness, in.

E' = adjusted modulus of elasticity, psi.

$E_{parallel}'$ = adjusted modulus of elasticity of the vertical laminations of timber panel composed of cross-laminated timber, psi.

E_h = horizontal seismic load effect in accordance with this Standard, lb or lb-in.

$E_{h,amp}$ = amplified horizontal seismic load effect in accordance with Section 14.5.2.7.3, lb or lb-in.

$(EI)_{app-min}'$ = adjusted apparent bending stiffness of cross-laminated timber for panel buckling stability calculations, lb-in²/ft of panel width.

E_{ufp} = elastic modulus of UFP component plate material, psi.

F_b' = adjusted bending design value for laminations used in timber panel, psi.

F_b^* = reference bending design value for laminations used in timber panel multiplied by all applicable adjustment factors except C_L , psi.

F_{bE} = critical buckling design value for timber panel, psi.

F_c^* = reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_P , lb.

$F_c(A_{parallel})'$ = adjusted compression design value of cross-laminated timber, lb.

$F_c(A_{parallel})^*$ = reference compression design value of cross-laminated timber multiplied by all applicable adjustment factors except C_P , lb.

$F_{ed,wh,j}$ = yield force in energy dissipation components at the PTRW heel for PTRW segment j , lb.

$F_{grav,j}$ = force corresponding to 0.9D gravity for PTRW segment j , lb.

$F_{pti,j}$ = initial force in post-tensioning components for PTRW segment j , lb.

$F_t(A_{parallel})'$ = adjusted tension design value of cross-laminated timber, lb.

$F_{x,hm}$ = Horizontal seismic force induced at level x representing higher modes, lb.

$M_{ed,rm,j}$ = nominal moment capacity of the rocking mechanism contributed by the energy dissipation components for PTRW segment j , lb-in.

$M_{n,rm,j}$ = nominal moment capacity of the rocking mechanism for PTRW segment j , lb-in.

$M_{n,rm,j}^*$ = nominal moment capacity of the rocking mechanism neglecting energy dissipation components for PTRW segment j , lb-in.

$M_{pr,rm,j}$ = probable moment capacity of the rocking mechanism for PTRW segment j , lb-in.

$M_{u,amp,j}$ = PTRW moment demand for PTRW segment j from this Standard's load combinations for strength design, excluding overstrength load combinations, with E_h replaced by $E_{h,amp}$, lb-in.

$M_{u,rm,j}$ = moment demand of the rocking mechanism from this Standard's load combinations for strength design, excluding overstrength load combinations, for PTRW segment j , lb-in.

P_{cE} = critical buckling design value for timber panel, lb.

Q_E = effect resulting from application of horizontal seismic forces in accordance with this Standard, lb or lb-in.

$Q_{E,hm}$ = effect resulting from application of $F_{x,hm}$ horizontal seismic forces, lb or lb-in.

R = response modification coefficient from this Standard.

R_t = ratio of expected tensile strength to specified minimum tensile strength taken in accordance with AISI 341.

S = edgewise bending section modulus, in³.

S_{DS} = design, 5% damped, spectral response acceleration parameter at short periods as defined in this Standard.

T = computed fundamental period of the structure in the direction under consideration, sec.

$V_{u,amp,j}$ = PTRW shear demand for PTRW segment j from this Standard's load combinations for strength design, excluding overstrength load combinations, with E_h replaced by $E_{h,amp}$, lb.

$V_{n,ufp}$ = nominal strength of a UFP component, lb.

$V_{pr,ufp}$ = probable strength of a UFP component, lb.

a = site class factor, 1/sec².

b_{ufp} = width of UFP component, in.

$f_b = f_{b1}$ = actual edgewise bending stress, psi.

f_c = actual compression stress, psi.

f_{ptu} = specified minimum tensile strength of a post-tensioning component, psi.

f_{pty} = specified minimum yield strength of a post-tensioning component, psi.

f_{ufpt} = specified minimum tensile strength of UFP component plate material, psi.

f_{ufpy} = specified minimum yield strength of UFP component plate material, psi.

f_v = actual shear stress in timber panel, psi.

h_c = height of the effective crushing zone in the timber panel, in.

h_n = height above the seismic base to the highest level of the seismic force-resisting system, ft.

h_w = height of the PTRW measured from rocking interface to highest diaphragm to which the PTRW attaches, ft.

h_x = height above the seismic base to level x , ft.

k_{ufp} = initial stiffness of a UFP component, lb/in.

$l_{crit,ufp}$ = the distance between the nearest attachment point for the UFP component and the beginning of the 180-degree bend, in.

l_{eb} = effective span length of timber panel for flatwise bending in accordance with NDS Section 3.3, in.

l_{ec} = effective span length of timber panel for flatwise compression buckling in accordance with NDS Section 3.7, in.

$l_{w,j}$ = length of PTRW segment j , in.

n = number of PTRW segments in a given PTRW.

t_{ufp} = thickness of UFP component, in.

$t_{v,j}$ = shear thickness of PTRW segment j at critical section, in.

$t_{w,j}$ = total thickness of PTRW segment j , in.

w_x = portion of the effective seismic weight that is located at or assigned to level x , lb.

Δ_{bm} = deformation for qualification testing of BRBs used as energy dissipation components, in.

δ_{DE} = design earthquake displacement in the plane of the PTRW at height h_w measured at the plan location of the center of the PTRW, in.

$\delta_{DE,y}$ = design earthquake displacement in the plane of the PTRW at height h_w measured at the plan location of the center of the PTRW and corresponding to development of the moment capacity of the rocking mechanism at the PTRW base, in.

ϵ_{mtmax} = maximum permissible compressive strain of a timber panel, in/in.

ϵ_{ptu} = strain at tensile strength of the post-tensioning component, in/in.

ϵ_{ufpu} = strain at tensile strength of the UFP component plate material, in/in.

θ_{design} = design gap opening rotation at the rocking interface, rad.

θ_{max} = maximum gap opening rotation at the rocking interface, rad.

ϕ = resistance factor.

ϕ_{rm} = resistance factor for the rocking mechanism.

ω_b = PTRW bending overstrength factor.

14.5.2.2 MATERIALS AND COMPONENTS

14.5.2.2.1 Timber Panels

Each timber panel shall conform to one of the following:

1. Structural composite lumber meeting the requirements of NDS Chapter 8. For timber panels composed of structural composite lumber, the edgewise compression stress-strain relationship and height of the effective inelastic compression zone, h_c , shall be in accordance with Section 14.5.2.2.1.2.
2. Cross-laminated timber meeting the requirements of NDS Chapter 10. For timber panels composed of cross-laminated timber, the edgewise compression stress-strain relationship and height of the effective inelastic compression zone, h_c , shall be in accordance with either Section 14.5.2.2.1.1 or Section 14.5.2.2.1.2.

14.5.2.2.1.1 Default Edgewise Compression Stress-Strain Relationship and h_c for Timber Panels Composed of Cross-Laminated Timber The timber panel edgewise compression stress-strain relationship shall be assumed in accordance with all of the following:

1. Be linearly elastic-perfectly plastic.
2. Have an elastic modulus equal to $E_{parallel}$.
3. Have a peak stress equal to $F_c(A_{parallel})^* / A_{parallel}$ except that the resistance factor, ϕ , in the calculation of $F_c(A_{parallel})^*$ shall be taken as 1.0.
4. Have a maximum permissible compressive strain, ϵ_{mtmax} , taken equal to the value of $3F_c(A_{parallel})^* / (E_{parallel}A_{parallel})$ except that the resistance factor, ϕ , in the calculation of $F_c(A_{parallel})^*$ shall be taken as 1.0.

The height of the effective crushing zone, h_c , shall be taken equal to the value of $2t_{w,j}$.

14.5.2.2.1.2 Alternate Edgewise Compression Stress-Strain Relationship and h_c for Timber Panels The timber panel edgewise compression stress-strain relationship and height of the effective crushing zone, h_c , shall be determined from edgewise compression testing of timber panels of the same grade and designation and provided by the same manufacturer as the timber panels used in construction. The value of ϵ_{mtmax} shall not be taken greater than the strain at which the post-peak stress reaches 80 percent of the peak compression stress determined from an edgewise compression stress-strain curve for the timber panel vertical direction. The value of h_c shall be determined at the point in time corresponding to ϵ_{mtmax} . For timber panels composed of structural composite lumber, the value of ϵ_{mtmax} shall not be taken greater than $6F_c^* / E'$ except that the resistance factor, ϕ , in the calculation of F_c^* shall be taken as 1.0. For timber panels composed of cross-laminated timber, the value of ϵ_{mtmax} shall not be taken greater than $6F_c(A_{parallel})^* / (E_{parallel}A_{parallel})$ except that the resistance factor, ϕ , in the calculation of $F_c(A_{parallel})^*$

shall be taken as 1.0. The value of h_c shall not be taken greater than $2t_{w,j}$. Timber panel test specimen length parallel to the applied load shall be sufficient to permit post-peak deformation localization away from the test setup boundary conditions.

14.5.2.2.2 Bounding Columns

Each bounding column, where present, shall conform to one of the following:

5. Sawn lumber meeting the requirements of NDS Chapter 4.
6. Structural glue laminated timber meeting the requirements of NDS Chapter 5.
7. Structural composite lumber meeting the requirements of NDS Chapter 8.
8. Cross-laminated timber meeting the requirements of NDS Chapter 10.

14.5.2.2.3 Post-Tensioning Components

Each post-tensioning component shall be unbonded from the timber panel over the full length of the post-tensioning component and shall conform to one of the following: ASTM A193 Grade B7; ASTM A354 Grade BC or BD; ASTM A449; ASTM A572 Grade 55, 60 or 65; ASTM F1554 Grade 55 or 105; ASTM A706 Grade 60 or 80

Each post-tensioning component shall be threaded with the same thread type over its full length.

EXCEPTION: Where the net area of the threaded portions equals or exceeds the gross area of the unthreaded portions, a post-tensioning component shall be permitted to be unthreaded except at the locations of couplers and nuts.

Couplers and nuts for post-tensioning components shall be capable of developing f_{ptu} in tension times the area of the post-tensioning components.

Post-tensioning components and their connections shall be detailed to prevent kinking of post-tensioning components at deformations associated with story drift and PTRW rocking.

Access that allows re-tensioning of the post-tensioning components shall be provided for the life of the structure.

Initial post-tensioning stresses shall not exceed the limits in Table 14.5.2.2.2. Energy dissipation components shall not be installed prior to post-tensioning unless the effect of force transfer through energy dissipation components is considered in the design.

Table 14.5.2.2.2 Maximum Initial Post-Tensioning Stresses

Stage	Location	Maximum Stress
		$0.94f_{pty}$
		$0.80f_{ptu}$
During stressing	At jacking end	Least of: Maximum jacking force recommended by the supplier of anchorage device
Immediately after force transfer	At post-tensioning anchorage devices and couplers	$0.70f_{ptu}$

14.5.2.2.4 Energy Dissipation Components

Each energy dissipation component shall conform to one of the following types:

1. U-shaped Flexural Plate (UFP) composed of ASTM A572 Grade 42 or Grade 50 steel or ASTM A240 Grade 304 stainless steel designed in accordance with Section 14.5.2.8.
2. Buckling-restrained braces (BRBs) qualified by testing in accordance with the procedures and acceptance criteria of ANSI/AISC 341 Section K3. Qualifying test results shall consist of a successful cyclic uniaxial test complying with ANSI/AISC 341 Section K3.3. Testing of a brace subassembly in accordance with ANSI/AISC 341 Section K3.2 shall not be required. The value of Δ_{bm} for ANSI/AISC 341 Section K3.4 shall be determined in accordance with Section 14.5.2.5.5. Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that considers the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes.
3. Yielding or frictional device qualified by testing in accordance with Section 18.6.2.2. The MCE_R device displacement shall be determined in accordance with Section 14.5.2.5.5.

14.5.2.3 SYSTEM REQUIREMENTS

14.5.2.3.1 LFRS Configuration

The PTRW lateral force-resisting system (LFRS) shall meet all of the following limitations:

1. Not more than one rocking interface shall occur for each PTRW segment.
2. The rocking interfaces for all PTRW segments within each structurally separate structure shall occur at the same elevation.
3. Inelastic response shall be limited to:
 - a. Timber panels at the rocking interface.
 - b. Energy dissipation components.
 - c. Post-tensioning components only for demands beyond θ_{design} .
4. PTRW segments shall be planar.
5. PTRW segments shall be balloon framed.

Exception: Where a PTRW segment does not continue above the highest diaphragm to which the PTRW attaches, the connection of the PTRW segment to that diaphragm shall be permitted to be platform-framed if the post-tensioning components connect directly to the PTRW segment. Connection of the post-tensioning components to the platform-framed diaphragm framing shall not be permitted.

6. Horizontal splices in PTRW segments, where present, shall be at the same vertical location over the length of the PTRW segment.
7. Vertical splices in PTRW segments, where present, shall be at the same horizontal location at full height.
8. Post-tensioning components and initial post-tensioning forces shall be balanced about the center of each PTRW segment length.
9. PTRWs shall not be permitted to be designed as Force-Transfer Around Openings (FTAO) Shear Walls or as Perforated Shear Walls in accordance with the AWC SDPWS.
10. Seismic criteria of this Standard specific to light-frame construction shall not apply to the design of PTRWs.

14.5.2.3.2 Non-LFRS Configuration

All elements and components not part of the designated LFRS shall accommodate the design of inter-story drift.

Timber wall panels that are not part of the designated LFRS and have an aspect ratio less than or equal to 5 shall not be permitted. Aspect ratio shall be measured as story height divided by wall panel length.

Exception: Non-load-bearing timber wall panels that are not part of the designated LFRS and have an aspect ratio less than or equal to 5 shall be permitted if they are detailed so as to not significantly affect the strength or stiffness of the LFRS.

14.5.2.4 MODELING AND ANALYSIS

14.5.2.4.1 General

Modeling and analysis of PTRW LFRSs shall be in accordance with this Standard and the additional requirements of this section. For the purposes of modeling and analysis of PTRWs, the rocking interface shall be represented by the stiffness before initiation of rocking.

14.5.2.4.2 Effective Stiffness for Linear Modeling

Effective stiffness of all elements and components shall be representative of the initial stiffness prior to yield. Effective stiffness for linear modeling of timber elements shall account for the effect of loading direction on the initial stiffness with respect to grain orientation and lamination direction. The edgewise shear flexibility of timber panels and the flexibility of vertical and horizontal splices in PTRW segments shall be considered.

14.5.2.5 DESIGN FOR EARTHQUAKE LOADS

14.5.2.5.1 Design Rotation at Rocking Interface

The design gap opening rotation at the rocking interface, θ_{design} , shall be computed in accordance with Equation 14.5-1 for each PTRW.

Exception: In lieu of the requirements of this section, the design gap opening rotation at the rocking interface, θ_{design} , for each PTRW shall be permitted to be computed from nonlinear response history analysis.

The value of C_1 shall be calculated in accordance with Equation 14.5-2 except that for periods less than 0.2 seconds, C_1 need not be taken greater than the value at a period equal to 0.2 sec and for periods greater than 1.0 sec, $C_1 = 1.0$. The value of a shall be taken as 130/sec² for Site Classes A and B, 110/sec² for Site Class BC, 90/sec² for Site Class C, 75/sec² for Site Class CD, and 60/sec² for Site Classes D, DE, E or F.

The value of δ_{DE} shall be computed under strength-level design earthquake forces in accordance with either of the following:

1. Using the equivalent lateral force procedure and using displacements determined in accordance with Equation 12.8-16. The exception to Section 12.8.6.1 shall be applied. Section 12.8.6.2 shall be applied.
2. Using the modal response spectrum analysis procedure and using displacements determined in accordance with Section 12.9.1.2 and 12.9.1.4.2.

The value of $\delta_{DE,y}$ shall be computed under earthquake forces corresponding to development of the moment capacity of the rocking mechanism at the PTRW base. It is permitted to determine $\delta_{DE,y}$ using strength-level design earthquake forces corresponding to the design base shear calculated in accordance with either of the following:

1. Using the equivalent lateral force procedure and using displacements determined in accordance with Equation 12.8-16. For the purposes of this calculation, C_d and I_e shall be taken as 1.0 only in Equation 12.8-16. The exception to Section 12.8.6.1 shall not be permitted. Section 12.8.6.2 shall not be permitted.
2. Using the modal response spectrum analysis procedure satisfying Section 12.9.1.2 and 12.9.1.4.1. For the purposes of this calculation, the last sentence of Section 12.9.1.2 shall be amended to read “The value of displacement and drift quantities shall not be further modified.”

The same analysis procedure shall be used for calculation of both δ_{DE} and $\delta_{DE,y}$.

$$\theta_{\text{design}} = \frac{C_1 \times \delta_{DE} - \delta_{DE,y}}{h_w} \quad (14.5-1)$$

$$C_1 = 1 + \frac{R-1}{aT^2} \quad (14.5-2)$$

14.5.2.5.2 Maximum Rotation at Rocking Interface

The maximum gap opening rotation at the rocking interface, $\theta_{\max}$, shall be computed in accordance with Equation 14.5-3 for each PTRW.

Exception: In lieu of the requirements of this section, the maximum gap opening rotation at the rocking interface, $\theta_{\max}$, for each PTRW shall be permitted to be computed from nonlinear response history analysis.

$$\theta_{\max} = \frac{1.5 \times C_1 \times \delta_{DE} - \delta_{DE,y}}{h_w} \quad (14.5-3)$$

14.5.2.5.3 Nominal Moment Capacity of the Rocking Mechanism

The nominal moment capacity of the rocking mechanism for PTRW segment j , $M_{n,r,m,j}$, shall be computed in accordance with all of the following:

1. Nominal material properties and nominal stress-strain material relationships per Section 14.5.2.2 shall be used. For a timber panel composed of cross-laminated timber, only the vertical laminations shall be considered.
2. Strain at any point in the compression zone at the rocking interface shall be taken equal to the product of the distance from the neutral axis and θ_{design}/h_c .
3. Gap opening at any point outside of the compression zone at the rocking interface shall be taken equal to the product of the distance from the neutral axis and θ_{design} .
4. The displacement at any point on the PTRW above the rocking interface shall be calculated as the displacement corresponding to Items 2 and 3 plus the displacement corresponding to the forces which produce $\delta_{DE,y}$.

14.5.2.5.4 Probable Moment Capacity of the Rocking Mechanism

The probable moment capacity of the rocking mechanism for PTRW segment j , $M_{pr,rm,j}$, shall be computed in accordance with all of the following:

1. Expected material properties and expected stress-strain material relationships per Section 14.5.2.2 shall be used. For a timber panel composed of cross-laminated timber, only the vertical laminations shall be considered.
2. Strain at any point in the compression zone at the rocking interface shall be taken equal to the product of the distance from the neutral axis and θ_{max}/h_c .
3. Gap opening at any point outside of the compression zone at the rocking interface shall be taken equal to the product of the distance from the neutral axis and θ_{max} .
4. The displacement at any point on the PTRW above the rocking interface shall be calculated as the displacement corresponding to Items 2 and 3 plus the displacement corresponding to the forces which produce $\delta_{DE,y}$.

14.5.2.5.5 Component Limits

The stress in each post-tensioning component determined in accordance with Section 14.5.2.5.3 shall not exceed the post-tensioning component yield stress, f_{pty} . The strain at the extreme compression fiber at the rocking interface of each PTRW segment determined in accordance with Section 14.5.2.5.3 shall not exceed ϵ_{mtmax} determined in accordance with Section 14.5.2.2.1.

The strain in each post-tensioning component determined in accordance with Section 14.5.2.5.4 shall not exceed 85 percent of the strain at tensile strength of the post-tensioning component, ϵ_{ptu} . The deformation in each energy dissipation component shall be checked in accordance with the following:

1. For U-shaped Flexural Plates (UFPs), the limits provided in Section 14.5.2.8.2 shall apply.

2. For buckling-restrained braces (BRBs), the deformation calculated in accordance with Section 14.5.2.5.3 Item 4 shall not exceed Δ_{bm} from testing in accordance with Section 14.5.2.2.4.
3. For all other energy dissipation components, the deformation calculated in accordance with Section 14.5.2.5.4 Item 4 shall not exceed the MCE_R device displacement from testing in accordance with Section 14.5.2.2.4.

14.5.2.5.6 Minimum Restoring

A minimum amount of restoring capability shall be provided by satisfying Equation 14.5-4 at each PTRW for both positive and negative loading directions.

Exception: Where post-tensioning components and gravity load are not located at the center of each PTRW segment length or where energy dissipation components at the PTRW heel are not located at the edge of each PTRW segment length, $l_{w,j}/2$ or $l_{w,j}$ shall be replaced by the respective distances to the PTRW toe.

$$\sum_{j=1}^n \left[(F_{pti,j} + F_{grav,j}) \times \frac{l_{w,j}}{2} \right] \geq \sum_{j=1}^n [F_{ed,wh,j} \times l_{w,j}] \quad (14.5-4)$$

14.5.2.5.7 Minimum Energy Dissipation

A minimum amount of energy dissipation shall be provided by satisfying Equation 14.5-5 at each PTRW for both positive and negative loading directions. The moment capacity of the rocking mechanism contributed by the energy dissipation components for PTRW segment j , $M_{ed,rm,j}$, shall be permitted to be calculated in accordance with Equation 14.5-6. The nominal moment capacity of the rocking mechanism neglecting energy dissipation components for PTRW segment j , $M_{n,rm,j}^*$, shall be permitted to be calculated in accordance with Section 14.5.2.5.3 but neglecting the contribution of the energy dissipation components.

$$\sum_{j=1}^n M_{ed,rm,j} \geq \frac{1}{8} \times \sum_{j=1}^n M_{n,rm,j} \quad (14.5-5)$$

$$M_{ed,rm,j} = M_{n,rm,j} - M_{n,rm,j}^* \quad (14.5-6)$$

14.5.2.6 DESIGN FOR WIND LOADS

14.5.2.6.1 Nominal Moment Capacity of the Rocking Mechanism

The nominal moment capacity of the rocking mechanism for PTRW segment j , $M_{n,rm,j}$, shall be computed in accordance with all of the following:

1. Nominal material properties shall be used.

2. Stress in the compression zone at the rocking interface shall vary linearly from the neutral axis location to the extreme end of the PTRW segment.
3. Force in the post-tensioning components shall be taken equal to the initial force in post-tensioning components for PTRW segment j , $F_{pti,j}$.
4. Force in the energy dissipation components shall be taken equal to their nominal strengths.

14.5.2.6.2 Component Limits

The stress at the extreme compression fiber at the rocking interface of each PTRW segment determined in accordance with Section 14.5.2.6.1 shall not exceed F_b^* divided by the strength reduction factor, ϕ , used in the calculation of F_b^* .

14.5.2.7 STRENGTH AND SERVICEABILITY

14.5.2.7.1 General

Strength and serviceability of PTRW LFRSs shall be in accordance with the additional requirements of this section. For earthquake loads, the requirements of Chapter 12 or 16, as applicable, shall also apply.

14.5.2.7.2 Flexural Strength of the Rocking Mechanism

The demand and capacity of the rocking mechanism shall satisfy Equation 14.5-7. The value ϕ_{rm} shall be equal to 0.9.

$$M_{u,rm,j} \leq \phi_{rm} \times M_{n,rm,j} \quad (14.5-7)$$

14.5.2.7.3 Amplification of Forces and Moments

For wind loads, amplification of forces and moments shall not be required.

For earthquake loads, the PTRW bending overstrength factor, ω_b , shall be calculated in accordance with Equation 14.5-8 at each PTRW. The ratio $\sum M_{pr,rm,j} / \sum M_{u,rm,j}$ shall be taken for this Standard's load combination for strength design, excluding overstrength load combinations, producing the largest value of ω_b .

$$\omega_b = \sum_{j=1}^n M_{pr,rm,j} / \sum_{j=1}^n M_{u,rm,j} \quad (14.5-8)$$

For earthquake loads, amplification of forces and moments in Sections 14.5.2.7.4 through 14.5.2.7.9 shall be determined by replacing the horizontal seismic load effect, E_h , in this Standard's load combinations for strength design, excluding overstrength load combinations, with $E_{h,amp}$ in accordance with Equation 14.5-9.

$$E_{h,amp} = \omega_b \times Q_E \pm 2.5 \times Q_{hm} \quad (14.5-9)$$

The value of Q_{hm} shall be taken as the effect resulting from application of horizontal seismic forces $F_{x,hm}$ at each level x in accordance with Equation 14.5-10.

$$F_{x,hm} = 0.55 \times w_x \times S_{DS} \times \sin \left(4.5 \times \left(1 - \frac{1}{2N} \right) \times \frac{h_x}{h_n} \right) \quad (14.5-10)$$

Exceptions:

1. For structures five stories or less above the seismic base, the value of $E_{h,amp}$ shall be permitted to be taken as $2 \times \omega_b \times Q_E$. Additionally, $E_{h,amp}$ used for design of an element or connection at any story in accordance with Sections 14.5.2.7.4 through 14.5.2.7.7 shall be taken as the maximum $E_{h,amp}$ over all stories.
2. The value of $E_{h,amp}$ shall be permitted to be determined from nonlinear response history analysis.
3. The value of $E_{h,amp}$ shall be permitted to be determined by a rational analysis that accounts for flexural overstrength of the rocking mechanism in the fundamental mode and reduced nonlinearity in modes other than the fundamental mode. The value of $E_{h,amp}$ shall not be taken less than $\max(\omega_b, 1.5) \times Q_E$.

14.5.2.7.4 Timber Panel In-Plane Shear

Timber panel in-plane shear stress at any section shall be determined in accordance with Equation 14.5-11.

$$f_v = \frac{3 \times V_{u,amp,j}}{2 \times t_{v,j} \times l_{w,j}} \quad (14.5-11)$$

14.5.2.7.5 Timber Panel Combined Edgewise Bending and Axial Loading

Combined edgewise bending and axial loading other than at the rocking interface which produces net tension at any point in the cross section for a timber panel composed of structural composite lumber shall satisfy Equations 14.5-12 and 14.5-13. Combined edgewise bending and axial loading other than at the rocking interface which produces net tension at any point in the cross section for a timber panel composed of cross-laminated timber shall satisfy Equations 14.5-12 and 14.5-14.

Combined edgewise bending and axial loading other than at the rocking interface which produces net compression at any point in the cross section for a timber panel composed of structural composite lumber shall satisfy NDS Section 3.9.2 with f_b and $f_{b\perp}$ determined in accordance with Equation 14.5-12. Combined edgewise bending and axial loading other than at the rocking interface which produces net compression at any point in the cross section for a timber panel composed of cross-laminated timber shall satisfy Equations 14.5-12 and 14.5-15 and NDS Section 3.3 and 3.7.1 as modified by Equations 14.5-16 through 14.5-18.

For a timber panel composed of cross-laminated timber, only the vertical laminations shall be considered when calculating f_c , f_b and S .

$$f_b = f_{b1} = \frac{M_{u,amp,j}}{S} \quad (14.5-12)$$

$$\frac{f_b - f_c}{F_b^*} \leq 1.0 \quad (14.5-13)$$

$$\frac{(f_b - f_c) \times A_{parallel}}{F_t(A_{parallel})'} \leq 1.0 \quad (14.5-14)$$

$$\left[\frac{f_c \times A_{parallel}}{F_c(A_{parallel})'} \right]^2 + \frac{f_b}{F_b' \times \left[1 - \frac{f_c \times A_{parallel}}{P_{cE}} \right]} \leq 1.0 \quad (14.5-15)$$

$$P_{cE} = \frac{\pi^2 \times (EI)'_{app-min}}{l_{ec}^2} \quad (14.5-16)$$

$$C_P = \frac{1 + P_{cE}/F_c(A_{parallel})^*}{2 \times 0.9} - \sqrt{\left[\frac{1 + P_{cE}/F_c(A_{parallel})^*}{2 \times 0.9} \right]^2 - \frac{P_{cE}/F_c(A_{parallel})^*}{0.9}} \quad (14.5-17)$$

$$F_{bE} = \frac{2.4 \times (EI)_{app-min}'}{S \times l_{eb}} \quad (14.5-18)$$

14.5.2.7.6 Timber Panel Horizontal Splice

Timber panel horizontal splices, where present, shall be designed for shear and for combined bending and axial loading with principles of engineering mechanics using design values for wood members and connections in accordance with the NDS provisions. The in-plane shear and edgewise bending demands at the splice location shall be taken as the shear and moment demands, respectively, from this Standard's load combinations for strength design, excluding overstrength load combinations, with E_h replaced by $E_{h,amp}$ and shall be taken as the greater of the following:

1. Shear and moment demands at the splice location.
2. One-half the maximum shear and moment demands over all stories.

14.5.2.7.7 Timber Panel Vertical Splice

Timber panel vertical splices, where present, shall be designed in accordance with applicable material standards and the NDS provisions. The in-plane shear demand at the splice location shall be taken as the shear demand from this Standard's load combinations for strength design, excluding overstrength load combinations, with E_h replaced by $E_{h,amp}$.

14.5.2.7.8 PTRW Segment In-Plane Shear Connection at the Rocking Interface

Connections to resist PTRW segment in-plane shear at the rocking interface shall be designed in accordance with applicable material standards and the NDS provisions. The shear demand transferred out of the PTRW segment at the rocking interface shall be taken as the shear demand from this Standard's load combinations for strength design, excluding overstrength load combinations, with E_h replaced by $E_{h,amp}$.

14.5.2.7.9 PTRW Segment to Diaphragm Connection

PTRW segment to diaphragm connections, including collectors, shall be designed in accordance with applicable material standards and the NDS provisions. The force demand for the PTRW segment to diaphragm connection shall be taken as the force demand from this Standard's load combinations for strength design, excluding overstrength load combinations, with E_h replaced by $E_{h,amp}$. Where this Standard requires the use of overstrength load combinations, the force demand for PTRW segment to diaphragm connections, including collectors, shall not be less than the force demand from the overstrength load combinations using E_h .

PTRW segment to diaphragm connections shall resist the force demands listed above in combination with the displacements associated with story drift and PTRW rocking.

14.5.2.7.10 Bounding Columns

Bounding columns, splices of bounding columns, and bounding column base connections shall be designed in accordance with applicable material standards and the NDS provisions. The force demands for bounding columns, splices of bounding columns, and bounding column base connections shall be taken as the force demand from this Standard's overstrength load combinations for strength design.

Exception: The seismic force demands on a bounding column, splice of a bounding column, and bounding column base connection need not exceed those corresponding to development of the sum of the probable capacity and/or adjusted brace strength of all energy dissipation elements connected to the bounding column. The probable capacity of UFPs shall be calculated in accordance with Section 14.5.2.8.3. The adjusted brace strength of BRBs shall be calculated in accordance with ANSI/AISC 341 Section F4.

14.5.2.7.11 Post-Tensioning and Energy Dissipation Connections

Post-tensioning and energy dissipation connections shall be designed in accordance with applicable material standards and the NDS provisions. Seismic force demands on post-tensioning connections shall be determined corresponding to the development of the tensile capacity of the post-tensioning components. Seismic force demands on energy dissipation connections shall be determined corresponding to the development of the probable capacity and/or adjusted brace strength of the energy dissipation components. The probable capacity of UFPs shall be calculated in accordance with Section 14.5.2.8.3. The adjusted brace strength of BRBs shall be calculated in accordance with ANSI/AISC 341 Section F4.

14.5.2.7.12 Other Structural Components and Connections

Structural components and connections other than those subject to Section 14.5.2.7.4 through 14.5.2.7.11 shall be designed in accordance with applicable material standards and the NDS provisions. The force or moment demand from horizontal earthquake load, E_h , in this Standard's load combinations for strength design, excluding overstrength load combinations, shall be multiplied by ω_b .

Exception: For soil actions, geotechnical limit states, overturning stability, sliding, diaphragms, and diaphragm chords, ω_b shall be permitted to be taken as 1.0.

Where this Standard requires the use of overstrength load combinations, the force or moment demand for structural components and connections shall not be less than the force or moment demand from the overstrength load combinations using E_h .

14.5.2.7.13 Post-Tensioning Losses

The requirements of Sections 14.5.2.5 through 14.5.2.7 shall be satisfied independently for both of the following cases:

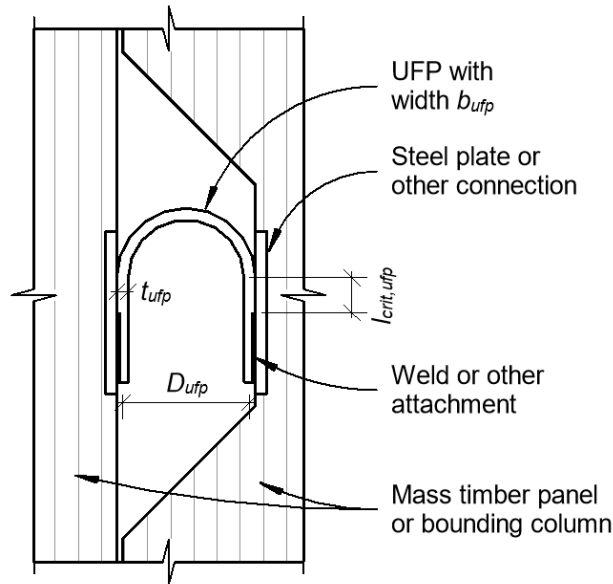
1. Initial prestress forces in the post-tensioning components equal to the specified value(s) on the construction documents.
2. Prestress forces in the post-tensioning components equal to reduced value(s) that account for post-tensioning losses including all of the following
 - a. Seating of post-tensioning components.
 - b. Relaxation of post-tensioning components.
 - c. Force transfer through energy dissipation components.
 - d. Shortening due to creep and shrinkage of the timber panels. It shall be permitted to calculate the creep component of shortening as $(K_{cr} - 1)$ multiplied by the immediate axial shortening of the PTRW. K_{cr} shall be permitted to be determined in accordance with AWC NDS Section 3.5.2. The immediate axial shortening of the PTMRW shall be permitted to be calculated due to the long-term component of gravity loads plus the initial prestress in the post-tensioning components.

14.5.2.8 DESIGN OF U-SHAPED FLEXURAL PLATES

14.5.2.8.1 General

The provisions in this section apply to the design of U-Shaped Flexural Plate (UFP) components that consist of steel plates bent through 180 degrees. See Figure 14.5.2.8.1.

Figure 14.5.2.8.1 UFP Component



14.5.2.8.2 Configuration Limits

The ratio t_{ufp}/D_{ufp} shall not exceed 85-percent of the strain at tensile strength of the UFP component plate material, ϵ_{ufpu} .

The distance between the nearest attachment point for the UFP component and the beginning of the 180-degree bend, $l_{crit,uFP}$, shall not be less than two-thirds times the shear displacement in the UFP component calculated in accordance with Section 14.5.2.5.4 Item 4. Sufficient clearance shall be provided around the UFP to permit unrestricted roll-out of the UFP to four-thirds times the shear displacement in the UFP component calculated in accordance with Section 14.5.2.5.4 Item 4.

14.5.2.8.3 Stiffness and Strength

The initial stiffness of a UFP component, k_{ufp} , shall be determined in accordance with Equation 14.5-19. The nominal strength of a UFP component, $V_{n,uFP}$, for use in Section 14.5.2.5.3 shall be determined in accordance with Equation 14.5-20. The probable strength of a UFP component, $V_{pr,uFP}$, for use in Section 14.5.2.5.4 shall be determined in accordance with Equation 14.5-21.

$$k_{ufp} = \frac{16 \times E_{ufp} \times b_{ufp}}{27\pi} \left(\frac{t_{ufp}}{D_{ufp}} \right)^3 \quad (14.5-19)$$

$$V_{n,uFP} = \frac{f_{ufpy} \times b_{ufp} \times t_{ufp}^2}{2 \times D_{ufp}} \quad (14.5-20)$$

$$V_{pr,uFP} = \frac{R_t \times f_{ufpt} \times b_{ufp} \times t_{ufp}^2}{2 \times D_{ufp}} \quad (14.5-21)$$

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CHAPTER 15, SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURES (Modifications)

Revise Section 15.4.10 as follows:

15.4.10 Requirements for Nonbuilding Structure Foundations on Liquefiable Sites Nonbuilding structure foundations on liquefiable sites shall comply with Section 12.13.9 and the requirements of Section 15.4.10.1.

15.4.10.1 Nonbuilding Structures on Shallow Foundations Nonbuilding structures shall not be permitted to be supported on shallow foundations at liquefiable sites unless it can be demonstrated that the structure's foundation, superstructure, and connecting systems can be designed to accommodate the soil strength loss, lateral spreading, and total and differential settlements induced by the applicable earthquake ground motions (as defined in Section 11.8.3) indicated in the geotechnical investigation report

Revise Section 15.6.1 as follows:

15.6.1 Earth-Retaining Structures This section applies to stand-alone earth-retaining structures (i.e., not connected to other structures) assigned to Seismic Design Category D, E, or F. The lateral earth pressures caused by earthquake ground motions shall be determined in accordance with Section 11.8.3 and Section 12.7.5. The risk category shall be determined by the proximity of the earth-retaining structure to other buildings and structures. If failure of the earth-retaining structure would affect the adjacent building or structure, the risk category shall not be less than that of the adjacent building or structure.

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CHAPTER 16, NONLINEAR RESPONSE HISTORY ANALYSIS (Modifications)

Revise Chapter 16 as follows:

16.1 GENERAL REQUIREMENTS

16.1.1 Scope It shall be permitted to use nonlinear response history analysis, in accordance with the requirements of this chapter, to design buildings or nonbuilding structures similar to buildings in lieu of the requirements of Section 12.6. When nonlinear response history analysis is performed, the design shall also satisfy the requirements of Section 16.1.2. Nonlinear response history analysis shall include the effects of horizontal motion, and where required by Section 16.1.3, vertical motion. Documentation of the design and analysis shall be prepared in accordance with Section 16.1.4. Ground motion acceleration histories shall be selected and modified in accordance with the procedures of Section 16.2. The structure shall be modeled and analyzed in accordance with the criteria in Section 16.3. Analysis results shall meet the acceptance criteria of Section 16.4. Independent structural design review shall be performed in accordance with the requirements of Section 16.5.

16.1.2 Structural Design Basis For buildings, the structure shall satisfy the requirements of Sections 12.1 and this chapter. The structural system shall meet the requirements of Section 12.2.1. Structural systems identified as Not Permitted in Table 12.2-1 for the structure's Seismic Design Category shall not be used. Structural systems permitted in Table 12.2-1 shall be permitted to exceed the height limits in the table. Combinations of seismic force-resisting systems in each direction or in the same direction shall be permitted, provided the seismic force-resisting systems meet the requirements of Section 12.2.1. Nonbuilding structures similar to buildings shall also be designed per the requirements of Section 15.5 and this chapter.

16.1.3 Vertical Response Analysis Nonlinear response history analysis shall explicitly include the effects of vertical response where any of the following occur:

1. Vertical elements of the gravity force-resisting system are discontinuous.
2. For nonbuilding structures when Chapter 15 requires consideration of vertical earthquake effects.

16.1.4 Documentation Before performing the nonlinear analysis, project-specific design criteria shall be approved by the independent structural design reviewer(s) and the Authority Having Jurisdiction. The project-specific criteria shall identify the following:

1. The selected seismic and gravity force-resisting systems and procedures used in the structural design.
2. Geotechnical parameters, including soil characteristics, recommended foundation types, design parameters, seismic hazard evaluation, target spectra, and the selection and scaling of acceleration histories.
3. Approach for modeling the foundation/soil interface, in particular determination of whether fixed-based modeling is acceptable, or modeling foundation lateral and/or vertical flexibility is required. The foundation design shall be performed in accordance with Section 16.3.1 and 16.3.9.
4. Design loading, including gravity and environmental loads.
5. Analytical modeling approach and assumptions, including software to be used, definition of mass, identification of force-controlled and deformation-controlled behaviors, description of which component actions are modeled elastically and inelastically, expected material properties, basis for hysteretic component modeling, component initial stiffness assumptions, joint stiffness assumptions, diaphragm modeling, damping, and procedure for modeling foundation-soil interaction.
6. Summaries of laboratory test and other applicable data used to justify the hysteretic component modeling or used to justify acceptable structural performance.
7. Specific acceptance criteria values used for evaluating the performance of elements of the seismic force-resisting system. Associated documentation shall also include identification of component failure modes deemed indicative of collapse.
8. Where drifts exceed 150% of the values permitted in Section 12.12, the criteria used to demonstrate acceptable deformation compatibility of components of the gravity force-resisting system.

Following completion of the analysis process, the following documentation shall be prepared and presented to the independent structural design reviewer(s) and the Authority Having Jurisdiction:

1. Final geotechnical report, including soil shear strength, stiffness, and damping characteristics; recommended foundation types and design parameters; and seismic hazard evaluation, including both the target spectra and selection and scaling of ground motions.
2. Overall building dynamic behavior, including natural frequencies, mode shapes, and modal mass participation.
3. Key structural system response parameter results and comparisons with the acceptance criteria of Section 16.4.

4. Detailing of critical elements.

16.2 GROUND MOTIONS

16.2.1 Target Response Spectrum A target, 5%-damped, MCE_R response spectrum shall be developed using either the procedures of Section 16.2.1.1 or Section 16.2.1.2. It shall be permitted to consider the effects of base slab averaging and foundation embedment in accordance with Chapter 19.

Where the effects of vertical earthquake shaking are included in the analysis, a target MCE_R vertical spectrum shall also be constructed.

16.2.1.1 Method 1 A single target response spectrum shall be developed, based on the requirements of either Section 11.4.6 or Section 11.4.7.

16.2.1.2 Method 2 Two or more site-specific target response spectra shall be developed. When this method is used, the following requirements shall be fulfilled, in addition to the other requirements of this chapter:

1. Two or more periods shall be selected, corresponding to those periods of vibration that significantly contribute to the inelastic dynamic response of the building in two orthogonal directions. In the selection of periods, lengthening of the elastic periods of the model shall be considered.
2. For each selected period, a target spectrum shall be created that either matches or exceeds the MCE_R value at that period. When developing the target spectrum, (1) site-specific disaggregation shall be performed to identify earthquake events that contribute most to the MCE_R ground motion at the selected period, and (2) the target spectrum shall be developed to capture one or more spectral shapes for dominant magnitude and distance combinations revealed by the disaggregation.
3. The envelope of the target spectra shall not be less than 75% of the spectral values computed using Method 1 of Section 16.2.1.1, for all periods in the range specified in Section 16.2.3.1.
4. For each target response spectrum, a ground motion suite for response history analyses shall be developed and used in accordance with Sections 16.2.3 through 16.2.4. The acceptance criteria requirements of Section 16.4 shall be evaluated independently for each of the ground motion suites.

Variations on the procedures described in this section are permitted to be used when approved by the design review.

16.2.2 Ground Motion Selection A suite of not less than 11 ground motions shall be selected for each target spectrum. Ground motions shall consist of pairs of orthogonal horizontal ground motion

components and, where vertical earthquake effects are considered, a single vertical ground motion component. Ground motions shall be selected from events within the same general tectonic regime and having generally consistent magnitudes and fault distances as those controlling the target spectrum and shall have a spectral shape similar to the target spectrum. For near-fault sites, as defined in Section 11.4.1, and other sites where MCE_R shaking can exhibit directionality and impulsive characteristics, the proportion of ground motions with near-fault and rupture directivity effects shall represent the probability that MCE_R shaking will exhibit these effects. Where the required number of recorded ground motions is not available, it shall be permitted to supplement the available records with simulated ground motions. Ground motion simulations shall be consistent with the magnitudes, source characteristics, fault distances, and site conditions controlling the target spectrum.

16.2.3 Ground Motion Modification Ground motions shall either be amplitude-scaled in accordance with the requirements of Section 16.2.3.2 or spectrally matched in accordance with the requirements of Section 16.2.3.3. Spectral matching shall not be used for near-fault sites unless the pulse characteristics of the ground motions are retained after the matching process has been completed.

16.2.3.1 Period Range for Scaling or Matching A period range shall be determined, corresponding to the vibration periods that contribute significantly to the building's lateral dynamic response. The upper bound of the period range shall be greater than or equal to twice the largest first-mode period in the principal horizontal directions of response, unless a lower value, not less than 1.5 times the largest first-mode period, is justified by dynamic analysis when subjected to MCE_R ground motions. The lower bound period shall be established such that the period range includes at least the number of elastic modes necessary to achieve 90% mass participation in each principal horizontal direction. The lower bound period shall not exceed 20% of the smallest first-mode period for the two principal horizontal directions of response. Where vertical response is considered in the analysis, the lower bound period used for modification of vertical components of ground motion need not be taken as less than the larger of 0.1 seconds or the lowest period at which significant vertical mass participation occurs.

16.2.3.2 Amplitude Scaling For each horizontal ground motion pair, a maximum-direction spectrum shall be constructed from the two horizontal ground motion components. Each ground motion shall be scaled, with an identical scale factor applied to both horizontal components, such that the average of the maximum-direction spectra from all ground motions generally matches or exceeds the target response spectrum over the period range defined in Section 16.2.3.1. The average of the maximum-direction spectra from all the ground motions shall not fall below 90% of the target response spectrum for any period within the same period range. Where vertical response is considered in the analysis, the vertical component of each ground motion shall be scaled such that the average of the vertical response spectra envelops the target vertical response spectrum over the period range specified in Section 16.2.3.1.

16.2.3.3 Spectral Matching Each pair of ground motions shall be modified so the average of the maximum-direction spectra for the suite equals or exceeds 110% of the target spectrum over the

period range defined in Section 16.2.3.1. Where vertical response is considered in the analysis, the vertical component of each ground motion shall be spectrally matched to the target vertical response spectrum, so the average of the matched spectra does not fall below the target vertical spectrum in the scaling range of Section 16.2.3.1.

16.2.4 Application of Ground Motions to the Structural Model Ground motions shall be applied to the supports of the structural model. For near-fault sites, as defined in Section 11.4.1, each pair of horizontal ground motion components representative of a nearby fault source shall be rotated to the fault-normal and fault-parallel directions of the causative fault and applied to the building in such orientation. For all other selected ground motions at near-fault sites, and for all ground motions at other sites, each pair of horizontal ground motion components shall be applied to the building at orthogonal orientations, such that the average (or mean) of the component response spectrum for the records applied in each direction is within $\pm 10\%$ of the mean of the component response spectra of all records applied for the period range specified in Section 16.2.3.1.

16.3 MODELING AND ANALYSIS

16.3.1 Modeling Mathematical models shall be three-dimensional and shall conform to the requirements of this section and Section 12.7. For structures that have subterranean levels, the structural model shall extend to the foundation level, and ground motions shall be input at the foundation level. All elements that significantly affect seismic response when subjected to MCE_R ground motions shall be included. Modeling of element nonlinear hysteretic behavior shall be consistent with ASCE 41 or applicable laboratory test data. Test data shall not be extrapolated beyond tested deformation levels. Degradation in element strength or stiffness shall be included in the hysteretic models unless it can be demonstrated that response is not sufficient to produce these effects.

Analysis models shall be capable of representing the flexibility of floor diaphragms where this is significant to the structure's response. Diaphragms at horizontal and vertical discontinuities in lateral resistance shall be explicitly modeled in a manner that permits capturing the force transfers and resulting deformations.

16.3.2 Gravity Load The modeling of, and demands on, elements in the analysis model shall be determined considering earthquake effects acting in combination with expected gravity loads, both with and without live load. Gravity loads with live load shall be taken as $1.0D + 0.5L$, where L shall be taken as 80% of unreduced live loads that exceed 100 lb/ft^2 (4.79 kN/m^2) and 40% of all other unreduced live loads. Gravity loads without live load shall be taken as $1.0D$.

EXCEPTION: Where the sum, over the entire structure of the expected live load ($0.5L$), as defined above, does not exceed 25% of the total dead load, D , and the live load intensity, L_o , over at least 75% of the structure is less than 100 psf (4.79 kN/m^2), the case without live load need not be considered.

16.3.3 Seismic Loads Earthquake effects in the analysis model shall be determined using MCE_R ground motions per Section 16.2. For Risk Category IV buildings, an additional analysis using Design Earthquake ground motions shall be performed. The Design Earthquake suite shall be derived by scaling the selected MCE_R records per Section 16.2 by a factor of 0.67. Alternatively, a new suite of ground motion records may be selected and scaled per Section 16.2.

16.3.4 P-Delta Effects P-delta effects, considering the spatial distribution of gravity loads, shall be included in the analysis.

16.3.5 Torsion

Inherent eccentricity resulting from any offset in the centers of mass and stiffness at each level shall be accounted for in the analysis. In addition, accidental eccentricity consisting of an assumed displacement of the center of mass each way from its actual location by a distance equal to 5% of the diaphragm dimension of the structure parallel to the direction of mass shift shall be considered. The required 5% displacement of the center of mass need not be applied in both orthogonal directions at the same time.

EXCEPTION:

It shall be permitted to exclude accidental torsion if the value of the torsional index, J , is less than 1.2 at every level. The torsional index shall be calculated as $J = (\delta_{max,ta} / \delta_{max,t})$ where $\delta_{max,t}$ is the maximum displacement at Level x computed considering inherent torsion, and $\delta_{max,ta}$ is the maximum displacement at Level x computed considering inherent plus accidental torsion where the mass is offset in the direction that produces the maximum torsional index, J . The torsional index, J , may be computed on the basis of either a linear or nonlinear analysis. For computing $\delta_{max,ta}$ and $\delta_{max,t}$, it shall be permitted to assume the diaphragms are rigid.

16.3.6 DAMPING

Hysteretic energy dissipation of structural members shall be modeled directly. Equivalent viscous damping, associated with damping provided by elements and components not modeled directly, shall be modeled using Rayleigh damping, modal damping, or other rational methodology. A combination of methods shall be permitted to capture this damping effect. Where equivalent viscous damping is implemented using mass and stiffness proportional methods, the equivalent viscous damping ratios shall be applied such that:

1. The average elastic equivalent viscous damping ratio, weighted by mass participation over the modes required to achieve 90% mass participation, shall not exceed the equivalent viscous damping ratio.
1. Equivalent viscous damping applied in highest translational mode required to achieve 90% mass participation shall not exceed eight times the equivalent damping applied to the first translational mode, unless substantiated through analysis or test data.

Equivalent viscous damping shall not exceed the equivalent viscous damping ratio modeled appropriate to the structure type and shall not exceed 2.5% equivalent viscous damping in the significant modes of response.

16.3.7 Diaphragms, Chords, and Collectors Diaphragms and chords shall be permitted to be evaluated in accordance with one of the following:

1. When the nonlinear hysteretic behavior of diaphragms and chords is explicitly modeled, they shall be evaluated as deformation-controlled in accordance with Section 16.4.2.2.
2. When the nonlinear hysteretic behavior of diaphragms and chords is not explicitly modeled, they shall be evaluated as force-controlled in accordance with Section 16.4.2.1. The value of $Q_u - Q_{ns}$ in Section 16.4.2.1 shall be determined based on the mean of the maximum floor accelerations times the weight tributary to the diaphragm, w_{px} , plus the mean of the maximum force being transferred through portions of the diaphragm between vertical components of the seismic forced resisting system at the level under consideration. The value of $1.3 \cdot I_e \cdot (Q_u - Q_{ns})$ in Section 16.4.2.1 shall not be taken less than Eq. (12.10-2).
3. When the diaphragms and chords are modeled with linear elastic elements, they shall be evaluated as force-controlled in accordance with Section 16.4.2.1. The value of $Q_u - Q_{ns}$ in Section 16.4.2.1 shall be determined based on the mean of the maximum force in the diaphragm. The value of $1.3 \cdot I_e \cdot (Q_u - Q_{ns})$ in Section 16.4.2.1 shall not be taken less than Eq. (12.10-2).
4. When the nonlinear hysteretic behavior of diaphragms and chords is not explicitly modeled and the diaphragms do not transfer forces between vertical seismic force-resisting elements, the design forces for diaphragms and chords shall be in accordance with the provisions of Chapter 12.

Collectors shall be permitted to be evaluated in accordance with one of the following:

1. When the nonlinear hysteretic behavior of collectors is not explicitly modeled, collectors shall be evaluated as force-controlled in accordance with Section 16.4.2.1.
2. When the nonlinear hysteretic behavior of collectors is explicitly modeled, collectors shall be evaluated as deformation-controlled in accordance with Section 16.4.2.2, except that their connections shall be evaluated as force-controlled in accordance with Section 16.4.2.1.

16.3.8 Structural Walls and Their Anchorage Structural walls and their anchorage shall satisfy the requirements of Section 12.11.

16.3.9 Foundation Design The design basis for the foundations shall be in accordance with Section 12.13, except that Sections 12.13.4, 12.13.5.3, and 12.13.6 shall not apply. Actions between the

foundation components and the soil shall be permitted to be modeled explicitly as deformation-controlled actions. Structural foundation components and the structure's connection to the foundation components shall be permitted to be designed as deformation-controlled subject to provisions in AISC 341 for structural steel, ACI 318 for reinforced concrete, and ASCE 41 for all other materials.

When soil spring elements are used to represent deformations of the foundation-soil system and/or dashpot elements are used to represent soil and foundation damping in the structural model, horizontal input ground motions shall be applied to the soil elements rather than being applied to the foundation elements directly.

When the structure is modeled with a pinned or fixed base where actions between the foundation components and the soil are not explicitly modeled, soil actions and overturning stability shall be considered non-critical, force-controlled actions.

16.3.10 Prohibited Irregularities Structures assigned to Seismic Design Categories B through D shall not have vertical irregularity Type 4b per Table 12.3-2. Structures assigned to Seismic Design Categories E and F shall not have a vertical irregularity Type 4a per Table 12.3-2

EXCEPTION: A vertical irregularity Type 4a or b is permitted under the following conditions:

1. The structure is assigned to Seismic Design Category B or C and is two-stories or 30 ft (9 m) in structural height or less.
2. The sum of the height of stories immediately above a weak story shall not exceed 25% of the structural height
3. No ground motion produces unacceptable responses as defined in Section 16.4.1.1.

16.4 ANALYSIS RESULTS AND ACCEPTANCE CRITERIA

Structures shall be demonstrated to meet the global acceptance criteria of Section 16.4.1 and the element-level acceptance criteria of Section 16.4.2.

The mean value of story drift, and element demand, Q_u , shall be used to evaluate acceptability.

EXCEPTION: Where a ground motion produces unacceptable response as permitted in Section 16.4.1.1, 120% of the median value, but not less than the mean value obtained from the suite of analyses producing acceptable response shall be used.

16.4.1 Global Acceptance Criteria

16.4.1.1 Unacceptable Response Unacceptable response to ground motion shall consist of any of the following:

1. Analytical solution fails to converge,
2. Predicted demands on deformation-controlled elements exceed the valid range of modeling,
3. Predicted demands on critical or ordinary force-controlled elements, as defined in Section 16.4.2, exceed the element expected strength, R_{ne} , as established in accordance with Section 16.4.2.1,
4. Predicted deformation demands on elements not explicitly modeled exceed the deformation limits at which the members are no longer able to carry their gravity loads,
5. Peak transient story drift ratio exceeds 150% of the permissible value of mean transient story drift, as per Section 16.4.1.3,
6. For structures exceeding 240 ft (73m) in height, the residual story drift for any story exceeds a value of $0.015 h_{sx}$; or
7. Gravity connections or supports for members spanning between structures cannot accommodate the maximum relative displacements.

Unacceptable response to any of the ground motions shall not be permitted.

EXCEPTION: For Risk Category I and II structures, where spectral matching of ground motion is not used, not more than one motion shall be permitted to produce unacceptable response.

16.4.1.2 Minimum Design Strength The mean of the maximum base shear in each orthogonal direction shall not be less than the value given by Eq. (12.8-1). Where the structure employs a combination of framing systems, the provisions of 12.2.3 shall be applicable to determine the minimum design strength.

16.4.1.3 Transient Story Drift The mean transient story drift for all building heights, $\bar{\Delta}$ in ft (m), shall not exceed two times the limits of Table 12.12-1. Additionally, for structures exceeding 100 ft (30 m) in height, $\bar{\Delta}$ shall not exceed the value obtained from Equation (16.4-1):

$$\bar{\Delta} \leq h_{sx} (4.71 \times 10^{-2} - 7.14 \times 10^{-5} h_n) \quad (16.4-1)$$

$$\bar{\Delta} \leq h_{sx} (4.71 \times 10^{-2} - 2.34 \times 10^{-4} h_n) \quad (16.4-1.SI)$$

where h_n and h_{sx} are measured in ft (m). The value obtained from Equation (16.4-1) need not be taken less than $0.03 h_n$.

The mean transient story drift for Risk Category IV building, $\bar{\Delta}_{DE}$ in ft (m), shall not exceed 1.3 times the limits of Table 12.12-1 when subjected to a suite of Design Earthquake ground motion records, as prescribed in Section 16.3.3.

The transient story drift ratio shall be computed as the absolute value of the largest difference in the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure within a single response history analysis. For masonry shear wall structures, the limits of Table 12.12-1 applicable to masonry cantilever wall structures and other masonry wall structures shall not apply, and these structures shall instead comply with the limits for other structures.

16.4.1.4 Residual Story Drift For structures exceeding 240 ft (73 m) in height, the mean residual story drift shall not exceed $0.01 h_{sx}$, where residual story drift is taken as the maximum value of story drift in a structure at rest, following response to an earthquake motion.

16.4.1.5 Members Spanning Between Structures Gravity connections or supports for members spanning between structures or seismically separate portions of structures shall be designed for $2.0I_e$ times the mean of the maximum relative displacement between the members.

16.4.1.6 Structural Separation All portions of the structure shall meet the requirements of Section 12.12.2. It shall be permitted to define the Design Earthquake Displacements as two-thirds of the mean of the maximum displacements from each record.

16.4.2 Element-Level Acceptance Criteria All element actions shall be classified either as force-controlled or deformation-controlled, in accordance with ACI 318 for reinforced concrete elements, AISC 342 for structural steel and topped and untopped steel deck diaphragms or ASCE 41 for elements of other materials.

Where the structure has a horizontal irregularity of Type 4 per Table 12.3-1 or a vertical irregularity of Type 3 per Table 12.3-2, the component actions of structural elements supporting the discontinuous wall or frames and the component actions of all connections between those elements shall be considered critical force controlled.

All modeled elements' component actions where the nonlinear hysteretic behavior is not explicitly included shall be considered force-controlled.

For each element action, the quantity, Q_u , shall be computed. Q_u shall be taken as the mean value of the response parameter of interest obtained from the suite of analyses.

Force-controlled actions shall be evaluated for acceptability in accordance with Section 16.4.2.1. Deformation-controlled actions shall be evaluated for acceptability in accordance with Section 16.4.2.2. Element actions shall be categorized as Critical, Ordinary, or Noncritical.

16.4.2.1 FORCE-CONTROLLED ACTIONS

Force-controlled actions shall satisfy Equation (16.4-1) and (16.4-2):

$$(1.2+0.12S_{MS})D+0.5L+1.3I_e(Q_u-Q_{ns}) \leq \phi BR_n \quad (16.4-1)$$

$$(0.9-0.12S_{MS})D+1.3I_e(Q_u-Q_{ns}) \leq \phi BR_n \quad (16.4-2)$$

where D and L are as defined in Section 16.3.2, S_{MS} is the site-adjusted Maximum Considered Earthquake Spectral Acceleration at a period of 0.2 seconds; I_e is the Importance Factor prescribed in Section 1.5.1; Q_u is the mean value of the demand computed from the suite of analyses; Q_{ns} is the portion of the demand caused by loads other than seismic; R_n is the nominal strength specified by the applicable material standard. The resistance factor ϕ for Critical elements shall be taken as the value specified by the applicable material standard. The resistance factor ϕ for Ordinary elements shall be taken as 0.9. The resistance factor ϕ for Noncritical elements shall be taken as 1.0. B is a factor to account for differences between expected strength R_{ne} and nominal resistance R_n . It is permitted to assign B a value of 1.0, or, alternatively, B can be taken as $0.9R_{ne}/R_n$, where R_{ne} is the expected strength of the element.

Where an industry standard referenced in Chapter 14 defines expected strength, that value shall be used. Where this is not defined, it shall be permitted to calculate expected strength as the nominal strength defined in industry standards, except that expected material properties as defined in ACI 318 for reinforced concrete elements, ANSI/AISC 341 for structural steel elements, ANSI/AISC 342 for topped and untopped steel deck diaphragms, or ASCE/SEI 41 for elements of other materials shall be used in lieu of specified values.

EXCEPTIONS:

1. Noncritical force-controlled actions that are modeled, including consideration of strength loss effects, need not satisfy Equations (16.4-1) or (16.4-2).
2. Force-controlled actions limited by formation of a yield mechanism, other than shear in structural walls, need only satisfy Equations (16.4-3) and (16.4-4):

$$(1.2+0.12S_{MS})D+0.5L+0.2S+E_{mc} \leq \phi BR_n \quad (16.4-3)$$

$$(0.9-0.12S_{MS})D+E_{mc} \leq \phi BR_n \quad (16.4-4)$$

Where E_{mc} is the capacity-limited earthquake effect associated with developing the plastic capacity of yielding components, determined in accordance with the applicable material standard, or alternatively, determined by rational analysis considering expected material properties, including strain-hardening effects where applicable.

3. Where response to vertical earthquake shaking is directly included in the analysis, the first term in equations 16.4-1 through 16.4-4 can be taken as 1.2D (16.4-1 and 16.4-3) or 0.9D (16.4-2 and 16.4-4).

16.4.2.2 DEFORMATION-CONTROLLED ACTIONS

The valid range of modeling for deformation-controlled element actions shall be established in the applicable material design standard. Where the material design standard does not specify the valid range of modeling, this parameter shall be established as the maximum value of the parameter at which the element model is capable of replicating the hysteretic behavior and load-carrying capability observed in laboratory testing of similar elements. Where suitable test data is not available, the valid range of modeling for deformation-controlled actions shall be as established in ACI 318 for reinforced concrete elements, ANSI/AISC 341 for structural steel elements, ANSI/AISC 342 for topped and untopped steel deck diaphragms, or the maximum deformation parameter as specified by ASCE/SEI 41 for elements of other materials. It shall be permitted to extend the valid range of modeling for an element beyond these deformations if the element's strength and stiffness are degraded to negligible values once these deformations are reached. Furthermore, where the range of modeling has been extended, it shall be verified by experimental data that the element's ability to support gravity loads is maintained, or the mathematical model of the structure shall simultaneously adjust to the component's loss of gravity support at that deformation.

For structures assigned to Risk Category IV, the mean of the maximum deformation parameters for deformation-controlled actions shall not exceed the limits specified for the Immediate Occupancy performance level defined in ASCE 41 when subjected to a suite of Design Earthquake ground motion records, as prescribed in Section 16.3.3.

16.4.2.3 Elements of the Gravity Force-Resisting System Elements that are not part of the seismic force-resisting system shall be demonstrated to be capable of supporting gravity loads using the mean building displacements from the suite of nonlinear response history analyses.

16.5 DESIGN REVIEW

An independent structural design review shall be performed in accordance with the requirements of this section. Upon completion of the review, the reviewer(s) shall provide the Authority Having Jurisdiction and the registered design professional with a letter attesting to:

1. Scope of review performed,
2. Whether the reviewer(s) concur with the analysis and its applicability to the design,
3. Conformance of the design to applicable requirements of the standard, and
4. Any items relating to the design or analysis that require further resolution by the Authority Having Jurisdiction.

16.5.1 Reviewer Qualifications Reviewer(s) shall consist of one or more individuals acceptable to the Authority Having Jurisdiction and possessing knowledge of the following items:

1. The requirements of this standard and the standards referenced herein, as they pertain to the design of the type of structure under consideration.
2. Selection and scaling of ground motions for use in nonlinear response history analysis.
3. Analytical structural modeling for use in nonlinear response history analysis, including use of laboratory tests in the creation and calibration of the structural analysis models, and including knowledge of soil-structure interaction if used in the analysis or the treatment of ground motions.
4. Behavior of structural systems, of the type under consideration, when subjected to earthquake loading.

At least one reviewer shall be a registered design professional.

16.5.2 Review Scope The scope of review shall include the items identified in Section 16.1.4, as well as the associated project documentation that demonstrates conformance to the design criteria.

16.6 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

See Chapter 23 for the list of consensus standards and other documents that shall be considered part of this standard to the extent referenced in this chapter.

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CHAPTER 19, SOIL–STRUCTURE INTERACTION FOR SEISMIC DESIGN (Modifications)

Revise Sections 19.2 and 19.4 as follows:

19.2 SSI ADJUSTED STRUCTURAL DEMANDS

19.2.1 Equivalent Lateral Force Procedure The inclusion of kinematic interaction effects, in accordance with Section 19.4 or other methods, is not permitted with the equivalent lateral force procedure. To account for the effects of SSI using a linear static procedure, the base shear, V , determined from Equation (12.8-1) is permitted to be modified as follows:

$$\tilde{V} = V - \Delta V \quad (19.2-1)$$

$$\Delta V = \left(C_s - \frac{\tilde{C}_s}{B_{\text{SSI}}} \right) \bar{W} \leq 0.3 V \quad (19.2-2)$$

$$B_{\text{SSI}} = 4 / [5.6 - \ln(100\beta_0)] \leq \begin{cases} 1.4 & \text{for } R \leq 3 \\ 1.7 - \frac{R}{10} & \text{for } 3 < R < 6 \\ 1.1 & \text{for } R \geq 6 \end{cases} \quad (19.2-3)$$

where

$\tilde{V}$ = Base shear adjusted for SSI;

V = Fixed-base structure base shear computed in accordance with Section 12.8.1;

R = Response modification factor in Table 12.2-1;

C_s = Seismic response coefficient determined in accordance with Section 12.8.1.1, assuming a fixed structural base at the foundation–soil interface;

$\tilde{C}_s$ = Seismic response coefficient determined in accordance with Section 12.8.1.1, assuming flexibility of the structural base at the foundation–soil interface in accordance with Section 19.1.1,

using $\tilde{T}$ as the fundamental period of the structure in lieu of the fundamental period of the structure, T , as determined by Section 12.8.2;

$\bar{W}$ = Weight caused by the effective modal mass in the fundamental mode, alternatively taken as the effective seismic weight in accordance with Section 12.7.2; and

β_0 = Effective viscous damping ratio of the soil–structure system, in accordance with Section 19.3.2.

19.2.2 Linear Dynamic Analysis The inclusion of kinematic interaction effects, in accordance with Section 19.4 or other methods, is not permitted with the linear dynamic procedure. To account for the effects of SSI, a linear dynamic analysis is permitted to be performed in accordance with Section

12.9, using either the SSI modified design response spectrum and MCE_R response spectrum in accordance with Sections 11.4.6 and 11.4.7 or SSI modified site-specific response spectrum, per Section 19.2.2.1 or an SSI modified site-specific response spectrum in accordance with Section

19.2.2.2 for spectral response acceleration, $\tilde{S}_a$, versus structural period, T . The resulting response spectral acceleration shall be divided by R/I_e , where I_e is prescribed in Section 11.5.1. The mathematical model used for the linear dynamic analysis shall include flexibility of the foundation and underlying soil in accordance with Section 19.1.1.

Scaling of the lateral forces from the modal response analysis shall be in accordance with Section 12.9.1.4 with calculated base shear, V , replaced with SSI adjusted base shear, $\tilde{V}$, determined in accordance with Equation (19.2-1) and the modal base shear, V_t , replaced by the modal base shear calculated with the effects of SSI, $\tilde{V}_t$.

The modal base shear calculated with the effects of SSI, $\tilde{V}_t$, shall not be less than $0.7V$.

19.2.2.1 SSI Modified General Design Response Spectrum The general design response spectrum, which includes the effects of SSI to be used with the modal analysis procedure in Section 19.2.2, shall be developed as follows:

$$\tilde{S}_a = \left[\left(\frac{5}{B_{SSI}} - 2 \right) \times \frac{T}{T_s} + 0.4 \right] \times S_{DS} \quad (19.2-4)$$

For $0 < T < T_0$, and

$$\tilde{S}_a = S_{DS} / B_{SSI} \text{ for } T_0 \leq T \leq T_s, \text{ and}$$

$$\tilde{S}_a = S_{D1} / (B_{SSI} T), \text{ for } T_s < T \leq T_L, \text{ and}$$

$$\tilde{S}_a = S_{D1} T_L / (B_{SSI} T^2), \text{ for } T > T_L$$

where S_{DS} and S_{D1} are defined in Section 11.4.5; T_s , T_0 , and T_L are as defined in Section 11.4.6; T is the period at the response spectrum ordinate; and B_{SSI} is defined in Equation (19.2-3).

19.2.2.2 SSI Site-Specific Response Spectrum A site-specific response spectrum that incorporates modifications caused by SSI can be developed in accordance with the requirements of Chapter 21.

The spectrum is permitted to be adjusted for the effective viscous damping ratio of β_0 , of the soil-structure system, as defined in Section 19.3.2, in the development of the site-specific spectrum.

19.2.3 Nonlinear Response History Procedure It is permitted to account for the effects of SSI using a nonlinear response history analysis performed in accordance with Chapter 16 using acceleration histories scaled to a site-specific response spectrum modified for kinematic interaction, in accordance with Section 19.4 or other approved methods. The mathematical model used for the analysis shall include foundation and soil flexibility, per Section 19.1.1, and shall explicitly incorporate the effects of foundation damping, per Section 19.3 or by other approved methods. Kinematic interaction effects, per Section 19.4, are permitted to be included in the determination of the site-specific response spectrum.

Where kinematic effects are incorporated, the site-specific response spectrum shall be developed, per the requirements of Chapter 21, with the following additional requirements:

1. The spectrum is permitted to be adjusted for kinematic SSI effects by multiplying the spectral acceleration ordinate at each period by the corresponding response spectrum ratios for either base slab averaging or embedment, or both base slab averaging and embedment ($RRS_{bsa} \times RRS_e$) per Section 19.4. Alternatively, if the unadjusted response spectrum is developed using the site-specific ground motion procedures of Chapter 21 with a minimum of 11 pairs of horizontal-component ground motion time histories, then $RRS_{bsa} \times RRS_e$ shall be permitted to be computed on the basis of 3D nonlinear, dynamic analysis of the soil and below-grade structure considering the same set of ground motions used to develop the unadjusted spectrum. For this alternative case, $RRS_{bsa} \times RRS_e$ shall be taken as the ratio of the arithmetic average response spectrum at the base of the structure as calculated from a series of analyses including the stiffness of the below-grade structure and the arithmetic

average response spectrum at the surface level as calculated from a series of analyses that exclude the below-grade structure. The same analysis methodology, platform, and soil properties shall be used for all analyses.

2. For structures embedded in the ground, the site-specific response spectrum is permitted to be developed at the depth of the embedded base level in lieu of at grade. For this case, the response spectrum ratio for embedment effects (RRS_e) shall be taken as 1.0.
3. The site-specific response spectrum modified for kinematic interaction shall not be taken as less than 80% of S_a as determined from a site-specific response spectrum in accordance with Section 21.3.
4. The site-specific response spectrum modified for kinematic interaction shall not be taken as less than 70% of S_a , as determined from the design response spectrum and MCE_R response spectrum in accordance with Sections 11.4.6 and 11.4.7. This limit shall not apply if the site-specific MCE_R was derived using the peer-reviewed site-specific site response procedures defined in Section 21.1 and $RRS_{bsa} \times RRS_e$ is computed on the basis of peer-reviewed, 3D nonlinear, dynamic analysis of the soil and below-grade structure.

If the acceleration histories are scaled to a site-specific spectrum modified for kinematic soil–structure interaction, it is permitted to include kinematic soil–structure interaction in the equivalent lateral force procedure or the linear dynamic procedure conducted per Section 16.1.2 using the site-specific response spectrum modified for kinematic soil–structure interaction, subject to the limitations herein. Where foundation damping is included in the nonlinear model, the equivalent lateral force procedure or the linear dynamic procedure, in conjunction with Section 16.1.2, shall be based on the provisions of 19.2.1 or 19.2.2, respectively.

19.4 BASE SLAB AVERAGING AND EMBEDMENT (KINEMATIC) SSI EFFECTS

Kinematic SSI effects are permitted to be represented by response spectral modification factors RRS_{bsa} for base slab averaging and RRS_e for embedment, which are multiplied by the spectral acceleration ordinates of the response spectrum at each period. The modification factors are calculated in accordance with Sections 19.4.1 and 19.4.2. Modifications of the response spectrum for kinematic SSI effects are permitted only for use with the nonlinear response history analysis provisions of Chapter 16, using the site-specific response spectrum developed in accordance with Chapter 21 and subject to the limitations in Sections 19.2.3, 19.4.1, and 19.4.2.

The equations in Sections 19.4.1 and 19.4.2 shall be permitted to be used for structures that are located on sites with $2100 \text{ ft/s} > \bar{v}_s > 700 \text{ ft/s}$ as defined in Section 20.4.1 and that are not Site Class E on the basis of the soft clay layer thickness criteria given in Section 20.2.2.

19.4.1 Base Slab Averaging Modifications for base slab averaging using the procedures of this section are permitted for structures on permitted sites that have structural mats or foundation elements interconnected with concrete slabs or that are continuously connected with grade beams or other foundation elements of sufficient lateral stiffness, so as not to be characterized as flexible under the requirements of Section 12.3.1.3.

The *modification* factor for base slab averaging, RRS_{bsa} , shall be permitted to be determined using Equation (19.4-1) for each period required for analysis.

$$RRS_{bsa} = 0.25 + 0.75 \times \left\{ \frac{1}{b_0^2} [1 - (\exp(-2b_0^2)) \times B_{bsa}] \right\}^{1/2} \quad (19.4-1)$$

where

$$B_{bsa} = \begin{cases} 1 + b_0^2 + b_0^4 + \frac{b_0^6}{2} + \frac{b_0^8}{4} + \frac{b_0^{10}}{12} & b_0 \leq 1 \\ [\exp(2b_0^2)] \times \left[\frac{1}{\sqrt{\pi}b_0} \left(1 - \frac{1}{16b_0^2} \right) \right] & b_0 > 1 \end{cases} \quad (19.4-2)$$

$$b_0 = 0.00071 \times \left(\frac{b_e}{T} \right) \quad (19.4-3)$$

where b_e is the effective foundation size (ft),

$$b_0 = 0.0023 \times \left(\frac{b_e}{T} \right) \quad (19.4-3.SI)$$

where b_e is the effective foundation size (m),

$$b_e = \sqrt{A_{base}} \leq 260 \text{ ft} \quad (19.4-4)$$

$$b_e = \sqrt{A_{base}} \leq 80 \text{ m} \quad (19.4-4.SI)$$

T is the response spectra ordinate period, which shall not be taken as less than 0.20 s when used in Equations (19.4-3) or (19.4-3.SI); and A_{base} is the area of the base of the structure [ft^2 (m^2)].

19.4.2 Embedment The response spectrum shall be developed based on a site-specific study at the depth of the base of the structure. Alternatively, modifications for embedment are permitted using the procedures of this section.

The modification factor for embedment, RRS_e , shall be determined using Equation (19.4-5) for each period required for analysis.

$$RRS_e = 0.25 + 0.75 \times \cos\left(\frac{2\pi e}{Tv_{s,e}}\right) \quad (19.4-5)$$

where

e = Foundation embedment depth [ft (m)], not greater than 20 ft (6.1 m) and not greater than the bottom of the base slab diaphragm; a minimum of 75% of the foundation footprint shall be present at the embedment depth, and the foundation embedment for structures located on sloping sites shall be the shallowest embedment;

$v_{s,e}$ = The average effective shear wave velocity for site soil conditions used in determining embedment effects, taken as average value of velocity over the embedment depth of the foundation determined using $v_{so,e}$ and Table 19.3-1 or a site-specific study and shall not be taken as less than 650 ft/s (200 m/s);

$v_{so,e}$ = The average low strain shear wave velocity used in determining embedment effects computed using Equation (20.4-1) over the embedment depth of the foundation; and

T = Response spectra ordinate period, which shall not be taken as less than 0.20 s when used in Equation (19.4-5).

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CHAPTER 21, SITE-SPECIFIC GROUND MOTION PROCEDURES FOR SEISMIC DESIGN (Modifications)

Replace Section 21.1 with the following:

21.1 SITE RESPONSE ANALYSIS

Section 21.1 requirements shall be satisfied where site response analysis is performed or required by Section 11.4.7. A site response analysis involves the application of either the *hybrid ground motion* approach or the *site-specific site response* approach as defined herein. The analysis shall be documented in a project report.

Site response evaluated relative to a reference condition defined in Section 21.1.1.1 using ground motions, site dynamic properties, and an analytical wave propagation model (hereafter, *ground response analysis*) as defined in Section 21.1.3.1 shall be considered *site-specific site response*. Site amplifications derived using site-specific site response procedures are permitted to be combined with reference site hazards according to the requirements in Section 21.1.5.1 to preserve target probability and risk levels in surface ground motions. When site-specific site response is applied, an independent peer review of the site data, site amplification model, and hazard results shall be conducted in accordance with Section 21.1.6.

Alternatively, site amplification can be evaluated relative to the reference condition defined in Section 21.1.1.2 using procedures in Section 21.1.3.2. When those amplifications are enveloped and then multiplied by reference site MCE_R response spectral ordinates as described in Section 21.1.5.2 to compute surface ground-motion MCE_R response spectrum, the resulting MCE_R response spectrum shall be denoted *hybrid ground motions*.

The following definitions of terms and symbols apply to the provisions of Section 21.1 in addition to the definitions presented in Chapter 11:

Aleatory Variability: Variability of ground motion (*IMs*) or site amplification, as observed from recordings or predicted from a ground-motion model. Aleatory variability is represented by a standard deviation parameter, $\sigma_{\ln IM}$, which has two components (between-event variability, $\tau_{\ln IM}$, and within-event variability, $\phi_{\ln IM}$).

Between-Event Variability: Aleatory variability of ground-motion *IMs* between earthquakes, expressed as standard deviation $\tau_{\ln IM}$. Generally obtained from ground-motion models.

Ground Response Analysis: An analysis performed using an analytical model of the site that considers seismic velocities, nonlinear soil responses, and material damping, and which calculates the effects on ground motions (time series or response spectra) of wave propagation through the model. Ground response analyses are permitted to use input motions represented by ground motion time series or a ground motion response spectrum or Fourier spectrum. Ground response analyses that use a spectrum-based input utilize Random Vibration Theory.

Hybrid Ground Motions: Ground motions derived from site response analysis procedures that consider site-specific site conditions as described in Section 21.1.2, that evaluate site amplification as described in Section 21.1.3.2, and that integrate the site response with hazard results as described in 21.1.5.2. Analyses that meet these conditions use full within-event variability provided in suitable ground-motion models.

Intensity Measure: Parameter that measures a ground motion feature, denoted (IM), such as peak ground acceleration, peak ground velocity, or response spectral accelerations at a given period.

Reference horizon: Location within a site profile selected to be the reference site condition. The horizon is characterized by its depth below the ground surface, $Z_{h,ref}$, and the time-averaged shear-wave velocity from $Z_{h,ref}$ to $Z_{h,ref}+30$ m, $V_{S30,ref}$.

Single-Station Within-Event Variability: Aleatory variability of ground-motion IM s within earthquakes for an individual site, expressed as standard deviation ϕ_{SS} .

Site Period: Fundamental mode period of a site as derived from a shear-wave velocity profile that reaches reference rock conditions or from geophysical testing.

Site-to-Site Variability: Variability of site amplification between sites when site response is estimated using an ergodic model. Expressed using standard deviation ϕ_{S2S} .

Single-Station Site Response Variability: Aleatory variability of site amplification for ground-motion IM s between earthquakes, expressed as standard deviation ϕ_{InY} .

Site-Specific Site Response: Site response analysis procedures that consider site-specific site conditions, as described in Section 21.1.2, that evaluate site amplification as described in Section 21.1.3.1, and that integrate the site response with hazard results as described in Section 21.1.5.1. Analyses that meet these conditions can use single-station within-event variability.

Within-Event Variability: Aleatory variability of ground-motion IM s within earthquakes, expressed as standard deviation σ_{InIM} . Generally obtained from ground-motion models.

21.1.1 Reference Site Motion

21.1.1.1 APPLICATION TO SITE-SPECIFIC SITE RESPONSE MODELING APPROACH

The reference site condition shall be defined as one of the following:

1. A V_{S30} value, defined in Section 20.4.1, for a firm site condition (site classes BC, B or A as given in Table 20.2-1).
2. A shear-wave velocity (V_s) profile for a firm site condition (site classes BC, B or A as given in Table 20.2-1).
3. A specific horizon within the site profile at which either bedrock, or where bedrock is sufficiently deep that the fundamental-mode site period exceeds the period range of interest for the project, firm soil conditions are encountered (equivalent to Class C or stiffer as given in Table 20.2-1).

The definition of the reference site condition, and the basis for it, shall be documented in the project report.

Ground motions for the reference site condition shall be derived in a form that is compatible with the approach used for combining site response with reference site ground motions as given in Section 21.1.5.1.

When a site-specific site response procedure is applied using ground response analyses, at least one probabilistic-based response spectrum corresponding to the MCE_R level as provided in Sections 11.4.3 or 21.2 shall be provided for the reference site condition. As needed for the analytical model, the development of the target spectrum and the selection and modification of ground motion time series are permitted to be developed using either Method 1 or Method 2 as specified in Sections 16.2.1.1 and 16.2.1.2, respectively (for target spectra) and Section 16.2.2 and 16.2.3 (for time series selection and modification), with two exceptions (1) a minimum of 7 input time series are required and (2) the period range for scaling or matching, as provided in Section 16.2.3.1, shall be based on the period of the soil column. It is permitted for Method 1 spectra to be defined based on uniform hazard instead of uniform risk for Section 21.1 applications when ground motion levels distinct from MCE_R are considered in addition to the MCE_R spectrum.

Target ground-motion response spectra shall be developed based on median component ground motions.

21.1.1.2 APPLICATION WITH HYBRID GROUND-MOTION APPROACH

The reference site condition shall be defined as a specific horizon within the site profile at which either bedrock, or where bedrock is very deep, firm soil conditions, are encountered (equivalent to Class C or stiffer as given in Table 20.2-1). A V_{S30} value for the reference condition, $V_{S30,ref}$, shall be

computed as given in Section 20.4.1 but with the depth interval taken as the depth to the reference horizon ($Z_{h,ref}$) to $Z_{h,ref} + 100$ ft (30 m).

The following two target response spectra shall be defined using $V_{S30,ref}$: (1) an MCE_R response spectrum as provided in Sections 11.4.3 or 21.2 and (2) a reduced response spectrum corresponding to $\frac{2}{3}$ of the MCE_R response spectrum. As needed for the ground response analysis program being used, a minimum of 7 single-component ground-motion time series shall be selected and modified to be compatible with each of the two target spectra using either Method 1 or Method 2 as specified in Section 16.2.2 as modified in Section 2.1.1.1. It is permissible for the same time series, with different scale factors, to be used with both target spectra.

21.1.2 Site Properties Model

A site properties model shall be developed that contains the soil properties for site response analysis. The site properties model shall include shear wave velocity as a function of depth (i.e., V_s profiles), modulus reduction and damping versus shear strain (MRD) relationships for each soil layer, and unit weights for each soil layer.

V_s profiles shall be developed from field measurements using reliable techniques at the site or from measurements in similar soils in the site vicinity. Methods based on a single-direction linear array of surface instruments without a controlled vibration source shall not be used for the derivation of V_s profiles. The site V_s profile shall extend to a bedrock or firm soil horizon as defined in Section 21.1.1. MRD relationships and unit weights shall be selected based on laboratory tests or published relationships.

Where site response analyses indicate that large-strain soil responses (i.e., responses that exceed the calibration range for the MRD relations) may occur in particular layers within the site profile, MRD relationships shall be developed in consideration of the shear strength of those layers. For soil layers consisting of liquefiable soils or highly sensitive clays (Site Class F, Section 20.2.1), the nonlinear constitutive model used for those layers shall consider the effects of potential strength loss and phase transformation behavior.

The uncertainties in V_s shall be estimated for all soil layers. The uncertainties in MRD shall be estimated for soil layers that produce strongly nonlinear site responses in accordance with Section 21.1.4. The manner by which uncertainties are estimated shall be provided in the project report.

21.1.3 Site Response Modeling

Site response modeling shall be performed using either a site-specific site response approach in Section 21.1.3.1 or the hybrid ground-motion approach in Section 21.1.3.2.

21.1.3.1 SITE-SPECIFIC SITE RESPONSE APPROACH

The site amplification model shall provide the median site amplification, which shall be represented by an equation that accounts for both the linear amplification for weak input motion levels and

nonlinear effects for stronger input motions. Uncertainties in soil properties shall be estimated and considered in the development of coefficients for the median site amplification equation. Separate equations shall be developed for each period of interest.

The linear component of the median site amplification model is permitted to be derived from ground motions recorded at or near the site, if available, and otherwise from ground response analyses using the site properties model. If the median site amplification model is developed using local ground-motion records from one or more stations that are not located at the site of interest, the applicability of these ground motions to the site of interest shall be evaluated and documented. Nonlinear effects shall be evaluated using ground response analyses that utilize the site properties model. Material constitutive models shall be used in consideration of the potential for shear strength mobilization at large strains or pore pressure and phase transformation behavior from liquefaction or cyclic softening.

The fundamental period of the site V_s profile considered in the analysis (T_{col}) shall be derived from ground response analyses and provided in the project report. For applications in which spectral ordinates for $T > T_{col}$ are relevant to the project, the potential for site responses involving deeper portions of the site stratigraphy shall be considered in the development of the median site amplification model.

21.1.3.2 HYBRID GROUND-MOTION APPROACH

Site response modeling utilizing a hybrid ground-motion approach shall be in accordance with the requirements of this section.

At least five realizations of site responses for portions of the site profile above the reference horizon shall be derived from ground response analyses that utilize the site properties model. These realizations shall consider alternate values of soil properties given their statistical uncertainties and shall be considered for each of the input ground motions, as described in Section 21.1.1.2, corresponding to the MCE_R spectrum (Section 21.2) and a reduced spectrum with ordinates that are $\frac{2}{3}$ of the MCE_R spectrum.

The fundamental period of the site V_s profile considered in the analysis (T_{col}) shall be provided in the project report. For applications in which spectral ordinates for $T > T_{col}$ are relevant to the project, the potential for site responses involving deeper portions of the site stratigraphy shall be considered in the development of the median site amplification model.

21.1.4 Ground-Motion Variability and Uncertainty

Seismic hazard analyses shall consider the applicable levels of ground-motion variability and uncertainty for the selected analysis approach as provided in Section 21.1.4.1 for the site-specific site response approach or Section 21.1.4.2 for the hybrid ground-motion approach.

21.1.4.1 APPLICATION WITH SITE-SPECIFIC SITE RESPONSE APPROACH

The aleatory ground-motion variability model is permitted to apply single-station variability (σ_{SS}) in the evaluation of reference site and surface ground motions. Single-station site response variability (ϕ_{InY}) shall be estimated for utilization in seismic hazard analyses.

If the median site amplification model (Section 21.1.3.1) was developed using ground response analyses, site response modeling uncertainty shall be considered in addition to soil parameter uncertainties not accounted for in ϕ_{InY} . If the median site amplification model was developed using local ground-motion records from one or more stations that are not located at the site of interest, the statistical uncertainty in the mean estimate shall be considered in addition to uncertainty related to the use of recordings that are not located at the site, if applicable.

21.1.4.2 APPLICATION WITH HYBRID GROUND-MOTION APPROACH

Ground-motion variability and uncertainty shall be considered in the hybrid ground motion approach in accordance with the requirements of this section.

The aleatory ground-motion variability model shall incorporate the effects of site-to-site variability (ϕ_{S2S}) in addition to single-station variability and between-event variability.

21.1.5 Analysis of Ground-Motion Hazard Including Site Response

Ground-motion response spectra for the reference site condition shall be combined with the site response model to develop surface ground-motion response spectra using either the site-specific site response approach in Section 21.1.5.1 or using the hybrid ground-motion approach in Section 21.1.5.2.

21.1.5.1 APPLICATION WITH SITE-SPECIFIC SITE RESPONSE APPROACH

The median site amplification model from Section 21.1.3.1 shall be combined with reference site ground motions from Section 21.1.1.1 in a manner that incorporates variability and uncertainty as given in Section 21.1.4.1 to derive site hazard curves that represent the relationships between rate of exceedance and ground-motion level.

Results shall be presented as mean hazard curves for response spectral accelerations at periods of interest, peak ground accelerations, or peak ground velocities. Response spectral acceleration results shall be converted to MCE_R ground motions using procedures given in Section 21.2.1.

21.1.5.2 APPLICATION WITH HYBRID GROUND-MOTION APPROACH

Ground-motion hazard analysis shall be calculated in the hybrid ground motion approach in accordance with the requirements of this section.

For each set of reference site ground motions as specified in Section 21.1.1.2, the mean amplification ratio shall be calculated at each period based on the input ground motion and soil

property realizations considered in the analysis. The mean amplification ratios obtained from each set of reference site ground motions (MCE_R and $\frac{2}{3}MCE_R$) shall be enveloped at each period.

For each period, the envelope of the amplification ratios shall be multiplied by the reference site MCE_R ordinate to obtain the ground-surface MCE_R .

21.1.6 Peer Review

When site-specific site response is applied, an independent peer review shall be performed in accordance with the requirements of this section. Upon completion of the review, the reviewer(s) shall provide the Authority Having Jurisdiction and the registered design professional with a letter attesting to:

1. Scope of review performed,
2. Whether the reviewer(s) concur with the analysis and its applicability to the design,
3. Conformance of the results with applicable requirements of this standard, and
4. Any items relating to the results or analysis that require further resolution by the Authority Having Jurisdiction.

21.1.6.1 REVIEWER QUALIFICATIONS

Reviewer(s) shall consist of one or more individuals acceptable to the Authority Having Jurisdiction and having experience with the following items:

1. The requirements of this standard and the standards referenced herein, as they pertain to site response analysis.
2. Ground response analysis as a means by which to derive nonlinear site response, including measurement and selection of dynamic soil properties and the interpretation of results to derive site amplification models.
3. Selection and scaling of ground motions for use in nonlinear ground response analysis.
4. Application of site amplification models in probabilistic seismic hazard analyses.

At least one reviewer shall be a registered design professional.

21.1.6.2 REVIEW SCOPE

The scope of review shall include the subjects identified in Sections 21.1.1 to 21.1.5, as well as the associated project documentation that demonstrates conformance to the criteria in these Provisions for site response and probabilistic seismic hazard analysis.

Revise Section 21.2.3 as follows:

21.2.3 Site-Specific MCE_R Response Spectrum

The site-specific MCE_R spectral response acceleration at any period, S_{aM} , shall be taken as the lesser of the spectral response accelerations from the probabilistic ground motions of Section 21.2.1 and the deterministic ground motions of Section 21.2.2.

EXCEPTION: The site-specific MCE_R response spectrum may be taken as equal to the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase for the applicable site class.

The site-specific MCE_R spectral response acceleration at any period, when derived using the peer-reviewed, site-specific site response procedures defined in Section 21.1, shall not be taken as less than 70% of the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase for the applicable site class.

The site-specific MCE_R spectral response acceleration at any period, when derived using hybrid ground motion procedures as defined in Section 21.1 or site-specific ground motion hazard analysis per Section 21.2, shall not be taken as less than 80% of the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase for the applicable site class, except that for sites classified as Site Class DE or E located in the conterminous United States east of -105 degrees, the site-specific MCE_R shall not be taken as less than 80% of the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase for Site Class D.

21.2.3.1 Site Class F Sites For sites classified as Site Class F requiring site-specific analysis in accordance with Section 11.4.7, the site-specific MCE_R spectral response acceleration at any period shall not be less than 80% of the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase for Site Class E, except that for sites located in the conterminous United States east of -105 degrees, the site-specific MCE_R shall not be taken as less than 80% of the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase for Site Class D.

EXCEPTION: Where a different site class can be justified using the site-specific classification procedures of Section 20.3.3, a lower limit of 80% of S_{aM} for the justified site class shall be permitted to be used.

Revise Section 21.5 and add Section 21.5.4 as follows:

21.5 MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN PEAK GROUND ACCELERATIONS

The site-specific MCE_G peak ground acceleration, PGA_M , shall be taken as the geometric mean or median-component peak ground acceleration with a 3% probability of exceedance within a 50-year period.

The site-specific MCE_G peak ground acceleration, when derived using the peer-reviewed, site-specific site response procedures defined in Section 21.1, shall not be taken as less than 70% of the value of the MCE_G peak ground acceleration parameter PGA_M obtained from the USGS Seismic Design Geodatabase for the applicable site class.

The site-specific MCE_G peak ground acceleration, when derived using hybrid ground motion procedures as defined in Section 21.1 or site-specific ground motion hazard analysis per Section 21.2, shall not be taken as less than 80% of the value of the MCE_G peak ground acceleration parameter PGA_M obtained from the USGS Seismic Design Geodatabase for the applicable site class.

21.5.4 Site-Specific MCE_L Peak Ground Acceleration PGA_L The site-specific, liquefaction-targeted peak ground acceleration shall be taken as the geometric mean peak ground acceleration that, when used to assess liquefaction triggering with no magnitude scaling adjustment, results in a liquefaction factor of safety FS_L with a 2.5% probability of non-exceedance within a 50-year period.

For the purposes of this standard, the liquefaction-targeted PGA_L shall be determined using one of two methods:

1. Computing the liquefaction-targeted PGA_L at the applicable site class as the product of the liquefaction coefficient, C_L , and the geometric mean peak ground acceleration PGA that has a 3% probability of exceedance within a 50-year period as determined from the site-specific ground motion procedures in Section 21.5.1. The value of the liquefaction coefficient, C_L , shall be obtained from the USGS Seismic Design Geodatabase for the applicable site class. The site-specific MCE_L peak ground acceleration determined using this method shall not be taken as less than 80% of the value of the MCE_L peak ground acceleration parameter PGA_L obtained from the USGS Seismic Design Geodatabase for the applicable site class.
2. Integration of site-specific PGA hazard curve information (including the conditional percent contribution of magnitude M_w at a range of PGA hazard levels) with one or more liquefaction triggering fragility models (i.e., representing the probability of liquefaction triggering or probability of non-exceedance of liquefaction factor of safety FS_L as a function of PGA and M_w) for the reference soil condition or for soil conditions that are specific to the site of interest. The site-specific MCE_L peak ground acceleration determined using this method shall not be taken as less than 70% of the value of the MCE_L peak ground acceleration parameter PGA_L obtained from the USGS Seismic Design Geodatabase for the applicable site class.

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CHAPTER 22, SEISMIC GROUND MOTION AND LONG-PERIOD TRANSITION MAPS (Modifications)

Revise Chapter 22 as follows:

Contained in this chapter are Figures 22-1 through 22-8, which map the risk-targeted maximum considered earthquake (MCE_R) spectral response acceleration parameters, S_{MS} and S_{M1} , for default site conditions, based on the most critical spectral response acceleration of Site Classes C, CD, and D; Figures 22-9 through 22-13, which map the maximum considered earthquake geometric mean (MCE_G) peak ground acceleration parameter, PGA_M , for the default site conditions; and Figures 22-14 through 22-17, which map the long-period transition period parameter, T_L . In accordance with Section 11.4.3, S_{MS} and S_{M1} values for Site Classes A, B, BC, C, CD, D, DE, and E—as well as values of the MCE_R spectral response acceleration parameters, S_s and S_1 , (for Site Class BC)—are contained in the USGS Seismic Design Geodatabase defined in Section 11.2; values of PGA_M for all the site classes are also contained in this geodatabase, in accordance with Section 11.8.3. For the definitions of these ground motion parameters, see Section 11.3.

These maps and the USGS Seismic Design Geodatabase were prepared by the US Geological Survey in collaboration with the Building Seismic Safety Council (BSSC) Provisions Update Committee and the ASCE 7 Seismic Subcommittee and have been updated for this standard.

Maps of T_L for Guam and the Northern Mariana Islands and for American Samoa are not provided because this parameter has not been developed for those islands via the same disaggregation computations done for the other US regions. Therefore, as in previous editions of this standard, the value of T_L shall be 12 s for those islands.

The following is a list of the maps contained in this chapter:

1. Figure 22-1 S_{MS} for the Default Site Conditions for the Conterminous United States
2. Figure 22-2 S_{M1} for the Default Site Conditions for the Conterminous United States
3. Figure 22-3 S_{MS} for the Default Site Conditions for Alaska
4. Figure 22-4 S_{M1} for the Default Site Conditions for Alaska
5. Figure 22-5 S_{MS} and S_{M1} for the Default Site Conditions for Hawaii

6. Figure 22-6 S_{MS} and S_{M1} for the Default Site Conditions for Puerto Rico and the United States Virgin Islands
7. Figure 22-7 S_{MS} and S_{M1} for the Default Site Conditions for Guam and the Northern Mariana Islands
8. Figure 22-8 S_{MS} and S_{M1} for the Default Site Conditions for American Samoa
9. Figure 22-9 PGA_M for the Default Site Conditions for the conterminous United States
10. Figure 22-10 PGA_M for the Default Site Conditions for Alaska
11. Figure 22-11 PGA_M for the Default Site Conditions for Hawaii
12. Figure 22-12 PGA_M for the Default Site Conditions for Puerto Rico and the United States Virgin Islands
13. Figure 22-13 PGA_M for the Default Site Conditions for Guam and the Northern Mariana Islands and American Samoa
14. Figure 22-14 T_L for the Conterminous United States
15. Figure 22-15 T_L for Alaska
16. Figure 22-16 T_L for Hawaii
17. Figure 22-17 T_L for Puerto Rico and the United States Virgin Islands

User Note: The USGS Seismic Design Geodatabase is available at the ASCE 7 Hazard Tool <https://asce7hazardtool.online/> or an approved equivalent.

22.1 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

See Chapter 23 for the list of consensus standards and other documents that shall be considered part of this standard to the extent referenced in this chapter.

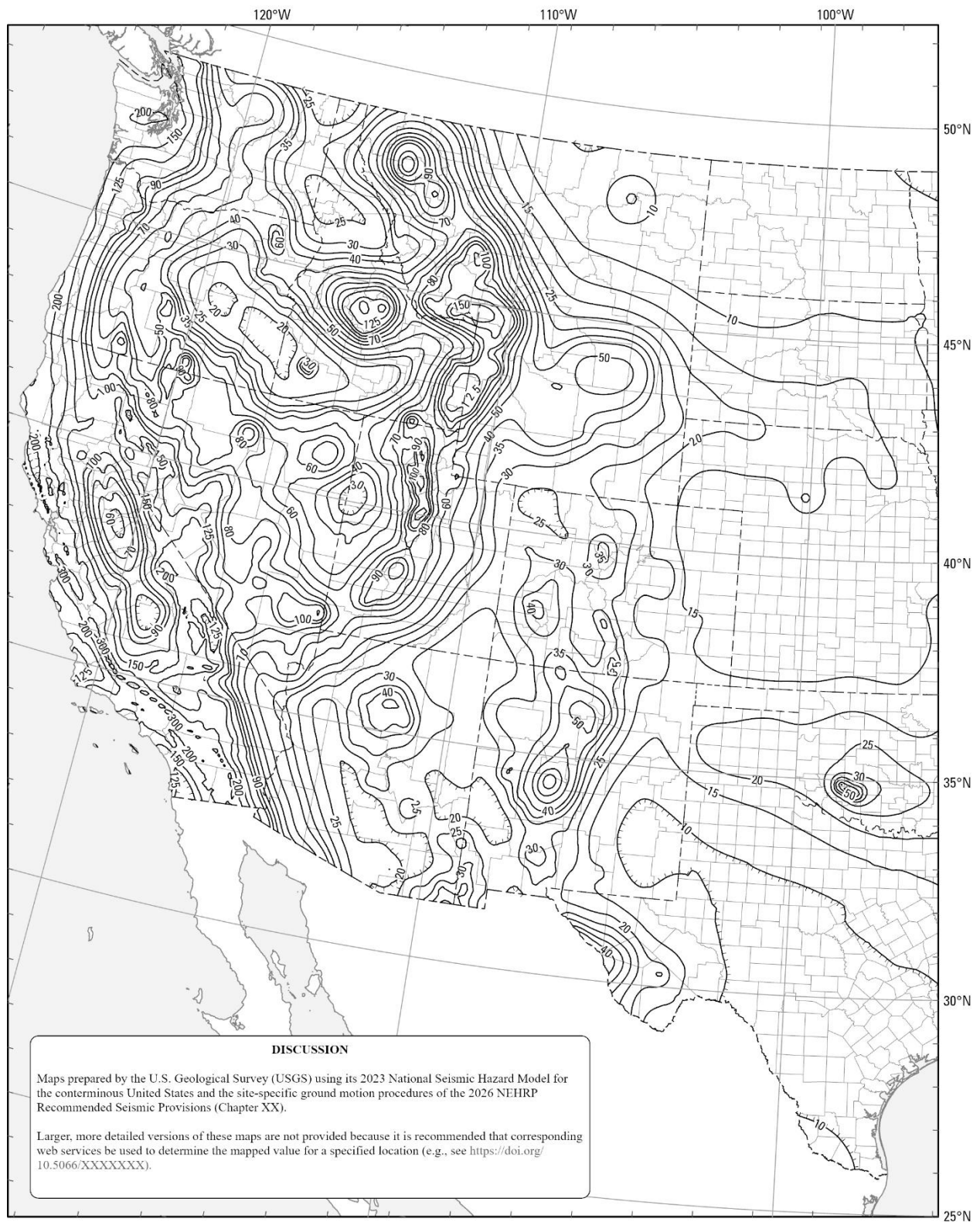


Figure 22-1 S_{MS} for the Default Site Class, for the Conterminous United States.

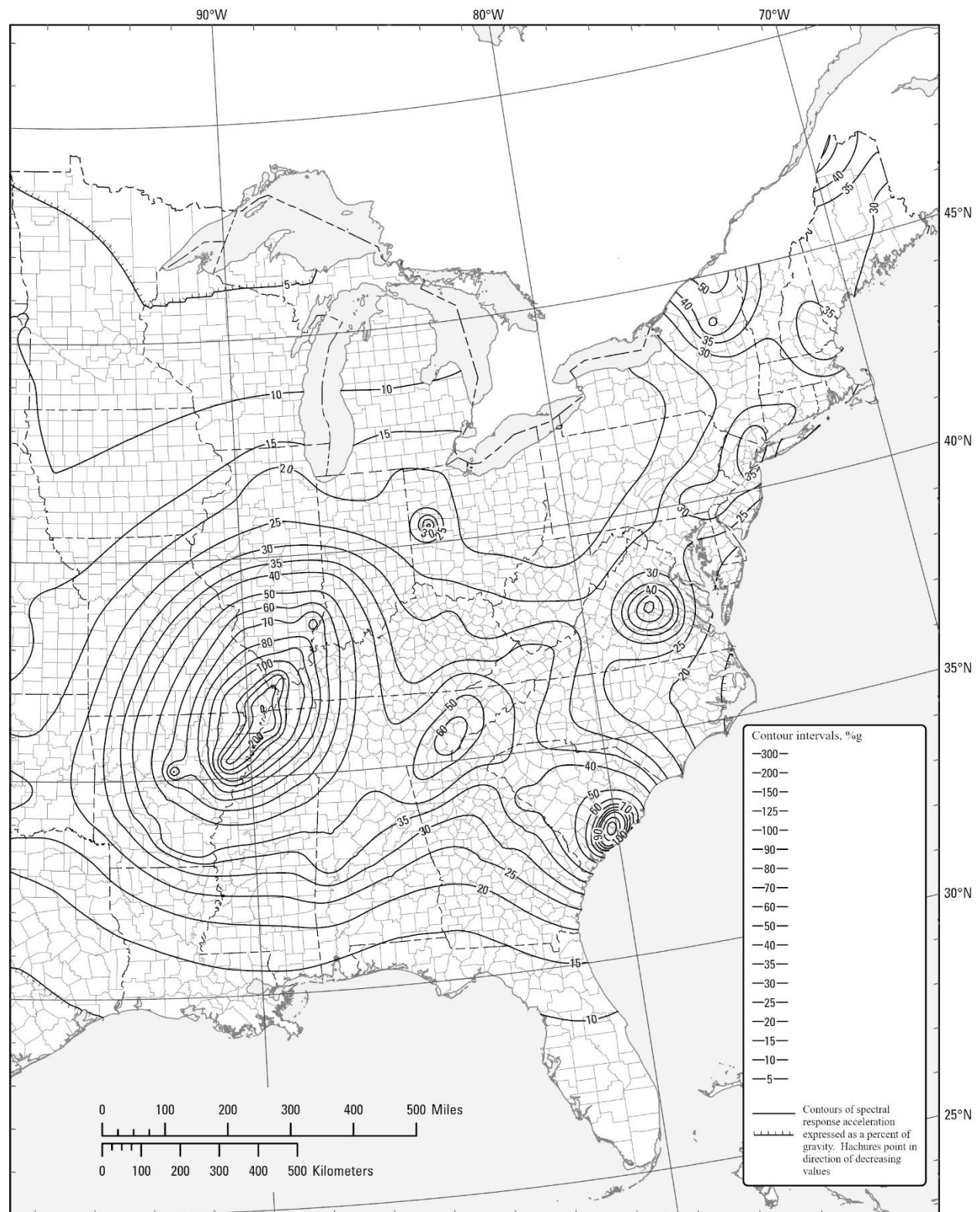


Figure 22-1 (continued) S_{MS} for the Default Site Class, for the Conterminous United States.

Figure 22-1. S_{MS} for the default site conditions for the conterminous United States.

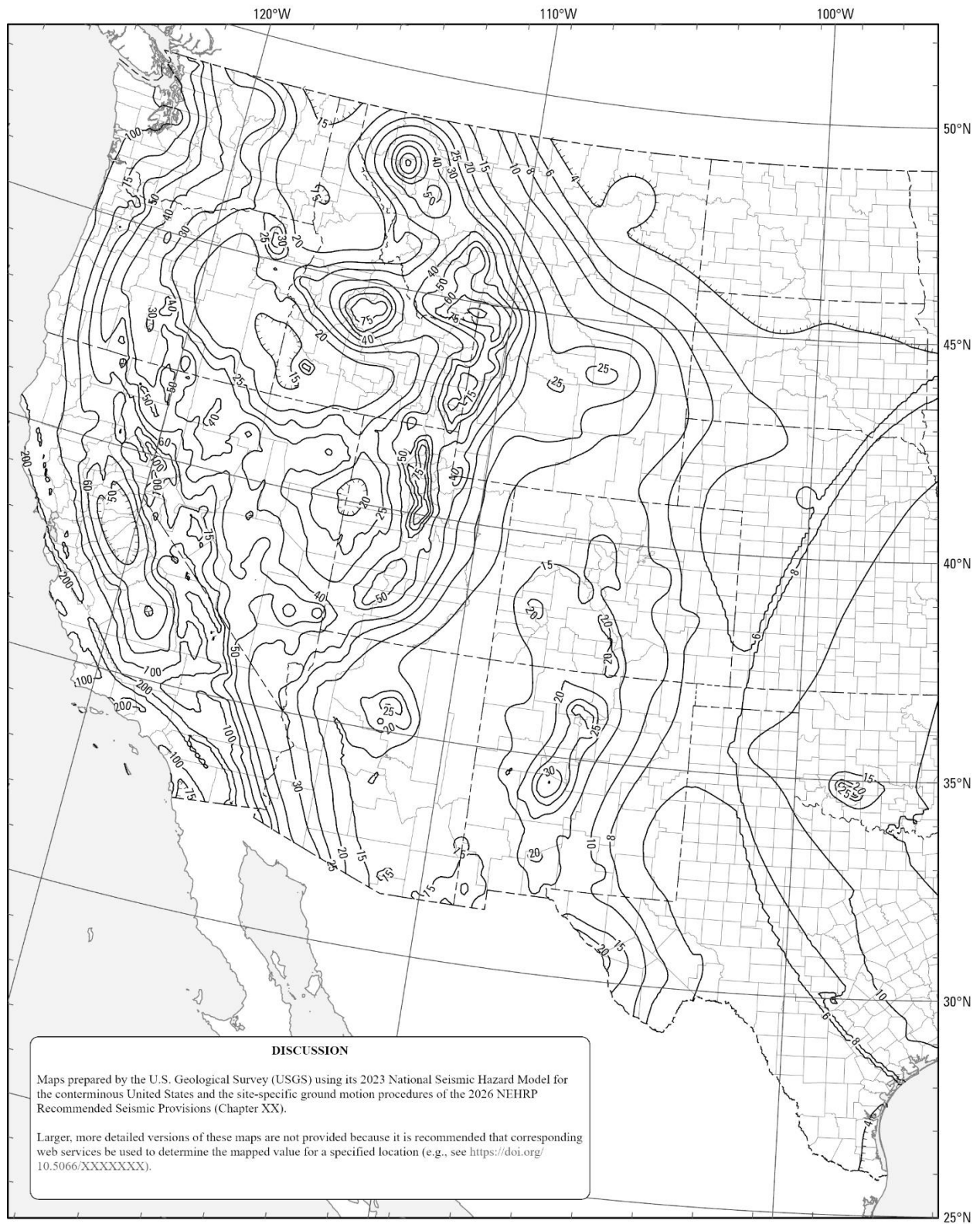


Figure 22-2 S_{MI} for the Default Site Class, for the Conterminous United States.

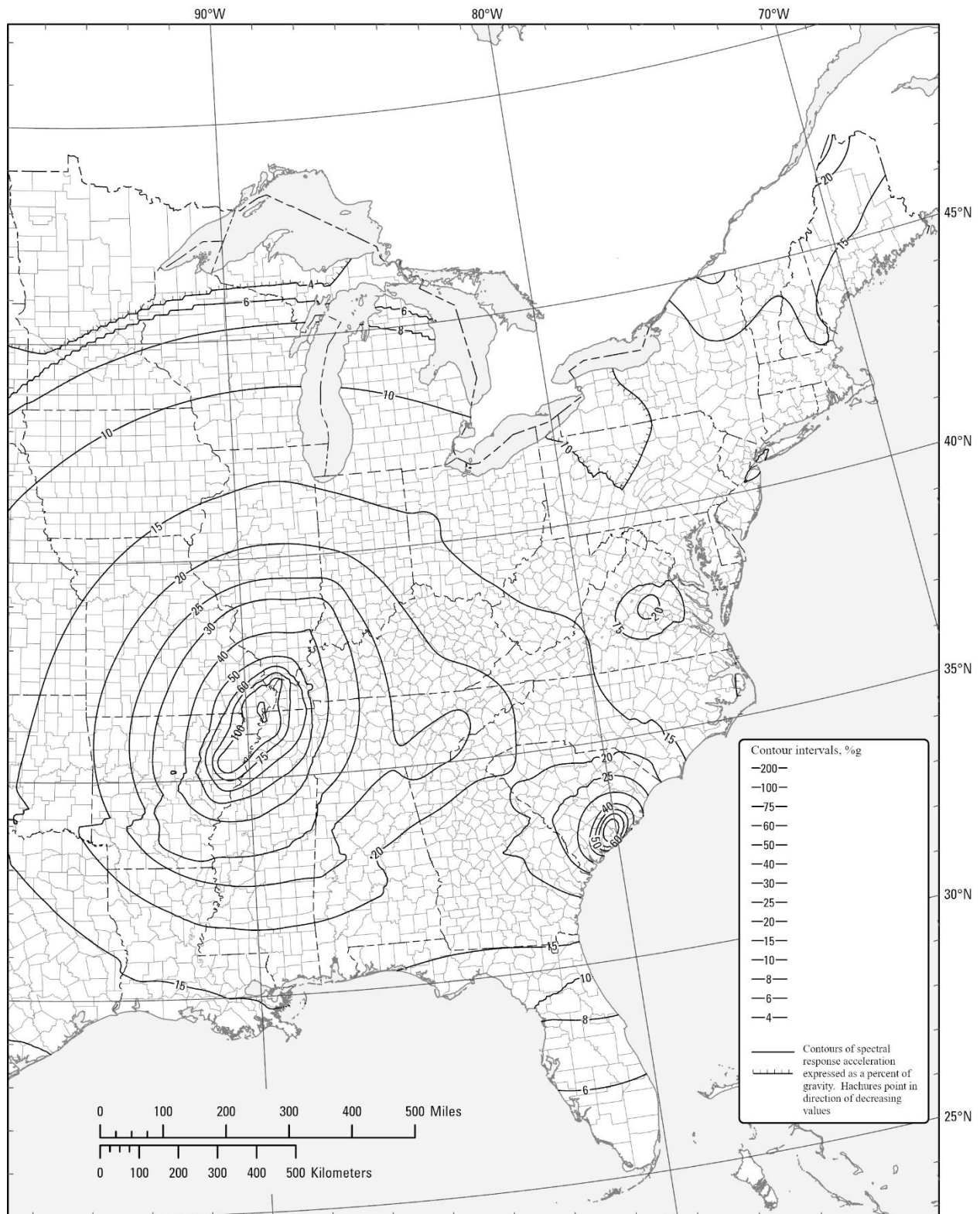


Figure 22-2 (continued) S_{M1} for the Default Site Class, for the Conterminous United States.

Figure 22-2. S_{M1} for the default site conditions for the conterminous United States.

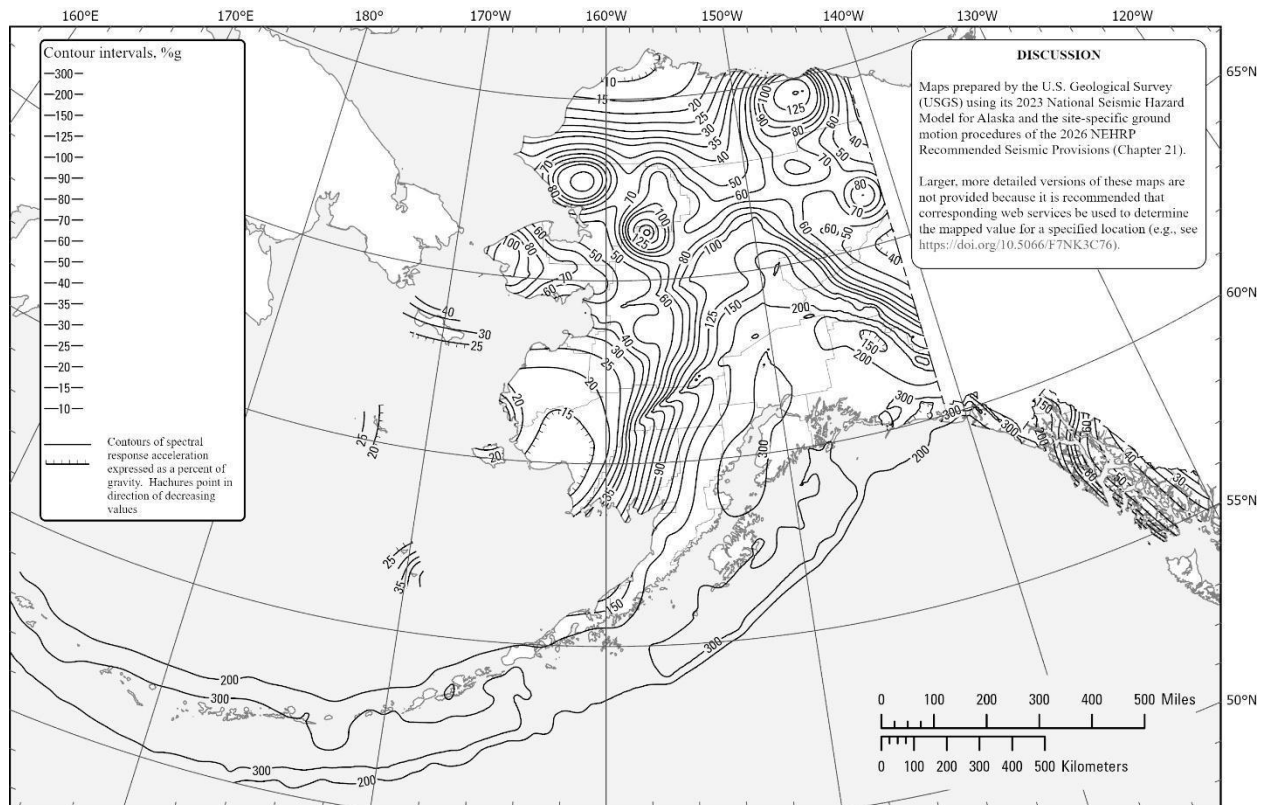


Figure 22-3. S_{Ms} for the default site conditions for Alaska.

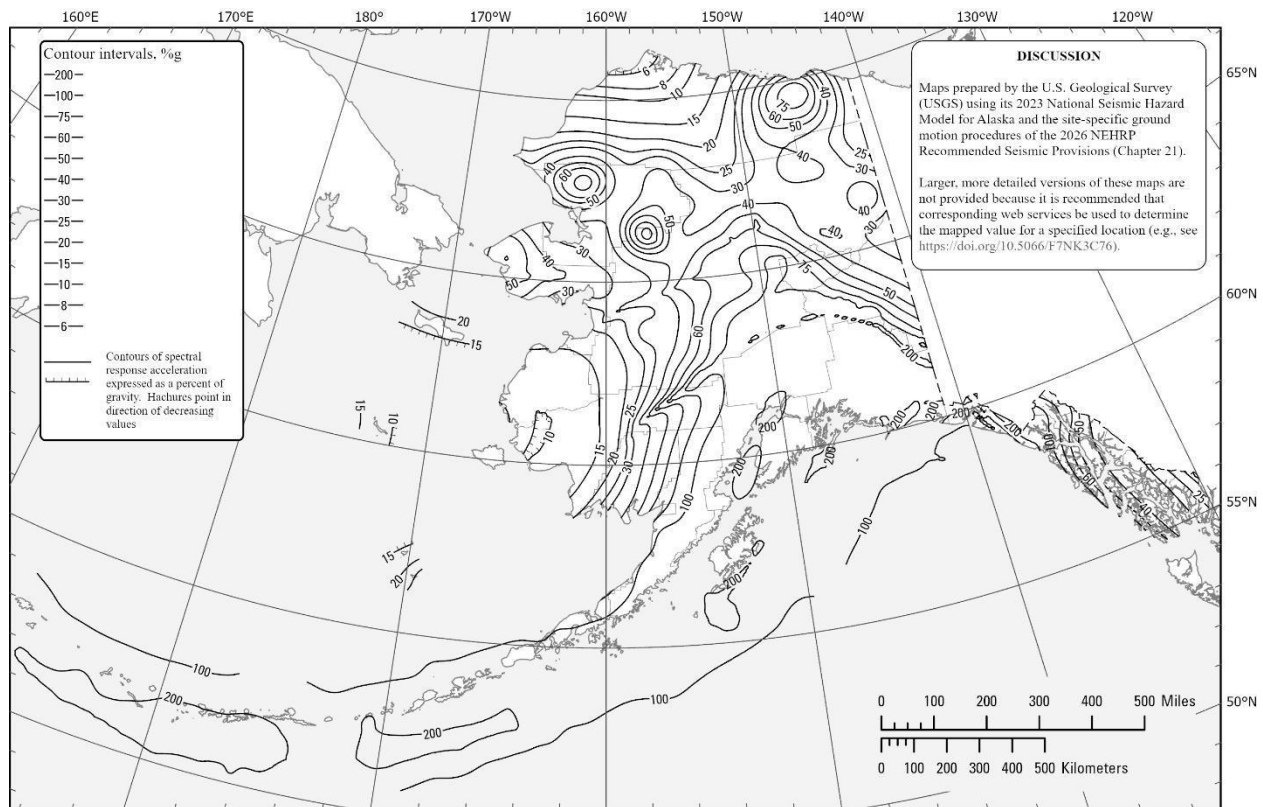
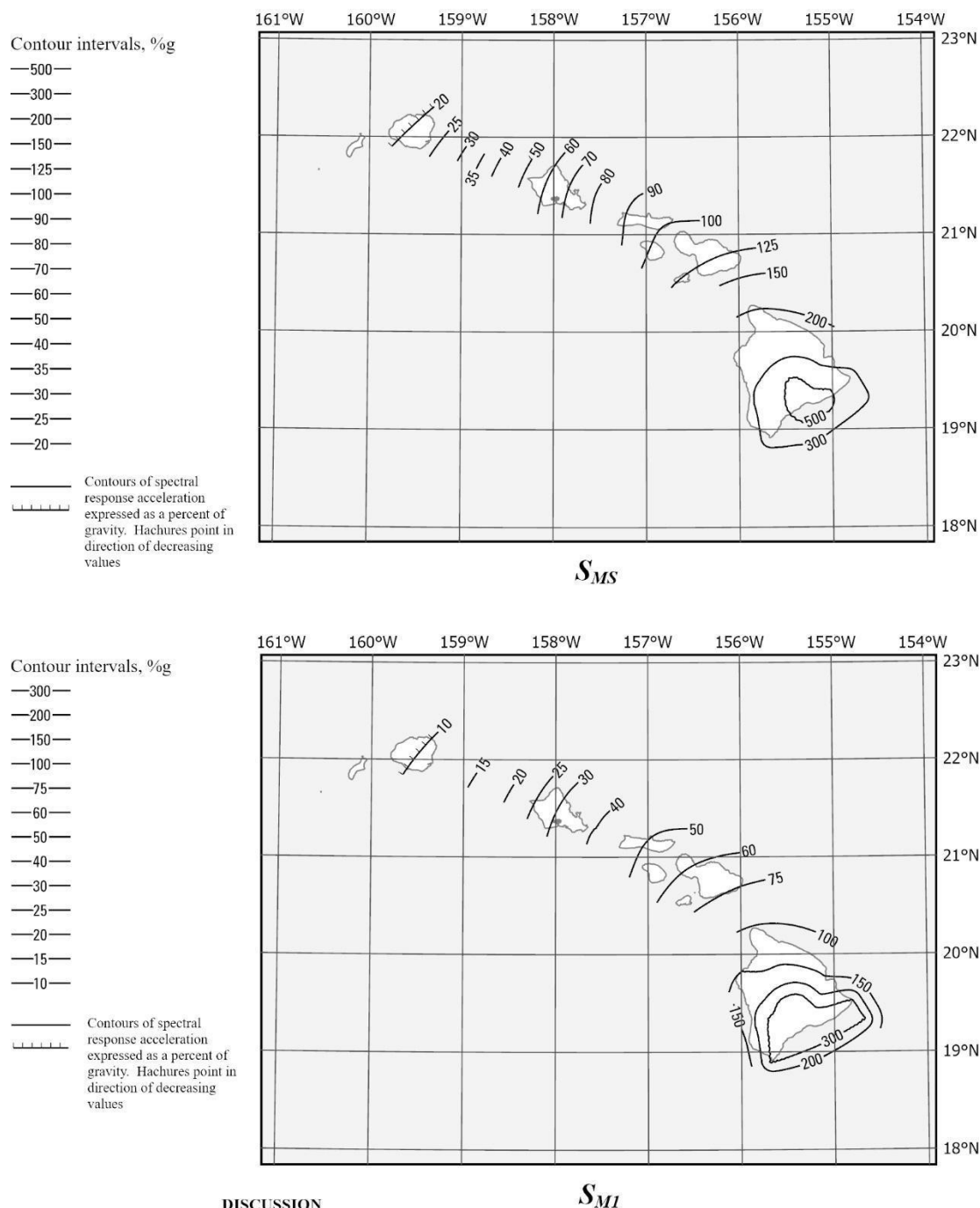


Figure 22-4. S_{M1} for the default site conditions for Alaska.



DISCUSSION

Maps prepared by the U.S. Geological Survey (USGS) using its 2021 National Seismic Hazard Model for Hawaii and the site-specific ground motion procedures of the 2026 NEHRP Recommended Seismic Provisions (Chapter 21).

Larger, more detailed versions of these maps are not provided because it is recommended that corresponding web services be used to determine the mapped value for a specified location (e.g., see <https://doi.org/10.5066/F7NK3C76>).

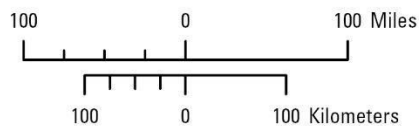


Figure 22-5. S_{MS} and S_{M1} for the default site conditions for Hawaii.

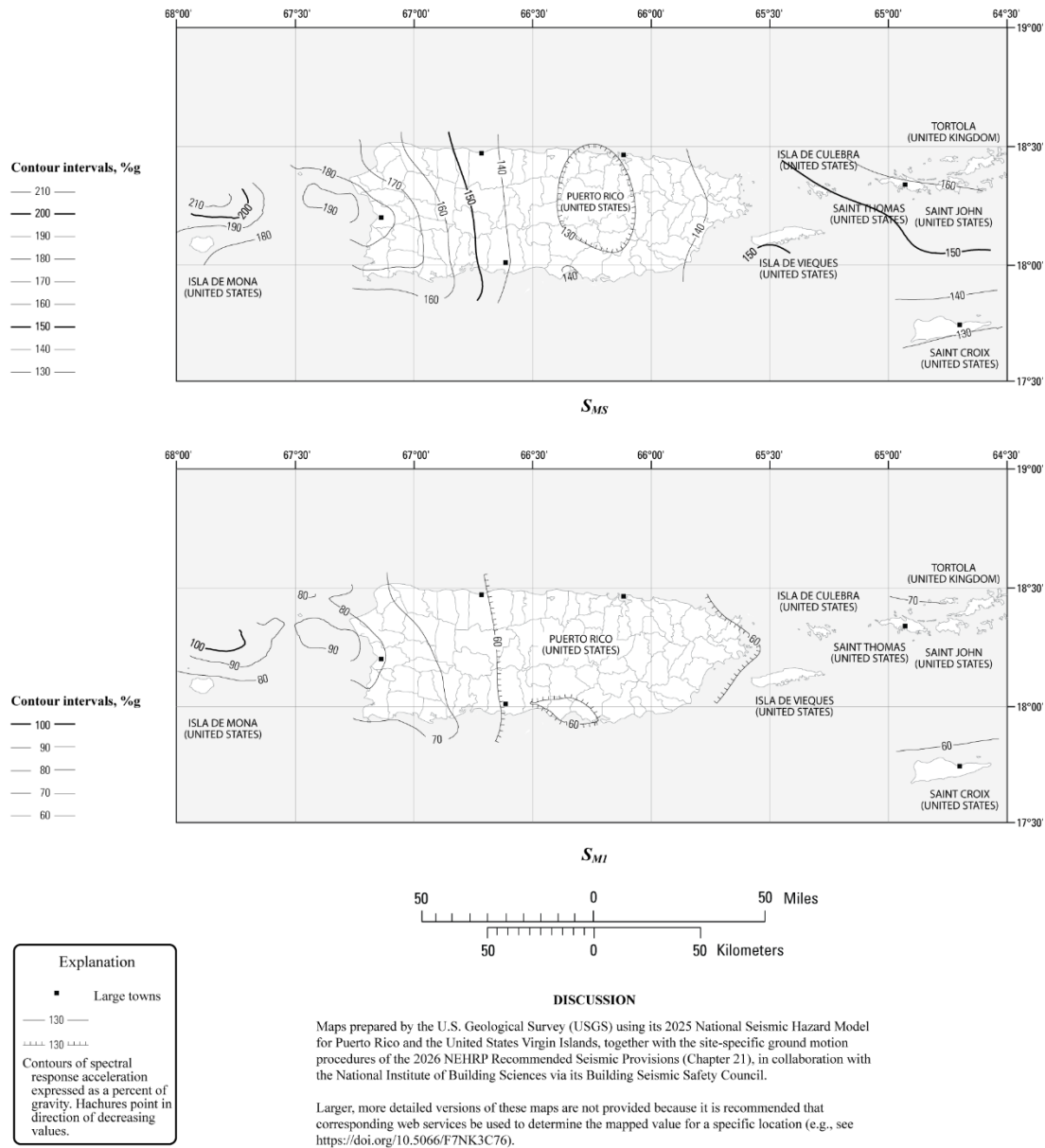


Figure 22-6. S_{Ms} and S_{M1} for the default site conditions for Puerto Rico and the US Virgin Islands.

Figure 22-7. S_{Ms} and S_{M1} for the default site conditions for Guam and the Northern Mariana Islands.

This map would not change with respect to ASCE/SEI 7-22 (for brevity it is not included here).

Figure 22-8. S_{Ms} and S_{M1} for the default site conditions for American Samoa.

This map would not change with respect to ASCE/SEI 7-22 (for brevity it is not included here).

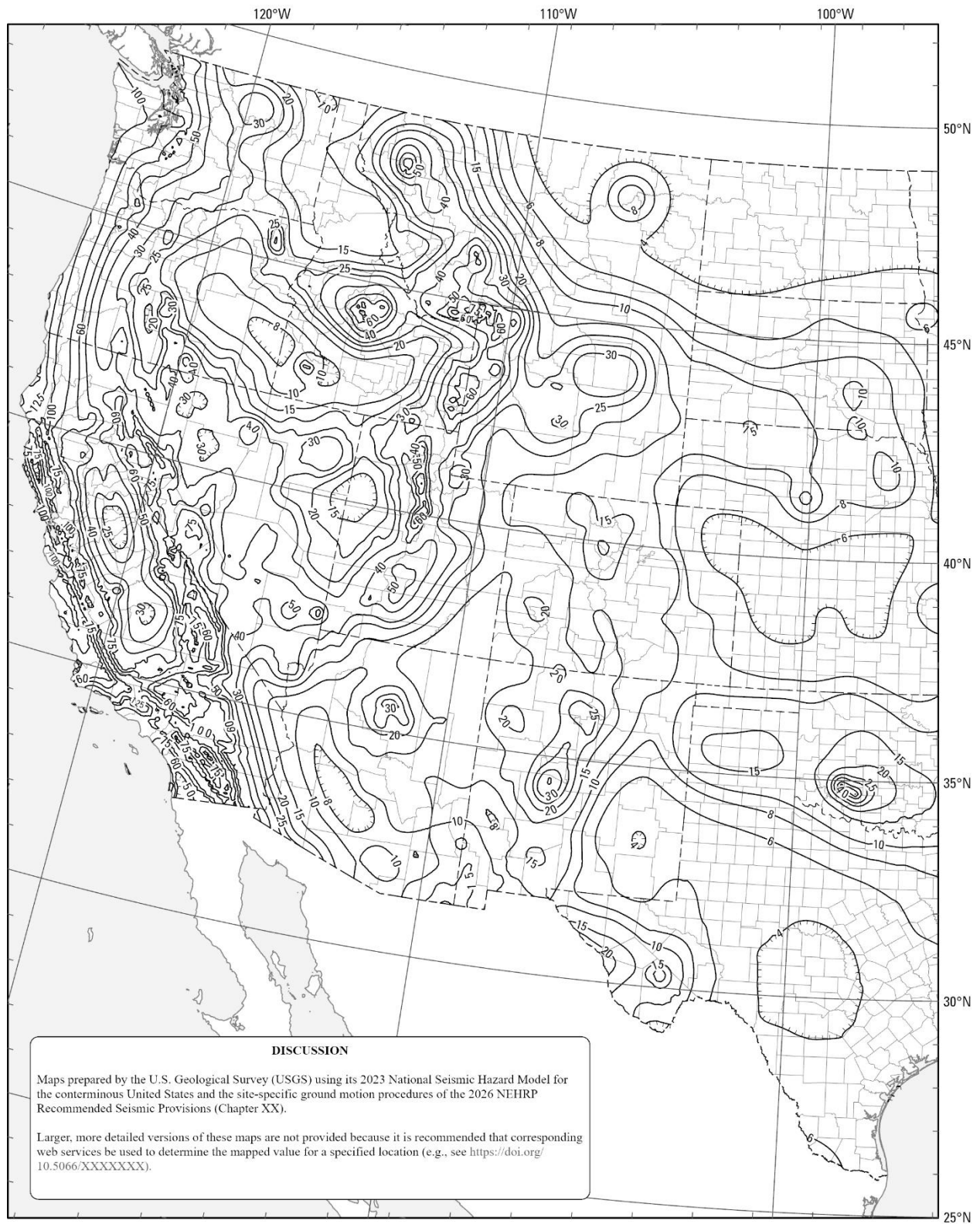


Figure 22-3 PGA_M for the Default Site Class, for the Conterminous United States.

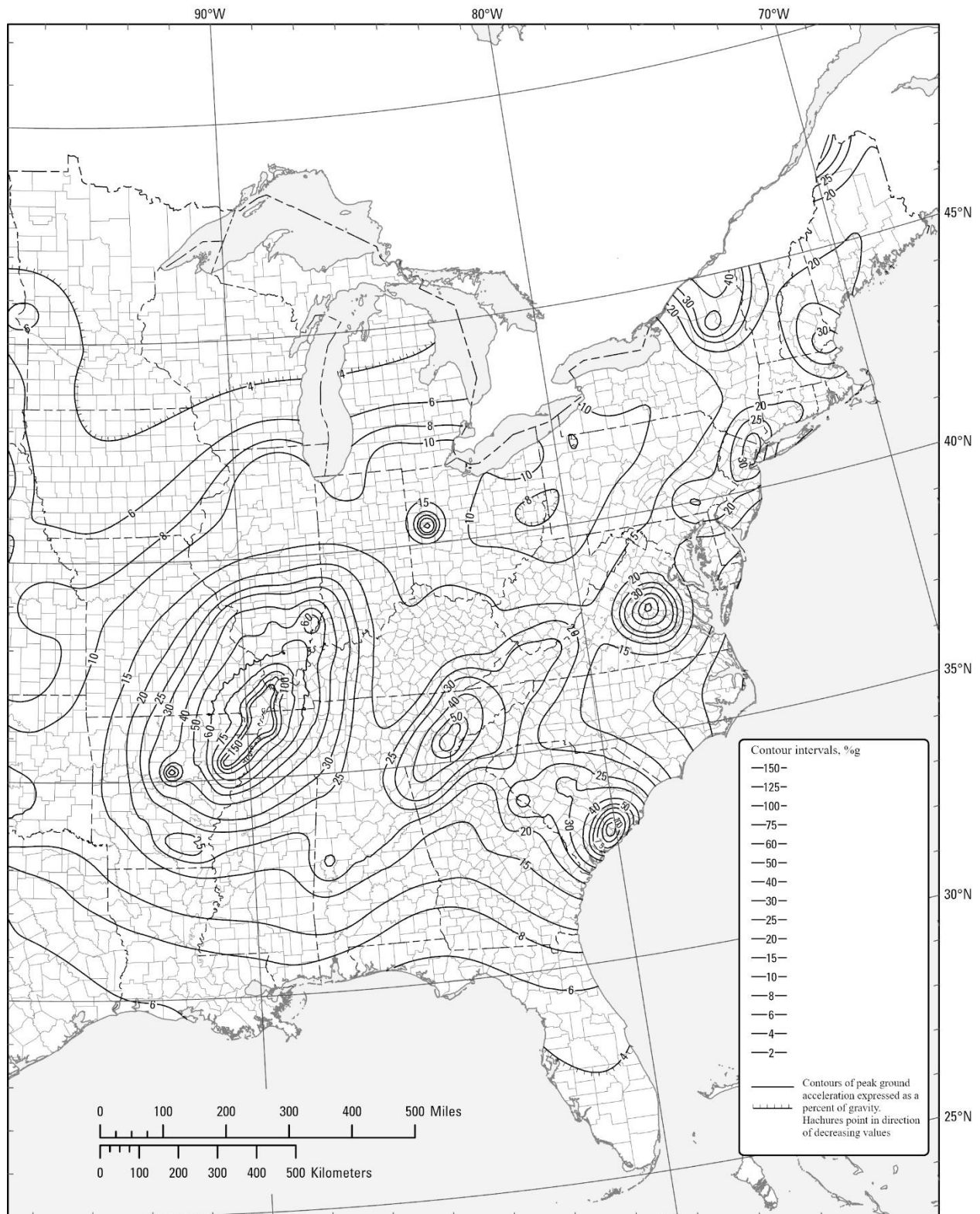


Figure 22-3 (continued) PGA_M for the Default Site Class, for the Conterminous United States.

Figure 22-9. PGA_M for the default site conditions for the conterminous United States.

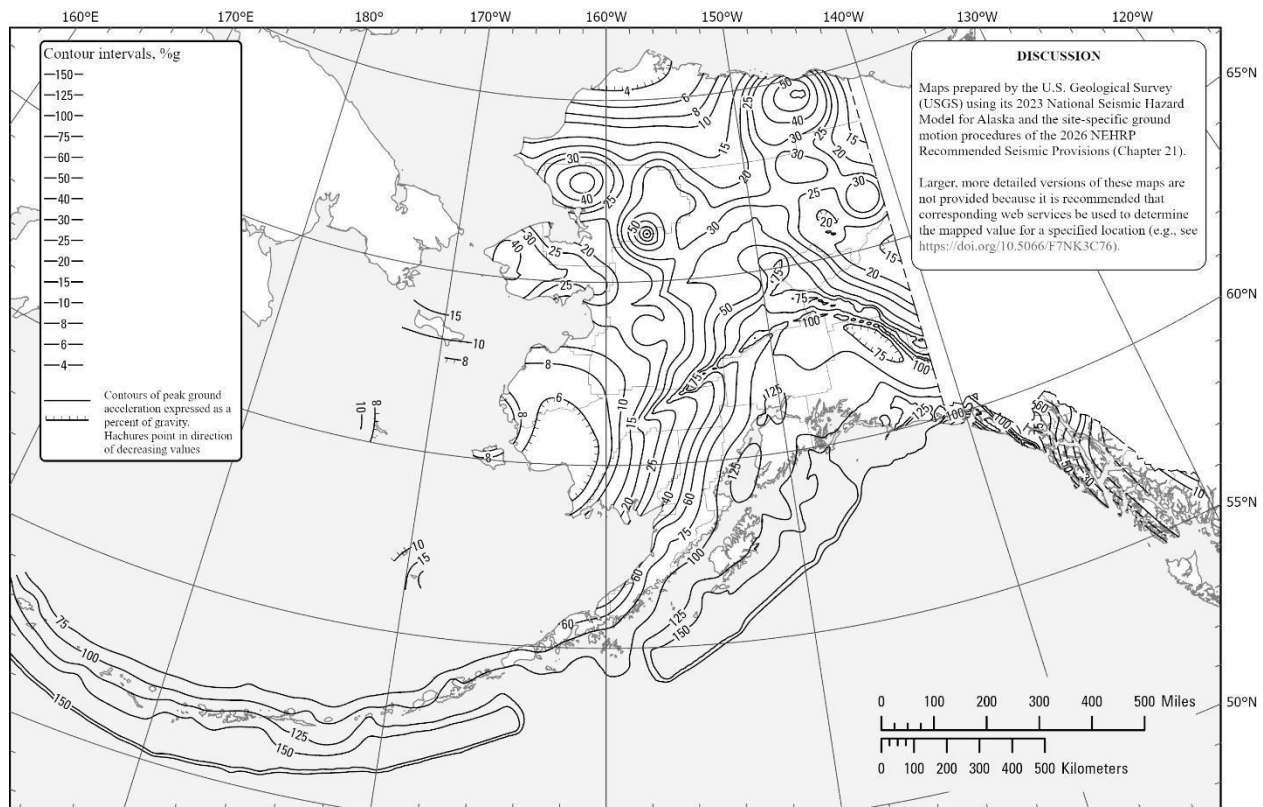
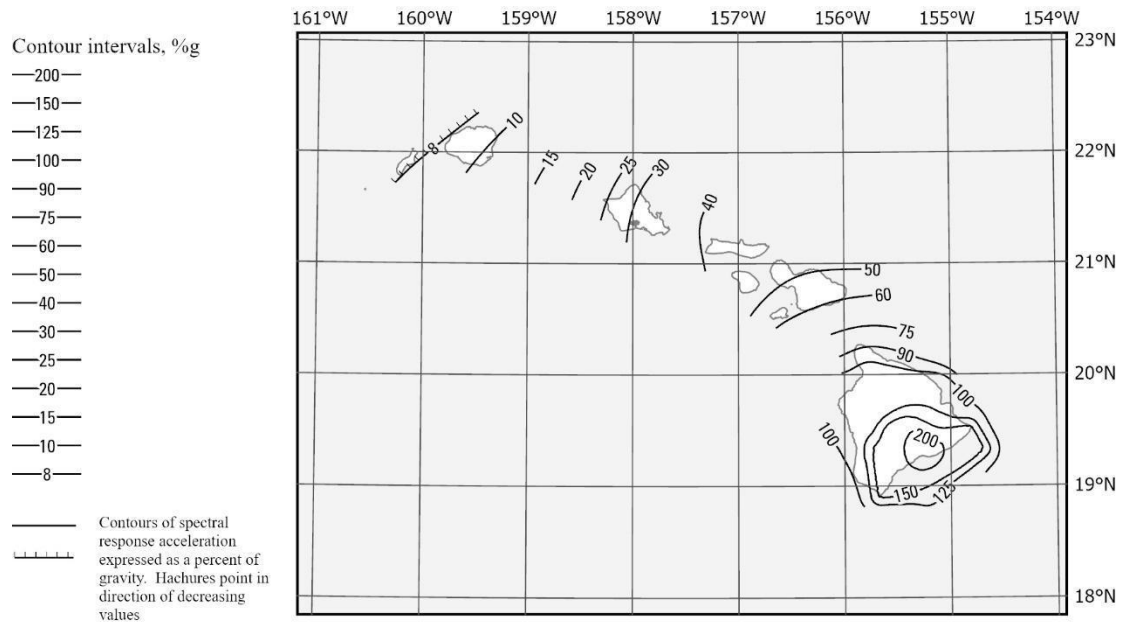


Figure 22-10. PGA_M for the default site conditions for Alaska.



DISCUSSION

 PGA_M

Maps prepared by the U.S. Geological Survey (USGS) using its 2021 National Seismic Hazard Model for Hawaii and the site-specific ground motion procedures of the 2026 NEHRP Recommended Seismic Provisions (Chapter 21).

Larger, more detailed versions of these maps are not provided because it is recommended that corresponding web services be used to determine the mapped value for a specified location (e.g., see <https://doi.org/10.5066/F7NK3C76>).

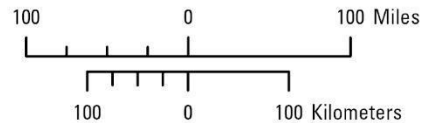


Figure 22-11. PGA_M for the default site conditions for Hawaii.

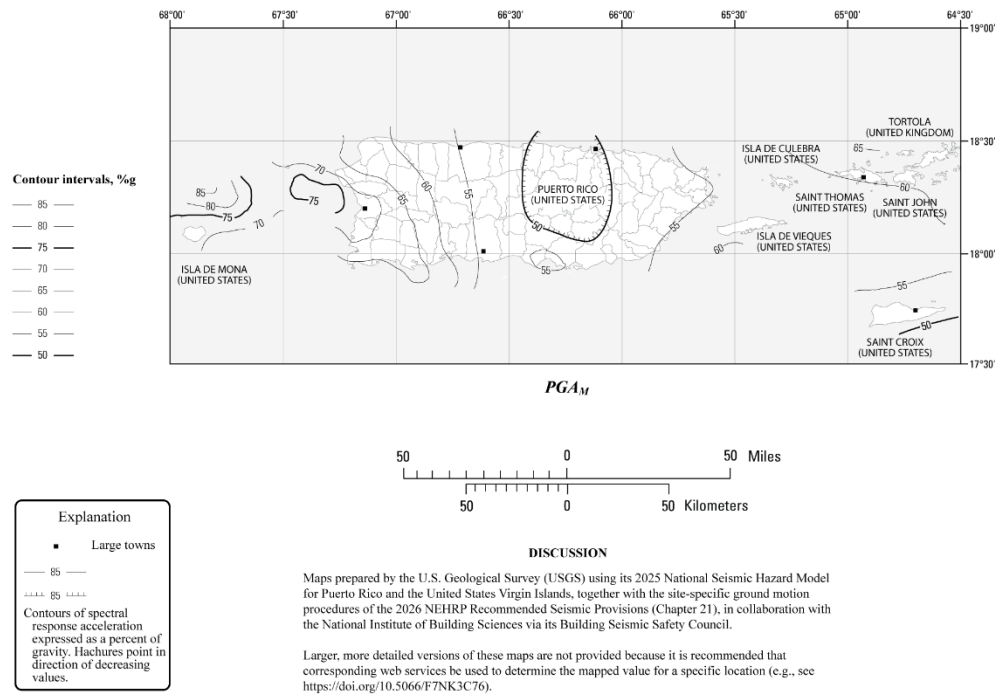


Figure 22-12. PGA_M for the default site conditions for Puerto Rico and the US Virgin Islands.

Figure 22-13. PGA_M for the default site conditions for Guam and the Northern Mariana Islands and American Samoa.

This map would not change with respect to ASCE/SEI 7-22 (for brevity it is not included here).

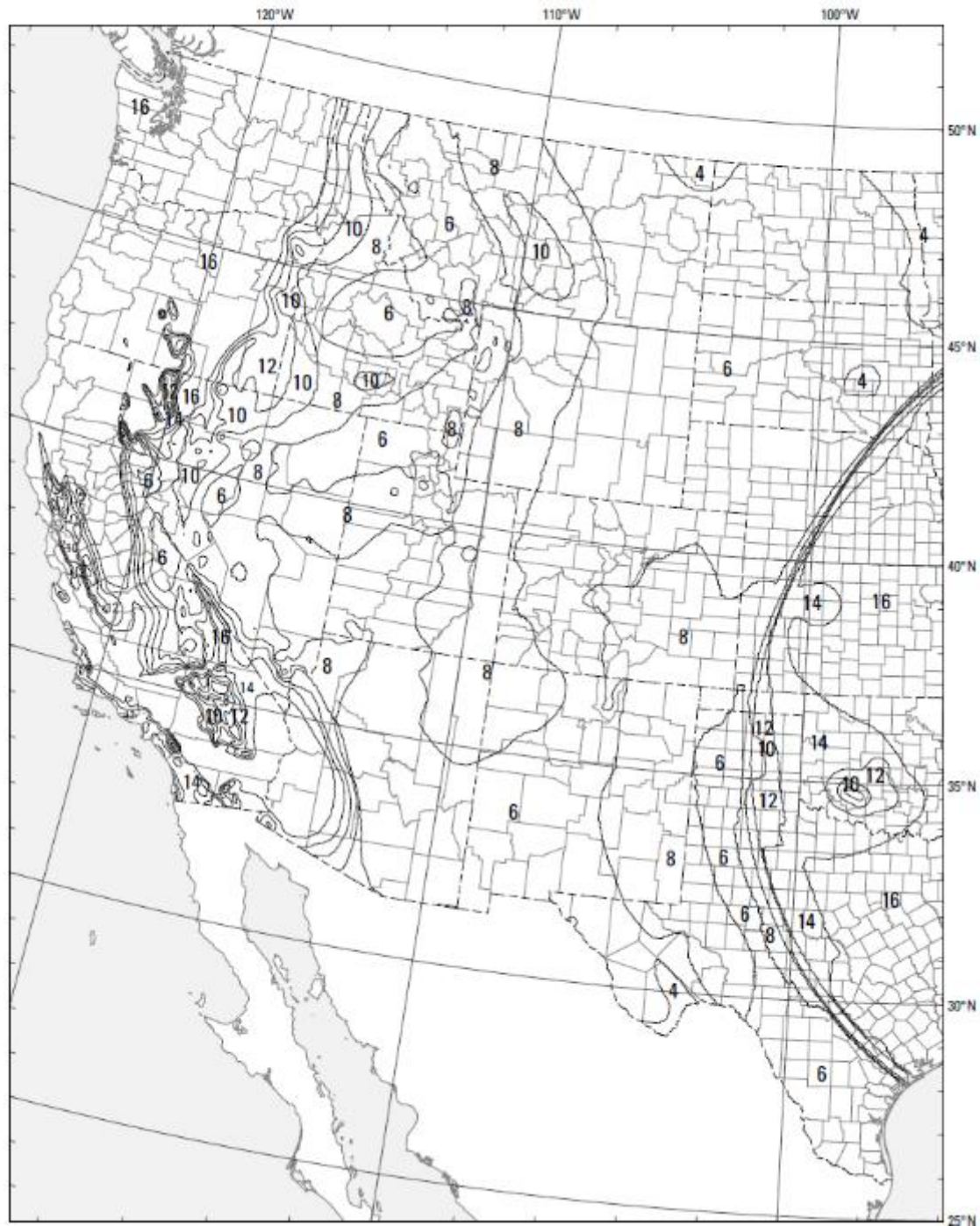


Figure 22-14.1. T_L for the Western conterminous United States.

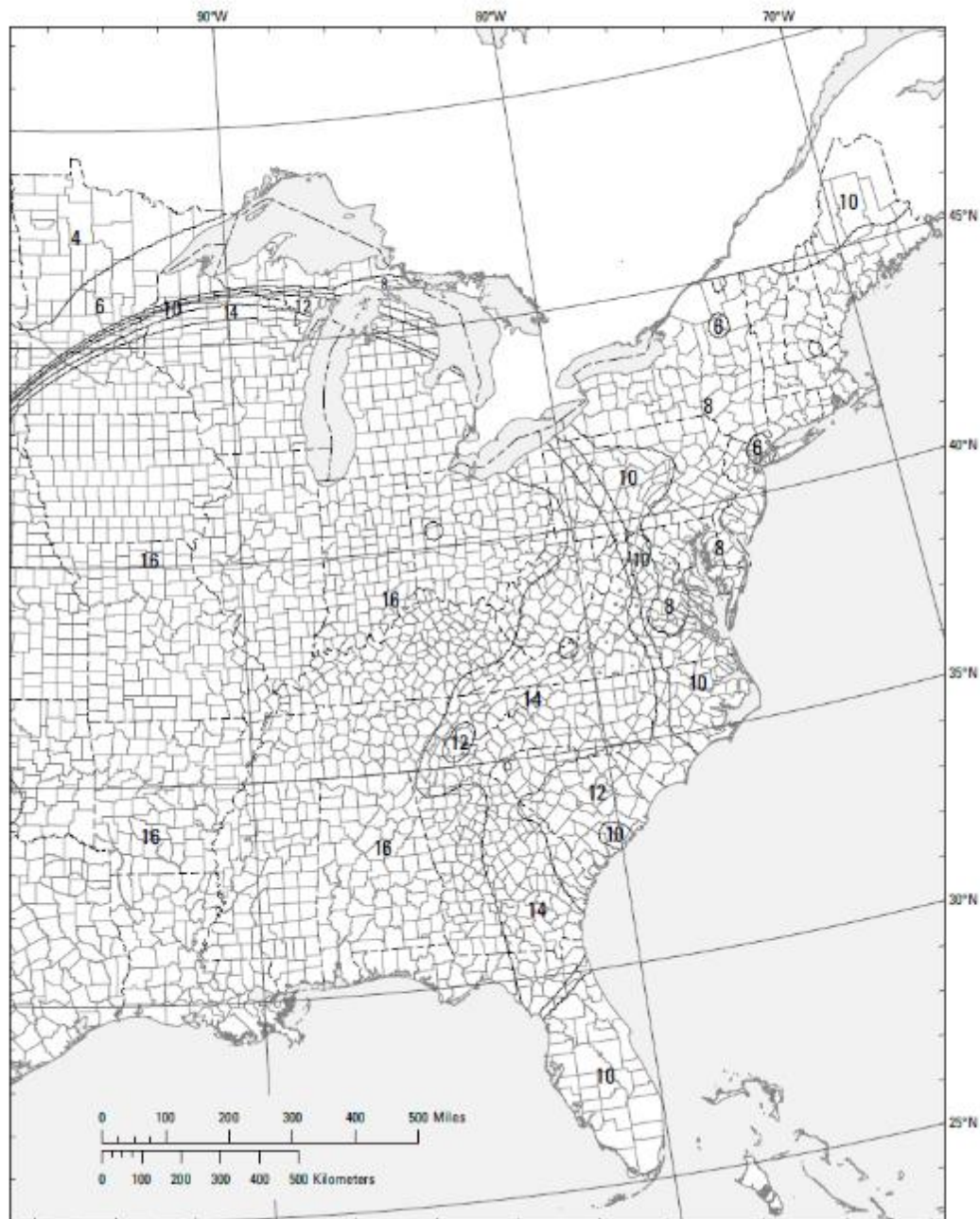


Figure 22-14.2. T_l for the Central and Eastern conterminous United States.

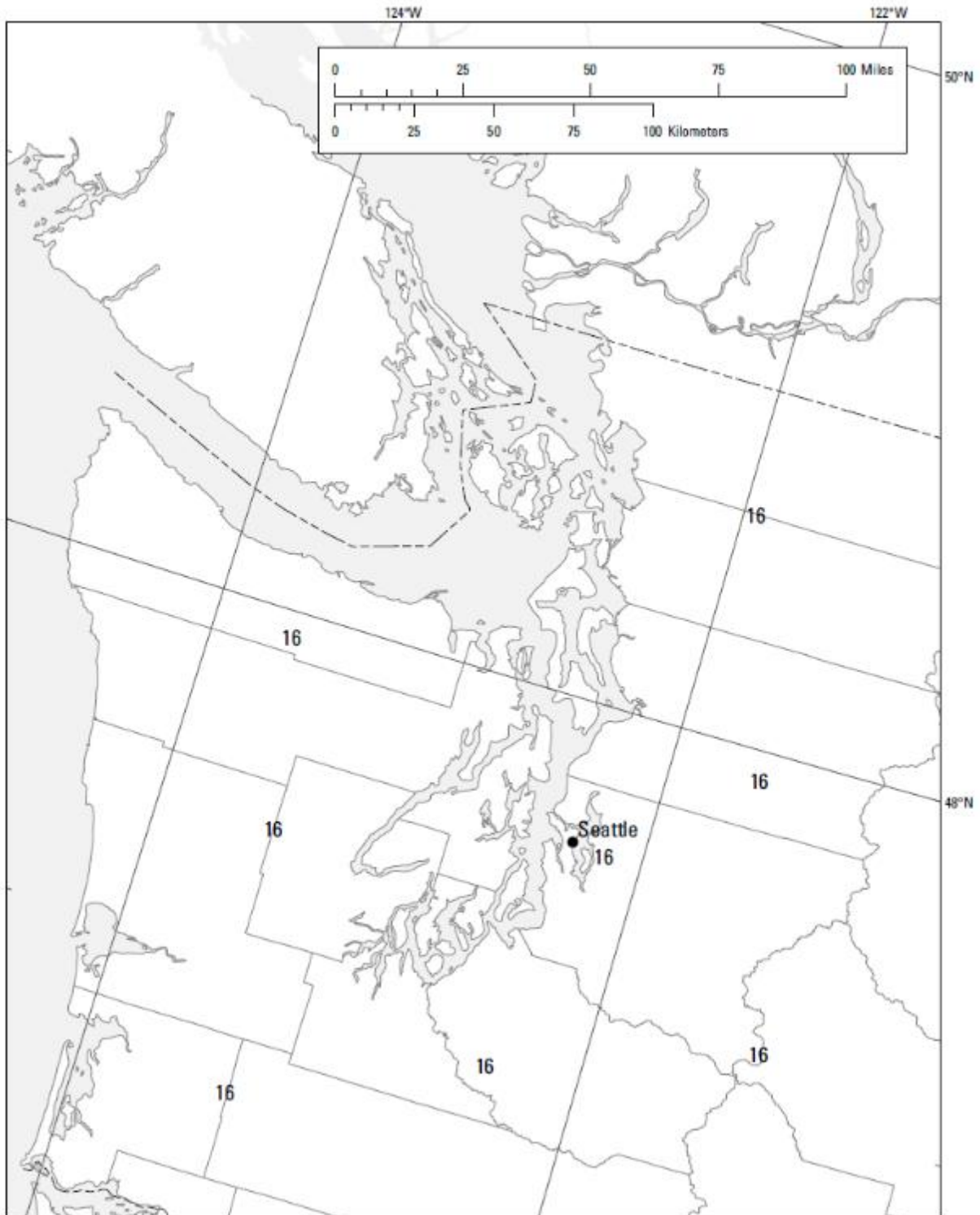


Figure 22-14.3. T_L for Seattle Area of the conterminous United States.

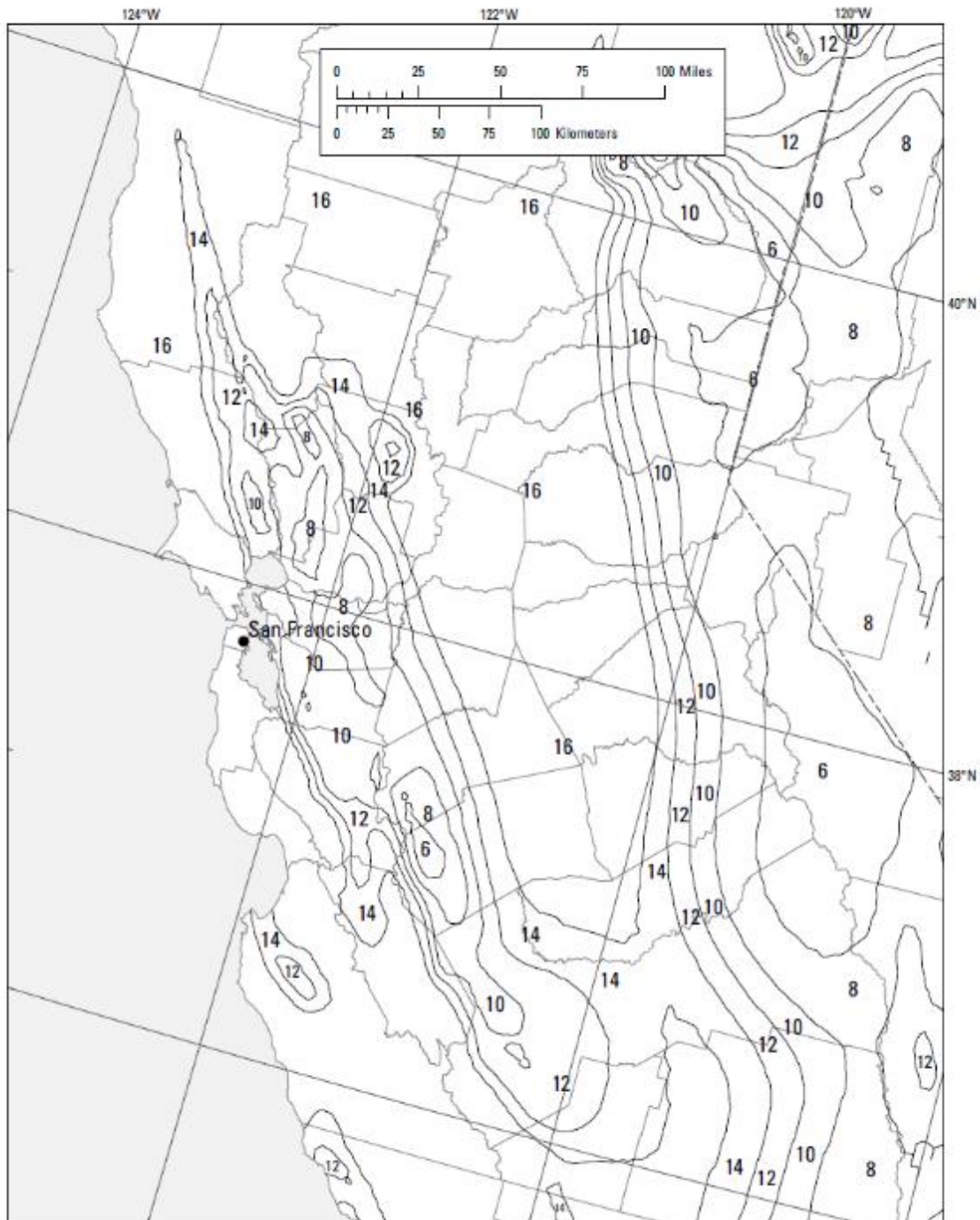


Figure 22-14.4. T_L for Bay Area of the conterminous United States.

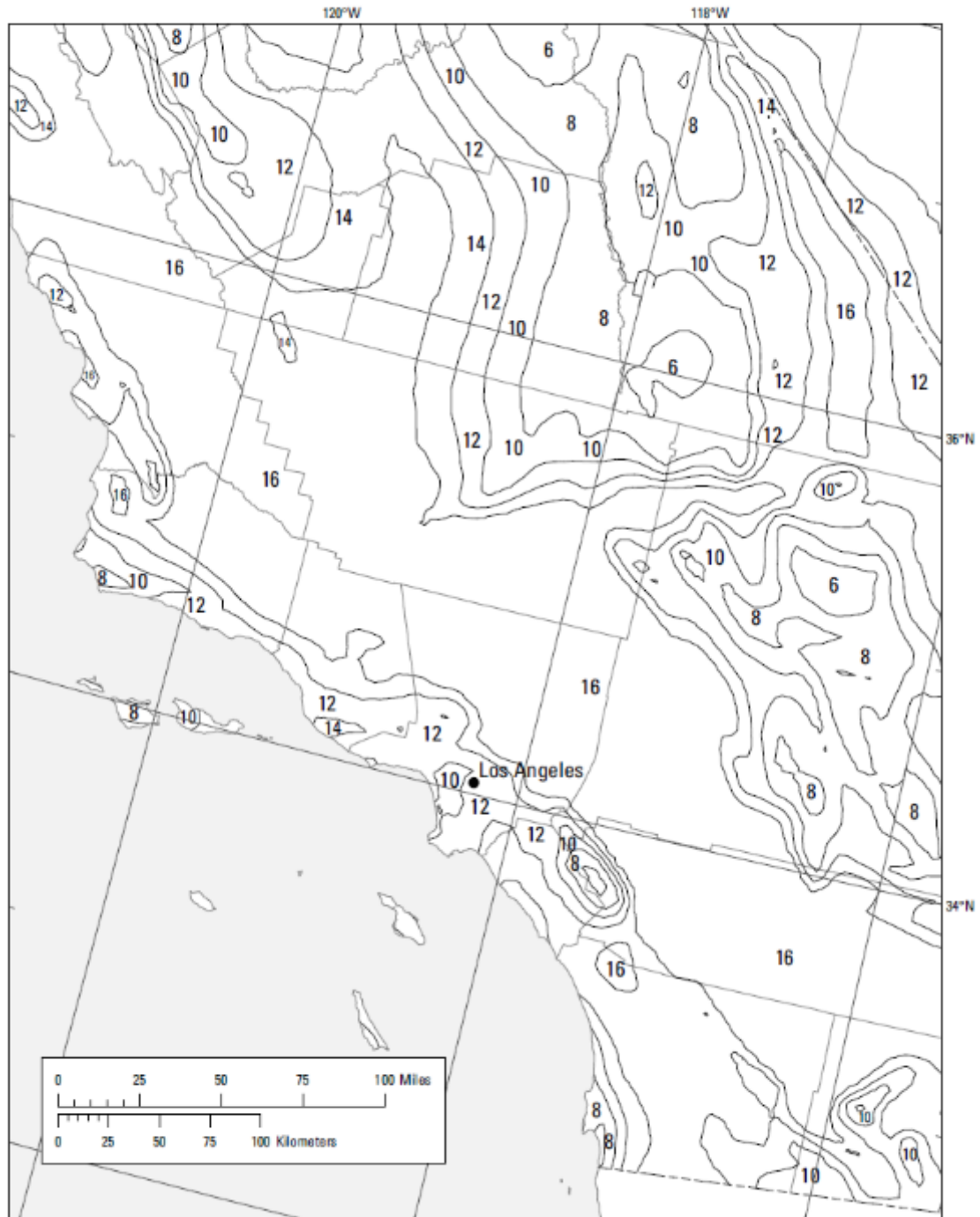


Figure 22-14.5. T_L for Los Angeles Area of the conterminous United States.

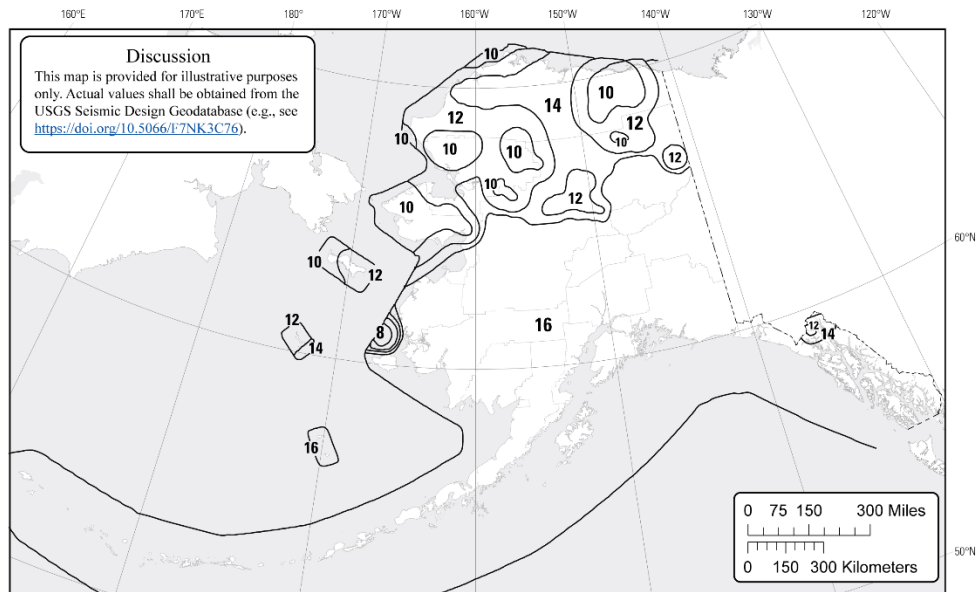


Figure 22-15. T_L for Alaska.

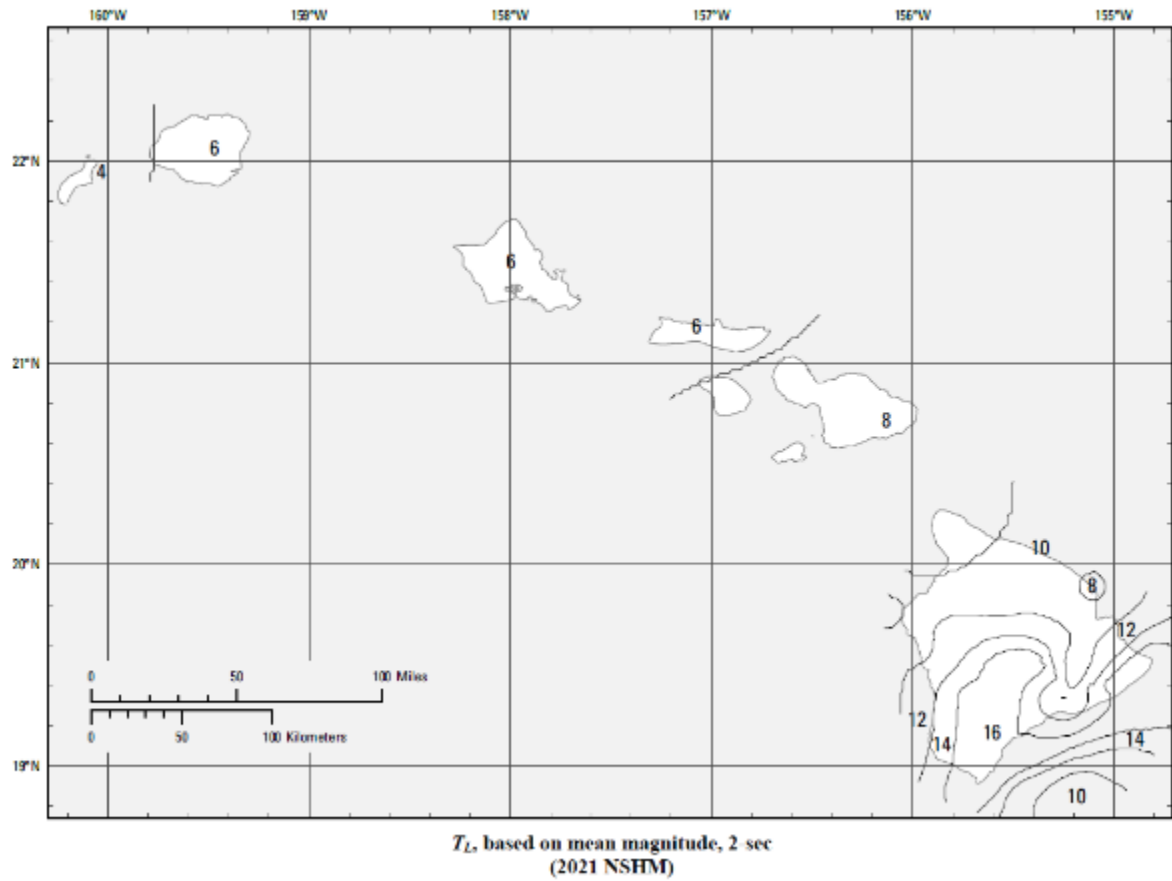


Figure 22-16. T_L for Hawaii.

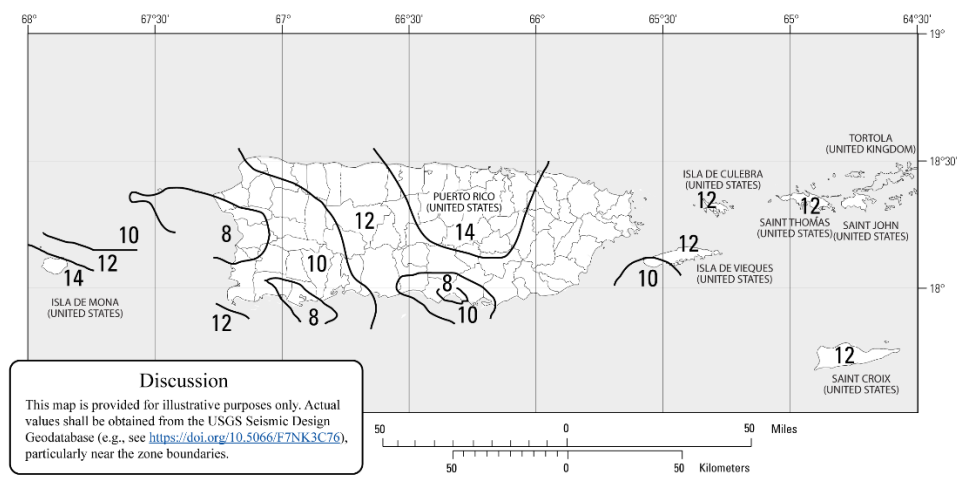


Figure 22-17. T_L for Puerto Rico and the US Virgin Islands.

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APPENDIX A, SEISMIC DESIGN CRITERIA AND OTHER REQUIREMENTS FOR FUNCTIONAL RECOVERY

Add Appendix A as follows:

A.1 GENERAL

A.1.1 Scope This appendix is not a mandatory part of the provisions but may be adopted by an Authority Having Jurisdiction. Where adopted, the seismic analysis, design, documentation, review, observation, and quality assurance for structures assigned to Functional Recovery Categories A through E shall comply with the applicable requirements of this appendix in full, in addition to all applicable requirements in Chapters 11 through 21 and 23.

A.1.2 Seismic Design Category The Seismic Design Category assignment for a structure shall be in accordance with Section 11.6 with S_1 , S_{DS} and S_{D1} unmodified by this appendix.

A.1.3 Exemptions The following structures shall be permitted to be exempt from the requirements of this appendix:

1. Structures assigned to Seismic Design Category A.
2. Structures exempt in accordance with Section 11.1.2.
3. Detached one- and two-family wood-frame dwellings with not more than two stories above grade plane, satisfying the limitations of and constructed in accordance with the IBC.
4. Structures assigned to Functional Recovery Category E.

A.1.4 Deemed to Comply The following structures shall be deemed to comply with the requirements of this appendix. No further analysis, design, documentation, review, observation, or quality assurance in accordance with this appendix shall be required.

1. Structures assigned to Functional Recovery Category D in Seismic Design Category B.
2. Seismically isolated structures assigned to Functional Recovery Category D designed in accordance with Chapter 17 for the MCE_R .

A.1.5 Applicable Requirements Except as permitted in accordance with Section A.1.3 or A.1.4:

- Buildings and their nonstructural components shall comply with the sections referenced for either a prescriptive or explicit assessment in Table A.1-1. The prescriptive assessment procedure shall not be permitted for buildings with damping systems.
- Nonbuilding structures and their nonstructural components shall comply with the sections referenced for a prescriptive assessment in Table A.1-2. The prescriptive assessment procedure shall not be permitted for nonbuilding structures with damping systems.

A.1.6 Independent Review Requirements for Prescriptive Assessment Procedure of Buildings

Assigned to Functional Recovery Category A Where a prescriptive assessment procedure is selected in Table A.1-1 for buildings assigned to Functional Recovery Category A, the independent review requirements of this section shall apply.

A.1.6.1 Review Scope and Reviewer Qualifications An independent review of the functional recovery design, documentation, and quality assurance shall be performed by one or more individuals acceptable to the Authority Having Jurisdiction and possessing knowledge of the following items, with a minimum of one reviewer being a registered design professional. The independent review shall include, but not be limited to, consideration of all of the following:

1. Determination of the basic intended function of the structure in accordance with Section A.4. This shall include review of the determination of nonstructural systems, elements, components, equipment, and distribution systems deemed not required to support the basic intended function of the structure as well as elements, components, equipment and distribution systems deemed rugged.
2. Determination of structural demands and capacities and supporting analyses in accordance with Section A.8.
3. Determination of nonstructural demands and capacities and supporting analyses in accordance with Section A.10.
4. Seismic certification and any exceptions in accordance with Section A.11 and A.12.4.
5. Documentation and delegated design in accordance with Sections A.12.2, A.12.3, and A.12.5.
6. Quality assurance in accordance with Section A.14.
7. Functional recovery plan in accordance with Section A.15.
8. Onsite backup requirements in accordance with Section A.16.

The independent review shall not be performed by the functional recovery coordinator. Where independent review of other aspects of the analysis or design are required by Chapters 1 or 16 or Section A.17, the independent review in accordance with this section shall be permitted to be

performed by the same individual(s), provided the individual(s) meet the qualification requirements above.

A.1.6.3 Review Letter Upon completion of the review, the reviewer(s) shall provide the Authority Having Jurisdiction with a letter attesting to all of the following:

1. Scope of review performed.
2. Whether the reviewer(s) concur that the functional recovery design, documentation, quality assurance, plan, and back-up requirements have been performed in accordance with this appendix.
3. Any items relating to the functional recovery design, documentation, or quality assurance that require further resolution by the Authority Having Jurisdiction.

Table A.1-1. Applicable Requirements for Buildings

Description	Section							
	A.1 - A.7	A.8	A.9	A.10	A.11 - A.14	A.15 - A.16	A.17	A.18
Buildings Assigned to Functional Recovery Category A or B								
Prescriptive Assessment	X	X		X	X	X		X
Explicit Assessment	X			X	X	X	X	X
Buildings Assigned to Functional Recovery Category C or D								
Prescriptive Assessment	X	X		X	X			X
Explicit Assessment	X			X	X		X	X

Note: X = applicable requirements

Table A.1-2. Applicable Requirements for Nonbuilding Structures

Description	Section							
	A.1 - A.7	A.8	A.9	A.10	A.11 - A.14	A.15 - A.16	A.17	A.18
Nonbuilding Structures Assigned to Functional Recovery Category A or B								
Prescriptive Assessment	X		X	X	X	X		X
Nonbuilding Structures Assigned to Functional Recovery Category C or D								

Prescriptive Assessment	X		X	X	X			X
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Note: X = applicable requirements

A.1.7 Target Risk and Reliability for Explicit Assessment Procedure Structures designed using the Explicit Functional Recovery Assessment Procedure in accordance with Section A.17 shall be demonstrated, by analysis, to provide a reliability that is consistent with the target reliability stipulated in this section. The target reliability shall not exceed a 25% probability of exceeding the target functional recovery time conditioned on occurrence of the functional recovery earthquake (FRE_R) response spectrum as defined in Section A.6. The FRE_R response spectrum shall be based on a risk target defined in accordance with Section A.6 as a 10% probability of exceeding the target functional recovery time in 50 years. The target recovery time shall be taken in accordance with Table A.1-3.

Table A.1-3. Target Functional Recovery Time

Functional Recovery Category	Target Functional Recovery Time	
	General Expectation	Acceptance Criteria
A	Within a few hours	0 hours
B	Within a few days	3 days
C	Within a couple of months	2 months
D	Within half a year	6 months
E	No target time	No target time

A.2 Definitions

The following terms are used in the *Provisions* of this appendix. Additional terms considered important for conceptual understanding, but not used in the *Provisions*, are provided in the *Commentary*.

ARCHETYPE: A prototypical representation of a combined gravity and seismic force-resisting system.

BASIC INTENDED FUNCTION: A designated subset of the activity or range of activities for which the *structure* is intended, as indicated by its occupancy classification(s) and use designation(s), in compliance with Section A.4.

EXTERNAL FACTOR: A condition that affects the functional performance of a *structure* that is outside the *system boundary* and independent of damage to the structure.

FUNCTIONAL RECOVERY: The post-earthquake *recovery state* in which a *structure* is maintained, or restored, to safely and adequately support the *basic intended function* associated with its pre-earthquake use or occupancy.

FUNCTIONAL RECOVERY CATEGORY: A categorization assigned to a *structure* for determination of functional recovery design criteria and other requirements, based on the necessity of its occupancy classification(s) and use designation(s) as they relate to recovery of community functions or services following an earthquake.

FUNCTIONAL RECOVERY COORDINATOR: The registered design professional responsible for coordinating design and construction of a structure to meet the functional recovery objective associated with the assigned Functional Recovery Category.

FUNCTIONAL RECOVERY EARTHQUAKE (FRE_R): The earthquake ground-shaking intensity for the design of *structures* to meet the *functional recovery objective*. General procedures for determining the FRE_R ground motion values are provided in Section A.6; site-specific procedures are provided in Section A.6.3.

FUNCTIONAL RECOVERY TIME: The time to achieve a functional *recovery state* after an earthquake, including the effects of *impeding factors* but excluding the effects of *external factors*.

IMPEDING FACTOR: An aspect of post-earthquake inspection, design, or construction that delays repair and is dependent on damage to the *structure*.

PERFORMANCE GROUP: A group of archetypes, binned for statistical evaluation.

RECOVERY STATE: A milestone on the post-earthquake recovery timeline that describes performance of a *structure* in terms of level of functionality.

STRUCTURE: That which is built or constructed and limited to buildings and nonbuilding structures, including the structural and nonstructural systems of such structures.

SYSTEM BOUNDARY: A demarcation of the limits considered in the design of a *structure*, including the *structure* itself, appurtenant *structures*, site equipment, and interconnecting elements that directly serve the functions of the *structure*.

A.3 SYMBOLS

The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. Symbols presented in this section apply only to the provisions of this appendix of this standard. The symbols of Chapter 11 shall also apply.

A_{FLX-H} = horizontal spectral acceleration calculated for flexible components for ICC-ES AC156

A_{FLX-V} = vertical spectral acceleration calculated for flexible components for ICC-ES AC156

A_{RIG-H} = horizontal spectral acceleration calculated for rigid components for ICC-ES AC156

A_{RIG-V} = vertical spectral acceleration calculated for rigid components for ICC-ES AC156

$C_{d-diaphfr}$ = functional recovery deflection amplification factor for diaphragm deflection

C_{dfr} = functional recovery deflection amplification factor

D_i = inside diameter of the tank

I_{pfr} = seismic functional recovery component importance factor

I_{qualfr} = Seismic functional recovery qualification importance factor

L = longitudinal plan dimension of rectangular tank

= horizontal distance between two points over which differential settlement is measured

$R_{diaphfr}$ = functional recovery response modification coefficient for design of diaphragms

R_{fr} = functional recovery response modification coefficient

$R_{\mu fr}$ = functional recovery structure ductility reduction factor

S_{afr} = FRE_R , 5% damped, spectral response acceleration parameter at any period adjusted for site class effects

S_{acfr} = FRE_R , 5% damped, spectral response acceleration parameter of the sloshing liquid

S_{D1fr} = FRE_R , 5% damped, spectral response acceleration parameter at a period of 1 s adjusted for site class effects

S_{DSfr} = FRE_R , 5% damped, spectral response acceleration parameter at short periods adjusted for site class effects

Δ_{afr} = allowable story drift for functional recovery

$\Omega_{0-diaphfr}$ = functional recovery diaphragm overstrength factor

Ω_{0fr} = functional recovery overstrength factor

δ_{sfr} = height of sloshing liquid for functional recovery

A.4 BASIC INTENDED FUNCTION

A.4.1 Responsibility The basic intended function of the structure and the elements, components, equipment, and distribution systems required to support the basic intended function shall be

determined by the registered design professional responsible for the architectural design, registered design professional responsible for the structural design, functional recovery coordinator, and owner.

A.4.2 Systems Supporting Basic Intended Function Except as exempted under Section A.4.3, the elements, components, equipment, and distribution systems composing the structural and nonstructural systems required to support the basic intended function of the structure shall be designed and constructed in accordance with this appendix.

A.4.2.1 Minimum Required Systems Elements, components, equipment and distribution systems composing the following systems shall meet the requirements of this appendix:

1. Structural seismic force-resisting systems
2. Structural gravity force-resisting systems
3. Means of egress systems
4. Fire protection systems
5. Electrical power systems
6. Exterior enclosure systems
7. Potable water systems
8. Sanitary drainage systems

A.4.2.2 Additional Systems Elements, components, equipment, and distribution systems composing the following systems shall meet the requirements of this appendix except where such systems are not required to support the basic intended function determined in accordance with Section A.4.1 and where damage to such systems does not adversely affect the basic intended function of the structure:

1. Heating, ventilation, and air-conditioning systems
2. Interior architectural components and lighting systems
3. Elevator systems
4. Telecommunication and data systems
5. Other structure- or occupancy-specific systems, determined in accordance with Section A.4.1.

A.4.3 Exemptions

1. Nonstructural elements, components, equipment, and distribution systems not required to support the basic intended function of the structure if damage to such elements,

components, equipment, or distribution systems does not adversely affect the basic intended function of the structure determined in accordance with Section A.4.1 shall be exempt from the requirements of this appendix.

2. Furniture (except storage cabinets, as noted in Table 13.5-1).
3. Temporary elements, components, equipment, and distribution systems that remain in place for 180 days or less.
4. For structures assigned to Functional Recovery Category C or D, nonstructural elements, components, equipment, and distribution systems that are designed to be mobile as part of their ordinary use.

Except as listed in Section A.10.2, Section A.11.1 or above, the exemptions of Section 13.1.4 shall not apply to this appendix.

A.5 Functional Recovery Category

A.5.1 Designation of Functional Recovery Category

Buildings and structures shall be assigned to a Functional Recovery Category based on their basic intended function, according to Table A.5-1.

Where a structure has multiple basic intended functions, it shall be assigned to the highest Functional Recovery Category for any of its basic intended functions. For this purpose, Functional Recovery Category A is the highest category, and Functional Recovery Category D is the lowest category. Category E is for structures not designed for functional recovery.

Table A.5-1 Functional Recovery Category for Structures for Earthquake Loads

Functional Recovery Category	Contribution of Structure to Community Resilience
A	Structures with a basic intended function that the community relies on for immediate post-earthquake safety and survival.
B	Structures with a basic intended function that supports urgent and basic human survival needs and prevents the escalation of adverse disaster consequences.
C	Structures with a basic intended function that supports recovery-focused self- and group-preservation, and sustains short- and long-term economic, educational, and governance activities and services.
D	Structures with a basic intended function that enhances a community's general well-being and expedites the return to normalcy.
E	Structures not assigned to FRC A, B, C, or D.

EXCEPTION:

Where portions of a structure provide different basic intended functions, it is permitted to assign the entire portions or specific nonstructural elements, components, equipment, and distribution systems within the portions of the structure to different Functional Recovery Categories for purposes of satisfying the requirements of this Appendix when all of the following requirements are satisfied:

1. Sections A.8 or A.9, as applicable, shall be satisfied to the requirements of the highest Functional Recovery Category assigned within the structure.
2. Nonstructural elements, components, equipment, distribution systems and contents specific to this exception shall be independent or have appropriate switches to disconnect at the boundary between the different portions of the structure.
3. Damage of nonstructural elements, components, distribution systems and contents specific to this exception in the portion of the structure with longer target recovery time shall not affect the functional recovery, either directly or indirectly, of the other portion(s).
4. Construction documents shall satisfy requirements of Section A.12.2.

A.5.1.1 Designation of Functional Recovery Category for Structures Assigned to Risk Category IV

Structures assigned to Risk Category IV and not supporting basic intended functions assigned to Functional Recovery Category A shall be assigned to Functional Recovery Category B regardless of their functions.

EXCEPTION:

Structures assigned to Risk Category IV solely based on use or occupancy that contain sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released shall be assigned to a Functional Recovery Category according to Table A.5-1.

A.5.1.2 Designation of Functional Recovery Category for Nonbuilding Structures

Nonbuilding structures shall be assigned to a Functional Recovery Category based on their basic intended function, according to Table A.5-1 For purposes of Section A.9 only, nonbuilding structures assigned to Functional Recovery Category A, B, C, or D, shall be designed according to design requirements for nonbuilding structures assigned to Functional Recovery Category B.

A.5.1.3 Designation of Functional Recovery Category for Appurtenant Structures

Where a structure is appurtenant to a main structure within its system boundary, that structure and the entire pathway between the appurtenant and main structure shall be assigned to the same Functional Recovery Category as the main structure.

EXCEPTION:

Where a structure is appurtenant to a main structure, and the performance of the appurtenant structure does not affect the functional recovery of the main structure, the appurtenant structure and the pathway between the appurtenant and main structure shall be assigned to a Functional Recovery Category based on their basic intended function, according to Table A.5-1.

A.6 SEISMIC GROUND MOTION VALUES

A.6.1 Site Class The site class shall be determined in accordance with Section 11.4.2 with the maximum considered earthquake (MCE_R) spectral response accelerations replaced by the functional recovery earthquake (FRE_R) response accelerations.

A.6.2 Functional Recovery Earthquake (FRE_R) Response Spectrum The FRE_R response spectrum shall be developed either as a multi-period response spectrum according to Section 11.4.5.1 with S_a replaced by S_{afr} and taking S_{afr} as equal to the multi-period 5%-damped FRE_R response spectrum from the USGS Seismic Design Geodatabase for the applicable site class, or as a two-period response spectrum according to Section 11.4.5.2 with S_{DS} and S_{D1} replaced by S_{DSfr} and S_{D1fr} , respectively. FRE_R spectral response acceleration parameters S_{afr} , S_{DSfr} and S_{D1fr} shall be obtained from the USGS Seismic Design Geodatabase for the applicable site class. At sites where the USGS Seismic Design Geodatabase is not applicable, a site-specific ground motion hazard analysis shall be performed in accordance with Section A.6.3.2 to determine the FRE_R response spectrum.

The site-specific FRE_R spectral response acceleration at any period need not exceed the design spectral acceleration defined in Section 11.4.5.

For structures on Site Class F, a site response analysis shall be performed in accordance with Section A.6.3.1, unless exempted in accordance with Section 20.2.1.

It shall be permitted to perform a site response analysis in accordance with Section A.6.3.1 and/or a site-specific ground motion hazard analysis in accordance with Section A.6.3.2 to determine the FRE_R response spectrum for any structure.

A.6.3 Site-Specific FRE_R Ground Motion Procedures

A.6.3.1 Site Response Analysis When an FRE_R site response analysis is performed, it shall comply with the requirements of Section 21.1 with MCE_R replaced by FRE_R , the references to Section 11.4.7 replaced by references to Section A.6.2, and the references to Section 21.2 replaced by references to Section A.6.3.2.

A.6.3.2 Ground Motion Hazard Analysis When a site-specific FRE_R ground motion hazard analysis is performed, it shall comply with the requirements of Section 21.2, with MCE_R replaced by FRE_R , S_{aM} replaced by S_{afr} , the references to Section 11.4.7 replaced by references to Section A.6.2, the references to Section 21.1 replaced by references to Section A.6.3.1, the term “maximum response” replaced by “median-component response”, and Section 21.2.2 replaced with Section A.6.3.2.1.

A.6.3.2.1 FRE_R Ground Motions The FRE_R response spectrum shall be taken as the probabilistic median-component spectral response acceleration represented by a 5% damped acceleration response spectrum that is expected to achieve a 10%-in-50-years functional recovery target risk.

At each spectral response period for which the acceleration is computed, ordinates of the probabilistic ground motion response spectrum shall be determined from iterative integration of a site-specific ground motion hazard curve with a lognormal probability density function representing the functional recovery fragility (i.e., probability of exceeding the target functional recovery time as a function of spectral response acceleration). The ordinate of the probabilistic ground motion response spectrum at each period shall achieve a 10%-in-50-years risk target for a functional recovery fragility that has (1) a 25% reliability target (probability of exceedance at said ordinate of the probabilistic ground motion response spectrum) and (2) a logarithmic standard deviation value of 0.6.

A.6.3.3 FRE_R Spectral Response Acceleration Parameters Where the site-specific ground motion hazard analysis is used to determine the FRE_R response spectrum, in accordance with Section A.6.3.2, the spectral response acceleration parameter S_{DSfr} shall be taken as 90% of the maximum FRE_R spectral acceleration, S_{afr} , obtained from the site-specific spectrum at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1fr} shall be taken as 90% of the maximum value of the product, $T \cdot S_{afr}$, for periods from 1 to 2 s for sites with $V_s > 1,450$ ft/s ($V_s > 442$ m/s) and for periods from 1 to 5 s for sites with $V_s \leq 1,450$ ft/s ($V_s \leq 442$ m/s), but not less than 100% of the value of S_{afr} at 1 s.

A.6.4 FRE_R Ground Motions for Nonlinear Response History Analysis When nonlinear response history analysis for functional recovery is performed, it shall be in accordance with the requirements of Section 16.2 as modified by this section.

A.6.4.1 Target Response Spectrum For analysis and design for functional recovery, the 5%-damped, MCE_R response spectrum in Section 16.2.1 shall be replaced by the 5%-damped, FRE_R response spectrum.

A.6.4.2 Ground Motion Selection For analysis and design for functional recovery, a suite of ground motions shall be selected in accordance with the requirements of Section 16.2.2, with MCE_R replaced by FRE_R.

A.6.4.3 Period Range for Scaling or Matching For analysis and design for functional recovery, the upper bound on the period range in Section 16.2.3.1 need not exceed 1.5 times the largest first-mode period in the principal horizontal directions of response.

A.6.4.4 Amplitude Scaling and Spectral Matching For analysis and design for functional recovery, a median-component spectrum and median-component spectra of the selected ground motion record pairs shall be constructed, instead of the maximum-direction spectrum and spectra specified in Section 16.2.3.2 and Section 16.2.3.3, respectively.

A.7 GEOLOGIC HAZARDS

A.7.1 Surface Fault Rupture A structure shall not be located where a known potential exists for an active fault to cause rupture of the ground surface at the structure.

EXCEPTION: Structures assigned to Functional Recovery Category C or D shall be permitted to be located on sites where an explicit functional recovery assessment in accordance with Section A.17 demonstrates the functional recovery objective is satisfied for the Functional Recovery Category of the structure. The explicit functional recovery assessment shall consider the effects of the surface fault rupture on functional recovery.

A.7.2 Other Geologic Hazards A structure shall not be located where a potential exists for any of the following under the seismic ground motion hazard of Section A.6:

1. Slope instability
2. Liquefaction
3. Seismically-induced permanent ground displacement
4. Seismically-induced soil strength-loss

EXCEPTIONS:

1. Structures shall be permitted to be located on any site identified above where (a) the structure is founded on deep foundations which extend through the soil layer(s) exhibiting the geologic hazard and which are designed for the effects of the geologic hazard under the applicable load combinations and, (b) include a structural floor system capable of spanning between deep foundation elements under the applicable load combinations.
2. Structures assigned to Functional Recovery Category B, C or D shall be permitted to be located on sites identified in Items 2, 3 or 4 where the geotechnical investigation report indicates a negligible potential for the occurrence of permanent horizontal ground displacement, negligible bearing capacity loss, negligible lateral resistance loss, and differential settlements of site soils not exceeding 0.002L. L shall be determined in accordance with Section 12.13.9.2.
3. Structures shall be permitted to be located on sites where an explicit functional recovery assessment in accordance with Section A.17 demonstrates the functional recovery objective is satisfied for the Functional Recovery Category of the structure. The explicit functional recovery assessment shall consider the effects of geologic hazards on functional recovery.

A.8 ANALYSIS AND DESIGN OF STRUCTURAL SYSTEMS FOR BUILDINGS

A.8.1 General Analysis and design for functional recovery shall be in accordance with one of the following:

1. The provisions of Chapter 12 as modified by Sections A.8.2, A.8.3, and A.8.4 shall apply. Where linear analysis in accordance with Chapter 12 is used for the design earthquake, the analysis and design for functional recovery in accordance with this section shall be performed in addition to, not in lieu of, the unmodified requirements of Chapter 12. Where nonlinear response history analysis in accordance with Chapter 16 is used for the MCE_R , the analysis and design for functional recovery in accordance with this section shall be performed in addition to, not in lieu of, the unmodified requirements of Chapter 16.
2. The provisions of Chapter 16 as modified by Sections A.8.2, A.8.3, and A.8.5 shall apply. Where a nonlinear response history analysis for the FRE_R is performed, the analysis and design for functional recovery in accordance with this section shall be performed in addition to, not in lieu of, the unmodified requirements of Chapter 16.
3. For seismically isolated buildings, the provisions of Chapter 17 as modified by Sections A.8.2, A.8.3, and A.8.6 shall apply.

A.8.2 Seismic Importance Factor For structural analysis and design for functional recovery, the seismic importance factor, I_e , shall be taken as 1.0 except as indicated below.

For out-of-plane forces, anchorage of structural walls, and transfer of design forces into diaphragms or other supporting structural elements analyzed and designed for functional recovery in accordance with Section 12.11, the seismic importance factor, I_e , shall be replaced by the seismic functional recovery component importance factor, I_{pfr} , from Table A.10-1.

EXCEPTION: For structural analysis and design for functional recovery, the requirements of Section 12.11.2.2.2 shall be permitted to be neglected.

A.8.3 Material-Specific Seismic Design and Detailing Requirements for Functional Recovery

A.8.3.1 General The design and detailing modifications in this section shall apply in addition to the applicable design and detailing requirements of Chapter 14.

A.8.3.2 Precast Concrete Diaphragms in precast concrete structures shall meet all of the following:

1. Conform to the definition of Cast-In-Place Concrete Equivalent Precast Diaphragms in accordance with Section 14.2,
2. Consist of a topping slab not less than 3 inches (76 mm) thick.

3. Be reinforced with reinforcing bars.

Bearing connections of precast concrete structural members not designated as part of the seismic force-resisting system shall meet all of the following:

1. Bearing pads shall be provided. Direct concrete-to-concrete or concrete-to-steel contact shall not be permitted.
2. Either embedded steel bearing plates shall be provided within the supported member or embedded steel armored edges shall be provided on the supporting member.

A.8.3.3 Reinforced Masonry Special reinforced masonry shear walls shall be fully grouted.

A.8.4 Modifications to Chapter 12

A.8.4.1 Spectral Acceleration Parameters For analysis and design for functional recovery, the design spectral acceleration parameters, S_a , S_{DS} , and S_{D1} , where they occur in Chapter 12, shall be replaced by the FRE_R spectral acceleration parameters, S_{affr} , S_{DSfr} , and S_{D1fr} , respectively, as determined in accordance with Section A.6.

A.8.4.2 Ground Motions For analysis and design for functional recovery using the linear response history analysis procedure of Section 12.9.2, the target response spectrum for each spectrally matched set shall be developed in accordance with Section A.6.

A.8.4.3 Prohibited Structural Irregularities Vertical Structural Irregularity Types 1b or 4b of Table 12.3-2 shall not be permitted for buildings designed for functional recovery.

For buildings assigned to Functional Recovery Category A, Vertical Structural Irregularity Type 4a of Table 12.3-2 shall also not be permitted. Additionally, buildings assigned to Functional Recovery Category A shall not have a Torsional Irregularity Ratio (TIR) calculated in accordance with Section 12.3.2.1.1 for any story above the base which exceeds 1.4.

EXCEPTION: For buildings assigned to Functional Recovery Category A with rigid or semi-rigid diaphragms and where the maximum story drift in the direction in which the torsional irregularity is calculated, computed including the torsional amplification factor, is less than 0.1 percent, the Torsional Irregularity Ratio (TIR) shall be permitted to exceed 1.4.

A.8.4.4 Prohibited Analysis and Design Procedures The provisions of Section 12.8.1.3 and 12.14 shall not be permitted for analysis and design for functional recovery.

A.8.4.5 Design Coefficients, Factors, and Allowable Drift Limits Analysis and design for functional recovery shall be in accordance with all of the following.

1. The response modification coefficient, R , shall be replaced by the functional recovery response modification coefficient, R_{fr} , provided in Table A.8-1.

2. The functional recovery story drift shall not exceed the functional recovery allowable story drift, Δ_{afr} , provided in Table A.8-1, for any story.
3. The deflection amplification factor, C_d , shall be replaced by the functional recovery deflection amplification factor, C_{dfr} . The value of C_{dfr} shall be taken equal to R_{fr} , except that, for steel damped special moment frames, C_{dfr} shall be taken equal to $R_{fr}/1.8$.
4. The overstrength factor, Ω_0 , shall be replaced by the functional recovery overstrength factor, Ω_{0fr} . The value of Ω_{0fr} shall be taken equal to Ω_0 from Table 12.2-1 except that Ω_{0fr} shall not be taken greater than R_{fr} .
5. The redundancy factor, ρ , for analysis and design for functional recovery shall not apply.

EXCEPTIONS:

1. For seismic force-resisting systems not included in Table A.8-1, the functional recovery response modification coefficient (R_{fr}) and functional recovery allowable story drift (Δ_{afr}) shall be permitted to be determined in accordance with Section A.8.4.6.
2. For light-frame (wood) walls sheathed with wood structural panels rated for shear resistance, the functional recovery story drift shall be permitted to be computed including the lateral resistance of the following:
 - a. Light-frame (wood) walls specifically designated as part of the seismic force-resisting system.
 - b. Light-frame (wood) walls not specifically designated as part of the seismic force-resisting system provided that such walls have positive connections to the diaphragms above and below the story in which they occur and are indicated as required to achieve functional recovery on the construction documents in accordance with Section A.12.2.

Lateral resistance of sheathing materials other than wood structural panels on light-frame (wood) walls shall be permitted to be included.

Table A.8-1. Functional Recovery Design Coefficients, Factors, and Allowable Drift Limits

Seismic Force-Resisting System	Functional Recovery Category A, B and C			Functional Recovery Category D		
	R_{fr}	Δ_{afr}	$R_{\mu fr}$	R_{fr}	Δ_{afr}	$R_{\mu fr}$
BEARING WALL SYSTEMS						

Special reinforced concrete shear walls	3.0	$0.01h_{sx}$	1.1	4	$0.01h_{sx}$	1.25
Reinforced concrete ductile coupled walls	5.5	$0.0075h_{sx}$	1.45	5.5	$0.01h_{sx}$	1.45
Special reinforced masonry shear walls not exceeding 130 ft (40 m) in structural height ^a	2.5	$0.005h_s$ x	0.65	3	$0.005h_s$ x	0.65
Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	4.5	$0.01h_{sx}$	1.3	5	$0.0125h_{sx}$	1.35
BUILDING FRAME SYSTEMS						
Steel special concentrically braced frames	3.5	$0.0075h_{sx}$	1.2	4	$0.0075h_{sx}$	1.3
Special reinforced concrete shear walls	3.5	$0.01h_{sx}$	1.2	5	$0.01h_{sx}$	1.35
Reinforced concrete ductile coupled walls	5.5	$0.0075h_{sx}$	1.45	6.5	$0.01h_{sx}$	1.6
Steel and concrete composite plate shear walls	4.5	$0.01h_{sx}$	1.3	5	$0.01h_{sx}$	1.35
Special reinforced masonry shear walls not exceeding 130 ft (40 m) in structural height ^a	2.5	$0.005h_s$ x	0.65	3	$0.005h_s$ x	0.65
Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	4.5	$0.01h_{sx}$	1.3	5	$0.0125h_{sx}$	1.35
Steel buckling-restrained braced frames, non-moment-resisting beam-column connections	5.5	$0.0075h_{sx}$	1.15	6	$0.0075h_{sx}$	1.15

Steel buckling-restrained braced frames, moment-resisting beam-column connections	5.5	$0.01h_{sx}$	1.2	6.5	$0.01h_{sx}$	1.25
Steel and concrete coupled composite plate shear walls ^b	3.5	$0.0075h_{sx}$	1.2	4.5	$0.0075h_{sx}$	1.3
MOMENT-RESISTING FRAME SYSTEMS						
Steel special moment frames	5.5	$0.0125h_{sx}$	-	6.5	$0.015h_{sx}$	-
Special reinforced concrete moment frames	4.5	$0.01h_{sx}$	1.3	5.5	$0.01h_{sx}$	1.4
Special reinforced concrete moment frames constructed using precast concrete conforming to ACI 550.3	5.5	$0.01h_{sx}$	1.4	6.5	$0.0125h_{sx}$	1.55
Steel damped special moment frames	5.5	$0.0125h_{sx}$	-	6.5	$0.015h_{sx}$	-

^a Values of R_{fr} and $R_{\mu fr}$ for special reinforced masonry shear walls shall be permitted to be changed to 3.5 and 0.85, respectively, for Functional Recovery Category A, B and C and 4 and 0.9, respectively, for Functional Recovery Category D where the design shear strength in each wall pier in the direction under consideration exceeds the shear corresponding to the development of 1.25 times the nominal flexural strength of the wall pier.

^b Values of R_{fr} and $R_{\mu fr}$ for steel and concrete coupled composite plate shear walls shall be permitted to be changed to 5.5 and 1.45, respectively, for Functional Recovery Category A, B and C and 6.5 and 1.55, respectively, for Functional Recovery Category D where all coupling beam lengths in the direction under consideration are greater than or equal to 7.5 ft (2.3 m).

A.8.4.6 Quantification of Functional Recovery Seismic Performance Factors For structural systems not included in Table A.8-1, the functional recovery response modification coefficient (R_{fr}) and functional recovery allowable story drift (Δ_{afr}) shall be determined in accordance with this section.

Archetype designs shall be developed representing either a subset of or the full design space considering occupancy and use, elevation and plan configuration, structural component types, gravity load, allowable heights in Table 12.2-1, and seismic behavioral effects. Where a subset of the full design space is used, the required limitations on the structural system shall be clearly identified.

Archetype designs shall be sorted into performance groups with each performance group having no fewer than three archetype designs. All archetype designs within a performance group shall utilize the same gravity force-resisting system.

An explicit assessment procedure in accordance with Section A.17 shall be used to calculate the functional recovery times of the archetype designs.

Archetype models shall include structural elements and components representing the gravity and seismic force-resisting systems and shall be permitted to exclude nonstructural elements and components. Overstrength of the seismic force-resisting system shall be considered. Overstrength from considering the gravity system or nonstructural systems and over-stiffness of the seismic, gravity or nonstructural systems shall only be permitted where approved by the independent reviewer(s).

For use in quantifying the functional recovery seismic performance factors in accordance with this section, the FRE_R ground motion parameters S_{DSfr} and S_{D1fr} shall be taken equal to 1.25g and 0.75g, respectively.

EXCEPTION: For a structural system that is designated as Not Permitted for Seismic Design Category D, E or F, it shall be permitted to take the FRE_R ground motion parameters S_{DSfr} and S_{D1fr} as equal to 0.35g and 0.1g, respectively.

For Functional Recovery Category A, B and C, the average probability of exceeding the target recovery time for the respective Functional Recovery Category shall not exceed 10% for any performance group. For Functional Recovery Category D, the average probability of exceeding a target recovery time of 2 months shall not exceed 15% for any performance group. The average shall be calculated over all archetype designs within a given performance group. Individual archetypes shall not exceed a 20% probability of exceeding the target recovery time for Functional Recovery Category A, B and C and shall not exceed a 25% probability of exceeding a target recovery time of 2 months for Functional Recovery Category D.

For Functional Recovery Categories A, B and C, R_{fr} and Δ_{afr} shall not be taken greater than $R/1.5$ and 1.25%, respectively. For Functional Recovery Category D, R_{fr} and Δ_{afr} shall not be taken greater than $R/1.25$ and 1.75%, respectively.

For each archetype, the following two criteria shall be satisfied:

1. The ratio of the peak floor acceleration from analysis, averaged across all structural levels, to the seismic floor acceleration determined in accordance with Section A.10.7 averaged across all structural levels shall be computed. The average of this ratio across all archetypes within a performance group shall not exceed 1.0.
2. The ratio of the largest peak floor acceleration from analysis at any level to the largest seismic floor acceleration determined in accordance with Section A.10.7 at any structural

level shall be computed. The average of this ratio across all archetypes within a performance group shall not exceed 1.2.

Where the peak floor acceleration from analysis is referenced, it shall be taken as the median value from analysis. Structural levels at or below the seismic base shall be permitted to be neglected. It shall be permitted to specify a value of $R_{\mu fr}$ specific to the seismic force-resisting system in Table A.8-1 to meet the above criteria.

An independent review of the quantification of functional recovery seismic performance factors shall be performed by three or more individuals, with a minimum of one reviewer being a registered design professional and a minimum of one reviewer possessing specific experience with the proposed structural system. The review shall include, but not be limited to, all of the following:

1. The adequacy of the archetype design space and performance groups.
2. The review scope outlined in Section A.17.4.1, except for Item 2 and items related to nonstructural systems.

A.8.4.7 Equivalent Lateral Force Procedure Lower Bound C_s For analysis and design for functional recovery, Equation (12.8-7) shall not apply.

A.8.4.8 Alternate Design Provisions for Diaphragms, Including Chords and Collectors Where the alternate design provisions of Section 12.10.2 for diaphragms, including chords and collectors, are used in the analysis and design for functional recovery, the value of R_s shall not be taken as greater than the least value from Table 12.10-1 and R_{fr} from Table A.8-1.

A.8.4.9 Diaphragm Design Provisions for One-Story Structures with Flexible Diaphragms and Rigid Vertical Elements Design of diaphragms, chords, and collectors for functional recovery in one-story structures with flexible diaphragms and rigid vertical elements meeting the limitations of Section A.8.4.9.1 shall be in accordance with Section A.8.4.9.2 and either Section A.8.4.9.3 or A.8.4.9.4.

A.8.4.9.1 Limitations The limitations of Section 12.10.3.1 shall apply except that, for Section 12.10.3.1 Item 7, the vertical elements of the seismic force-resisting system shall be designed in accordance with Section A.8.4.5.

EXCEPTION: For seismic force-resisting systems permitted by Section 12.10.3.1 Item 6 and not included in Table A.8-1, it shall be permitted to design the vertical elements of the seismic force-resisting system for the diaphragm seismic design force of Sections A.8.4.9.3 or A.8.4.9.4 using the seismic load effects, including the overstrength factor of Section 12.4.3.1. The functional recovery diaphragm overstrength factor, $\Omega_{0-diaphfr}$, shall be used in lieu of the overstrength factor, Ω_o .

A.8.4.9.2 Functional Recovery Story Drift The functional recovery story drift considering the contribution of diaphragm deformation shall not exceed the functional recovery allowable story drift, Δ_{afr} , provided in Table A.8-2, at any point on the diaphragm. The functional recovery story drift shall be computed in accordance with Equation (A.8-1).

$$\Delta = \frac{C_{d-diaphfr} (\delta_e + \delta_{di})}{h_n} \quad (A.8-1)$$

where $C_{d-diaphfr}$ is the functional recovery deflection amplification factor for diaphragm deflection taken equal to $R_{diaphfr}$ and δ_e and δ_{di} are the displacements of the vertical lateral force-resisting and the displacements due to diaphragm deformation, respectively, corresponding to the forces of Section A.8.4.9.3 or A.8.4.9.4.

In addition to the requirements above, the functional recovery story drift shall be checked in accordance with Section A.8.4.5 Item 2 neglecting the contribution of diaphragm deformation.

EXCEPTION: For seismic force-resisting systems permitted by Section 12.10.3.1 Item 6 and not included in Table A.8-1, it shall be permitted to use C_{dfr}/R_{fr} equal to 1.0 and $\Delta_{afr} = 0.005h_{sx}$ for the purposes of checking functional recovery story drift in accordance with Section A.8.4.5.

A.8.4.9.3 Standard Diaphragm Design Provisions Design of diaphragms, chords and collectors for functional recovery shall be in accordance with Section 12.10.1 with the following modifications:

1. The response modification coefficient, R , shall be replaced by the functional recovery response modification coefficient for design of diaphragms, $R_{diaphfr}$, provided in Table A.8-2.
2. The overstrength factor, Ω_0 , shall be replaced by the functional recovery diaphragm overstrength factor, $\Omega_{0-diaphfr}$, provided in Table A.8-2.

A.8.4.9.4 Alternative Diaphragm Design Provisions Design of diaphragms, chords and collectors for functional recovery shall be in accordance with Section 12.10.3 with the following modifications:

1. The response modification coefficient for design of diaphragms, R_{diaph} , shall be replaced by the functional recovery response modification coefficient for design of diaphragms, $R_{diaphfr}$, provided in Table A.8-2.
2. The diaphragm overstrength factor, $\Omega_{0-diaph}$, shall be replaced by the functional recovery diaphragm overstrength factor, $\Omega_{0-diaphfr}$, provided in Table A.8-2.

Table A.8-2. Functional Recovery Seismic Performance Factors for Diaphragms of One-Story Structures with Flexible Diaphragms and Rigid Vertical Elements

Diaphragm Type	Functional Recovery Category A, B and C				Functional Recovery Category D			
	$R_{diaphfr}$	$\Omega_{0-diaphfr}$	Δ_{afr}	$R_{\mu fr}$	$R_{diaphfr}$	$\Omega_{0-diaphfr}$	Δ_{afr}	$R_{\mu fr}$
Wood structural panel diaphragms	3.0	2.0	$0.0125h_{sx}$	$\frac{1.2}{5}$	3.5	2.0	$0.0175h_{sx}$	$\frac{1.3}{5}$

Bare steel deck diaphragms that meet the special seismic detailing requirements of AISI S400

3.0	2.0	$0.0125h_{sx}$	$\frac{1.2}{5}$	3.5	2.0	$0.0175h_{sx}$	$\frac{1.3}{5}$
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A.8.4.10 Deformation Compatibility Every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and the seismic forces resulting from the functional recovery story drifts.

A.8.4.11 Foundation and Superstructure-to-Foundation Connection Design Forces Foundations and the connection of the superstructure elements to the foundation shall be analyzed and designed for functional recovery for, in addition to the gravity loads, the least of all of the following:

1. Seismic load effects, including overstrength of Section 12.4.3.1, using the functional recovery seismic load effects and the functional recovery overstrength factor of this appendix.
2. Capacity-limited seismic load effects of Section 12.4.3.2.
3. Seismic load effects determined from a nonlinear response history analysis for the MCE_R and performed in accordance with Chapter 16.

EXCEPTION: Forces resulting from the above cases shall be permitted to be neglected for functional recovery for soil actions, geotechnical limit states, overturning stability, sliding, flexure in the foundation, and vertical reinforcing crossing the foundation-superstructure interface in walls and columns. Seismic load effects without overstrength for these actions and limit states shall still be required.

A.8.5 Modifications to Chapter 16

A.8.5.1 Ground Motions For analysis and design for functional recovery, the provisions of Section A.6.4 shall apply.

A.8.5.2 Unacceptable Response For analysis and the design for functional recovery, the provisions of Section 16.4.1.1 shall apply, except that no unacceptable response shall be permitted without exception.

A.8.5.3 Minimum Design Strength For analysis and design for functional recovery, Section 16.4.1.2 shall be replaced by the following: The mean of the maximum base shear in each direction shall not be less than the design base shear required by Section A.8.4.

A.8.5.4 Transient Story Drift For analysis and design for functional recovery, Section 16.4.1.3 shall be replaced by the following: The functional recovery mean transient story drift shall not exceed the functional recovery allowable story drift, Δ_{afr} , as obtained from Table A.8-1. The transient story drift

shall be computed as the absolute value of the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure within a single response history analysis.

EXCEPTIONS:

1. For seismic force-resisting systems not included in Table A.8-1, the functional recovery allowable story drift (Δ_{afr}) shall be permitted to be determined in accordance with Section A.8.4.6.
2. For light-frame (wood) walls sheathed with wood structural panels rated for shear resistance, the functional recovery story drift shall be permitted to be computed in a separate analysis meeting all of the requirements of Section A.8.4.5 Exception 2.

A.8.5.5 Residual Story Drift For analysis and design for functional recovery, Section 16.4.1.4 shall be replaced by the following: For structures assigned to Functional Recovery Category A, B or C, the functional recovery mean residual story drift shall not exceed $0.003h_{sx}$, where residual story drift is taken as the maximum value of story drift in a structure at rest, following response to an earthquake motion.

A.8.5.6 Force-Controlled Actions For analysis and design for functional recovery, Section 16.4.2.1 shall not apply.

A.8.5.7 Deformation-Controlled Actions For analysis and design for functional recovery, Section 16.4.2.2 shall be replaced by the following: For structures assigned to Functional Recovery Category A, B or C, demands on deformation-controlled actions shall not exceed the Immediate Occupancy nonlinear acceptance criteria as specified by ASCE 41. For structures assigned to Functional Recovery Category D, demands on deformation-controlled actions shall not exceed the Damage Control nonlinear acceptance criteria as specified by ASCE 41. Demands shall be taken as the mean value of the response parameter of interest obtained from the suite of nonlinear response history analyses.

A.8.5.8 Elements of the Gravity Force-Resisting System For analysis and design for functional recovery, Section 16.4.2.3 shall be replaced by the following: every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and the seismic forces resulting from the functional recovery mean building displacements from the suite of nonlinear response history analyses.

A.8.6 Modifications to Chapter 17

A.8.6.1 General Seismically isolated buildings assigned to Functional Recovery Category A, B or C shall meet the requirements of this section.

A.8.6.2 Drift Limits When performing analysis and design in accordance with Chapter 17 for the MCE_R , the allowable story drift limit in accordance with Section 17.5.6 or 17.6.4.4 shall not be taken as greater than $0.015h_{sx}$.

A.9 ANALYSIS AND DESIGN OF STRUCTURAL SYSTEMS FOR NONBUILDING STRUCTURES

A.9.1 General The structural analysis and design of elements and components for functional recovery for nonbuilding structures shall be in accordance with this section and one of the following:

1. Where the basic lateral and vertical seismic force-resisting system conforms to one of the types or a combination of types permitted in Section 15.4, analysis and design for functional recovery shall be in accordance with the provisions of Chapter 15 as modified by Sections A.9.2 and A.9.3. The analysis and design for functional recovery shall be performed in addition to, not in lieu of, the unmodified requirements of Chapter 15.
2. For a seismically isolated nonbuilding structure, see Section A.1.2.2.

A.9.2 Seismic Importance Factor For structural analysis and design for functional recovery, the seismic importance factor, I_e , shall not apply.

A.9.3 Modifications to Chapters 12 and 15

A.9.3.1 Spectral Acceleration Parameters For analysis and design for functional recovery, the design spectral acceleration parameters, S_a , S_{DS} , and S_{D1} , where they occur in Chapter 15, shall be replaced by the FRE_R spectral acceleration parameters, S_{afr} , S_{DSfr} , and S_{D1fr} , respectively, as determined in accordance with Section A.6.

A.9.3.2 Ground Motions For analysis and design for functional recovery using the linear response history analysis procedure of Section 12.9.2, the target response spectrum for each spectrally matched set shall be developed in accordance with Section A.6.

A.9.3.3 Prohibited Vertical Structural Irregularities Nonbuilding structures similar to buildings designed for functional recovery shall not have Vertical Structural Irregularity Types 1b or 4b of Table 12.3-2.

EXCEPTION: Where linear analysis demonstrates that the design strength of weak story elements in Vertical Strength Irregularity Type 4b is greater than the $R_{fr}=1$ forces.

A.9.3.4 Design Coefficients, Factors and Drift Limits Analysis and design for functional recovery shall be in accordance with all of the following:

1. The response modification coefficient, R , shall be replaced by the functional recovery response modification coefficient, R_{fr} , provided in Table A.9-1.

2. Based on a rational analysis, the story drift of nonbuilding structures designed for functional recovery shall not exceed the story drift that would adversely affect the functionality of the attached or interconnected components and elements such as walkways and piping. Additionally, the drift limits for nonbuilding structures similar to buildings shall not exceed the following maximum limits:
 - a. For nonbuilding structures similar to buildings with R values less than or equal to 4, the allowable story drift limits of Table 12.12-1 for Risk Category IV shall apply.
 - b. For nonbuilding structures similar to buildings with R values greater than 4, the allowable drift limits of Table A.8-1 for Functional Recovery Category B shall apply.
3. The deflection amplification factor, C_d , shall be replaced by the functional recovery deflection amplification factor, C_{dfr} . The value of C_{dfr} shall be taken equal to R_{fr} .
4. The overstrength factor, Ω_0 , shall be replaced by the functional recovery overstrength factor, Ω_{0fr} , provided in Table A.9-1.
5. The redundancy factor, ρ , for analysis and design for functional recovery shall not apply.

Table A.9-1. Functional Recovery Response Modification Coefficients and Overstrength Factors

Response Modification Coefficient, R , from Tables 15.4-1 or 15.4-2	R_{fr}	Ω_{0fr}
$R > 4$	See A.8.4.5	See A.8.4.5
$R = 4$	2.5	2.0
$R = 3.5$	2.25	1.75
$R = 3.25$	2.25	1.75
$R = 3$	2	1.75
$R = 2.5$	1.5	1.5
$R = 2$	1.25	1.25
$R = 1.5$	1.0	1.0
$R = 1.25$	1.0	1.0
$R \leq 1$	R	1.0

A.9.3.5 Deformation Compatibility Every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and the seismic forces resulting from the functional recovery story drifts.

A.9.3.6 Foundation Design Forces Foundations shall be analyzed and designed for functional recovery for, in addition to the gravity loads, the least of all of the following:

1. Seismic load effects, including overstrength of Section 12.4.3.1.
2. Capacity-limited seismic load effects of Section 12.4.3.2.
3. Seismic load effects determined from a nonlinear response history analysis for the MCE_R and performed in accordance with Chapter 16.

EXCEPTION: Forces resulting from the above cases need not be considered for functional recovery for soil actions, geotechnical limit states, overturning stability, sliding, and flexure in the foundation. Seismic load effects without overstrength shall still be required.

A.9.3.7 Nonbuilding Structure Anchorage For the anchorage in concrete foundations of all nonbuilding structures designed for functional recovery, all of the following special detailing requirements shall apply:

- a) Anchorage shall be in accordance with Section 15.4.9.1, whereby the anchor embedment into the concrete foundation shall be designed to develop the steel strength of the anchor in tension in accordance with ACI 318 Section, 17.10.5.3(a), as given by ACI 318, Equation (17.6.1.2), or shall be designed using anchor reinforcement in accordance with ACI 318, Section 17.4.2.9, to develop the steel strength of the anchor in tension per ACI 318, Equation (17.6.1.2).
- b) The minimum gauge length of the anchor rod, which is the length over which its elongation can occur, shall be at least eight times the rod diameter.
- c) Post-installed anchors are permitted to be used in accordance with Section 15.4.9.3, provided the anchor embedment into the concrete foundation is sufficient to develop the steel strength of the anchor rod in tension.
- d) Where the special detailing requirements of this section apply, the load combinations of Section 12.4.3 that include overstrength shall not apply.

Anchors (bolts or rods) embedded in concrete foundations shall not be designed to develop forces calculated using the seismic load effects, including overstrength of Section 12.4.3. Connections of anchors to seismic force-resisting elements shall be designed to develop forces calculated using the seismic load effects including overstrength of Section 12.4.3. In addition, the connection of the anchors to a tank or vessel shall be designed to develop the lesser of the strength of the anchor in tension as determined by API 650 or AWWA D100 or forces calculated using the seismic load effects including overstrength of Section 12.4.3. The overstrength requirements of Section 12.4.3 and the Ω_{Ofr} values tabulated in Table A.9-1 shall not apply to the design of walls, including interior walls, of tanks or vessels.

A.9.3.8 Additional Requirements for Tanks and Vessels

A.9.3.8.1 Convective Force The 1.5 value located in the denominator of Equation (15.7-6) shall be replaced with 1.0.

A.9.3.8.2 Flexibility of Piping Attachments All tanks and vessels designed for functional recovery shall comply with the requirements of Section 15.7.4 with the value of C_d set equal to C_{dfr} .

A.9.3.8.3 Sloshing All tanks and vessels shall be provided with seismic freeboard. Seismic freeboard shall be computed as the height of sloshing liquid, δ_{sfr} , above the product design height using Equation (A.18-1) and not reduced based on risk category:

$$\delta_{sfr} = 0.42 D_i S_{acfr}$$

For cylindrical tanks, D_i shall be the inside diameter of the tank; for rectangular tanks, D_i shall be replaced by the longitudinal plan dimension of the tank, L , for the direction under consideration. S_{acfr} is the spectral acceleration of the sloshing liquid and computed using Equations (15.7-10) or (15.7-11) with the appropriate FRE_R spectral acceleration parameters S_{afr} , S_{DSfr} , and S_{D1fr} computed at the period T_c .

A.9.3.8.4 Evaluation of Structures Sensitive to Buckling Failure All tanks and vessels designed for functional recovery supported by a pedestal or skirt shall comply with the requirements of Section 15.7.10.5 a and b, regardless of risk category.

A.10 ANALYSIS AND DESIGN OF NONSTRUCTURAL SYSTEMS

A.10.1 General All nonstructural elements, components, equipment, and distribution systems required by Section A.4 shall be analyzed and designed for functional recovery in accordance with the provisions of Chapter 13 as modified by this section. The analysis and design for functional recovery in accordance with this section shall be performed in addition to, not in lieu of, the unmodified requirements of Chapter 13.

A.10.2 Exemptions Section 13.1.4 exemptions shall not apply to this appendix except as listed below. In addition to the exemptions in accordance with Section A.4.3, the nonstructural elements, components, equipment, and distribution systems exempt from the requirements of this section shall include the following:

1. Discrete nonstructural elements, components, or equipment that are positively attached to the structure and weigh 20 pounds (9 kg) or less.
2. Discrete nonstructural elements, components, or equipment that meet all of the following:

- a. Are positively attached to the structure and where such anchorage is shown on the construction documents.
 - b. Where flexible connections are provided between the element, component, or equipment and the associated distribution systems, if applicable, and where such flexible connections are shown on the construction documents.
 - c. Weigh 400 pounds (18.4 kg) or less, and the center of mass is located 4 feet (1.2 m) or less above the adjacent floor or roof level that directly supports the element, component, or equipment.
3. Nonstructural distribution systems that do not cross a seismic separation joint, are positively attached to the structure, and weigh 5 pounds per foot (73 N/m) or less.
4. Cable tray and raceways that do not cross a seismic separation joint, that are positively attached to the structure and where such anchorage is shown on the construction documents, where flexible connections are provided between the cable tray or raceway and the associated discrete element, component or equipment, if applicable, and where such flexible connections are shown on the construction documents, and where one of the following apply:
 - a. Trapeze assemblies with 3/8 inch (10 mm) or 1/2 inch (13 mm) diameter rod hangers not exceeding 12 inches (305 mm) in length from the cable tray or raceway support point to the connection at the supporting structure are used to support the cable tray or raceway, and the total weight supported by any single trapeze is 100 pounds (445 N) or less.
 - b. The cable tray or raceway is supported by individual rod hangers 3/8 inch (10 mm) or 1/2 inch (13 mm) in diameter, and each hanger in the cable tray or raceway run is 12 inches (305 mm) or less in length from the cable tray or raceway support point connection to the supporting structure, and the total weight supported by any single rod is 50 pounds (220 N) or less.
5. Conduit less than 2.5 in (64 mm) trade size that does not cross a seismic separation joint and is positively attached to the structure.
6. Duct systems not designed to carry toxic, highly toxic, or flammable gases and not used for smoke control, that do not cross a seismic separation joint, that are positively attached to the structure and where such anchorage is shown on the construction documents, where flexible connections are provided between the duct and the associated discrete element, component or equipment, if applicable, and where such flexible connections are shown on the construction documents, and where one of the following apply:
 - a. Trapeze assemblies with 3/8 inch (10 mm) or 1/2 inch (13 mm) diameter rod hangers not exceeding 12 inches (305 mm) in length from the duct support point to the

connection at the supporting structure are used to support the duct, and the total weight supported by any single trapeze is less than 10 lb/ft (146 N/m) and 100 pounds (445 N) or less.

- b. The duct is supported by individual rod hangers 3/8 inch (10 mm) or 1/2 inch (13 mm) in diameter and each hanger in the duct run is 12 inches (305 mm) or less in length from the duct support point connection to the supporting structure, and the total weight supported by any single rod is 50 pounds (220 N) or less.
7. Duct systems not designed to carry toxic, highly toxic, or flammable gases and not used for smoke control, that do not cross a seismic separation joint, that are positively attached to the structure and where such anchorage is shown on the construction documents, where provisions are made to avoid impact with other ducts or mechanical components or to protect ducts in the event of such impact, and where ducts have a cross-sectional area of less than 6 square feet (0.557 m²) and weigh 20 lb/ft (292 N/m) or less.
8. Components of duct systems that are installed in line, have an operating weight of 75 pounds (334 N) or less, and are positively attached with mechanical fasteners to the duct on both sides.
9. Piping and tubing systems that do not cross a seismic separation joint, that are positively attached to the structure and where such anchorage is shown on the construction documents, where flexible connections are provided between the piping or tubing system and the associated discrete element, component or equipment, if applicable, and where such flexible connections are shown on the construction documents, and where one of the following apply:
 - a. Trapeze assemblies are supported by 3/8 inch (10 mm) or 1/2 inch (13 mm) diameter rod hangers not exceeding 12 inches (305 mm) in length from the pipe support point to the connection at the supporting structure, no single pipe diameter exceeds 2 inches (50 mm), and the total weight supported by any single trapeze is 100 pounds (445 N) or less.
 - b. Piping with nominal pipe size 1 inch (25 mm) or less that has an R_{po} in Table 13.6-1 of 2 or greater supported by rod hangers and provisions are made to avoid impact with other structural or nonstructural components or to protect the piping in the event of such impact.
 - c. Pneumatic tube systems supported by individual rod hangers 3/8 inch (10 mm) or 1/2 inch (13 mm) in diameter, and each hanger in the run is 12 inches (305 mm) or less in length from the tube support point to the connection at the supporting structure, and the total weight supported by any single rod is 50 pounds (220 N) or less.

EXCEPTION: Flexible connections shall not be required where the piping or tubing system is rigidly attached to the same floor or wall that provides vertical and lateral support for the discrete element, component, or equipment, or a fixture.

A.10.3 Seismic Functional Recovery Component Importance Factor For analysis and design for functional recovery, the component importance factor, I_p , where it occurs in Chapter 13, shall be replaced by the seismic functional recovery component importance factor, I_{pfr} . Elements, components, equipment, and distribution systems shall be assigned a seismic functional recovery component importance factor, I_{pfr} , in accordance with Table A.10-1. For analysis and design for functional recovery, the seismic importance factor, I_e , shall be taken as 1.0.

Table A.10-1. Functional Recovery Nonstructural Component Importance Factors by Functional Recovery Category for Earthquake Loads

Functional Recovery Category	Seismic Functional Recovery Component Importance Factor, I_{pfr}
A and B	1.75
C	1.5
D	1.25

A.10.4 Spectral Acceleration Parameters For analysis and design for functional recovery, the design spectral acceleration parameters, S_a , S_{DS} , and S_{D1} , where they occur in Chapter 13, shall be replaced by the FRE_R spectral acceleration parameters, S_{afr} , S_{DSfr} , and S_{D1fr} , respectively, as determined in accordance with Section A.6.

A.10.5 Seismic Design Forces Seismic design forces on nonstructural systems for functional recovery shall be computed in accordance with Section 13.3.1 as modified by this appendix.

EXCEPTION: Seismic design forces for functional recovery shall be permitted to be taken no greater than the seismic design forces of Section 13.3.1 without considering the modifications in this appendix but using $I_p = 1.5$. For the purpose of this exception, I_p shall be taken as 1.5 regardless of the assignment of Risk Category.

A.10.6 Structure Ductility Reduction Factor For analysis and design for functional recovery, the functional recovery structure ductility reduction factor, $R_{\mu fr}$, shall be taken in accordance with Table A.8-1 or A.8-2.

EXCEPTION: Where a value of $R_{\mu fr}$ is not provided in Table A.8-1 or for nonbuilding structures, it shall be permitted to calculate $R_{\mu fr}$ in accordance with Section 13.3.1.2 with all of the following modifications:

1. In lieu of R and Ω_0 , R_{fr} and Ω_{ofr} in accordance with Section A.8.4.6 or A.9.3.4 shall be used.
2. The value of I_e shall be taken as 1.0.
3. The lower limit on $R_{\mu fr}$ of 1.3 in Equation 13.3-6 shall not apply.

A.10.7 Piping Spanning Between Points of Lateral Restraint All piping in structures assigned to Functional Recovery Category A and B and piping carrying hazardous substances in structures assigned to any Functional Recovery Category shall be capable of spanning laterally between points of lateral restraint, as well as in any lateral cantilever conditions, considering forces in combination with displacements, where displacements are determined in accordance with Section A.10.10. Such piping shall be deemed to comply without further analysis where it meets all of the following:

1. The seismic floor acceleration of Section A.10.8 shall not be greater than the seismic floor acceleration threshold of Table A.10-2
2. For the span under consideration, the points of lateral restraint at each end shall connect to the same diaphragm.

**Table A.10-2. Seismic Floor Acceleration
Threshold for Piping in Functional Recovery
Category A and B**

Component or Distribution System	Seismic Floor Acceleration Threshold (g)
MECHANICAL AND PLUMBING	
Water or Steam Piping (threaded steel, diameter $\leq$ 2.5")	0.3
Water or Steam Piping (welded steel, diameter $>$ 2.5")	1.2
Sanitary Waste Piping (cast iron with bell and spigot)	0.9

FIRE PROTECTION

Fire Sprinkler Piping (mechanical grooved couplings)	1.1
Fire Sprinkler Drop	0.7

A.10.8 Seismic Floor Acceleration The seismic floor acceleration for functional recovery at the point of attachment of the element, component, equipment, and distribution system shall be taken as F_p/W_p where F_p is calculated in accordance with Equation (13.3-1) with all of the following modifications:

1. The ratio C_{AR}/R_{p0} shall be taken as 1.0.
2. The component importance factor, I_p , shall be taken as 1.0 in lieu of substituting I_{pfr} for I_p .
3. The calculation of R_μ shall be in accordance with Section A.10.6.

EXCEPTION: Where nonlinear response history analysis for functional recovery is used in accordance with Section A.8.5, the seismic floor acceleration for functional recovery shall be permitted to be taken as a_i in accordance with Section 13.3.1.5 as modified by Section A.10.9.

A.10.9 Nonlinear Response History Analysis Where nonlinear response history analysis for functional recovery is used in accordance with Section A.8.5, a_i in Equation (13.3-7) shall be taken as the maximum acceleration at level i obtained from nonlinear response history analysis at the FRE_R ground motion.

A.10.10 Seismic Relative Displacements Seismic relative displacements for analysis and design for functional recovery shall be computed in accordance with Section 13.3.2 with the following modifications:

1. The seismic importance factor, I_e , shall be replaced by the seismic functional recovery component importance factor, I_{pfr} , in Equation (13.3-8). In all other calculations, the seismic importance factor, I_e , shall be taken as 1.0.
2. The deflections of the structure shall be computed in accordance with Section A.8 or A.9.
3. Where Table A.8-1 applies for the seismic force-resisting system, the allowable story drift shall be replaced by the functional recovery allowable story drift taken in accordance with Table A.8-1.

A.10.11 Architectural Components The requirements of this section shall apply to all architectural elements, components, equipment, and distribution systems required to be considered by Section A.4.

A.10.11.1 Suspended Ceilings Design of suspended ceilings shall address, at a minimum, suspended ceiling support, bracing, edge conditions, splices, vertical offsets, and interfaces between dissimilar ceiling types. Additionally, the following modifications shall be made to ASTM E580:

1. *Exitways.* Lay-in ceiling assemblies in exitways shall be installed with a main runner or cross runner surrounding all sides of each piece of tile, board or panel and each light fixture or grille. A cross runner that supports another cross runner shall be considered a main runner for structural classification. Splices or intersections of runners shall be attached through connectors such as pop rivets, screws, pins, plates with end tabs, or other connectors. Lateral force diagonal bracing may be omitted in the short or transverse direction of exitways, not exceeding 8 feet (2.4 m) wide, when perimeter support in accordance with ASTM E580 Sections 5.2.2 and 5.2.3 is provided, and the perimeter wall laterally supporting the ceiling in the short or transverse direction is designed to carry the ceiling lateral forces. The connections between the ceiling grid, wall angle, and the wall shall be designed to resist the ceiling lateral forces.
2. *Corridors and lobbies.* Expansion joints shall be provided in the ceiling at intersections of corridors and at junctions of corridors and lobbies of other similar areas.
3. *Lay-in panels.* Metal panels and panels weighing more than $\frac{1}{2}$ psf (24 N/mm²) other than acoustical tiles shall be positively attached to the ceiling suspension runners.
4. *Lateral force bracing.* Lateral force bracing is required for all ceiling areas except that they shall be permitted to be omitted in rooms with floor areas up to 144 ft² (13.4 m²) when perimeter support in accordance with ASTM E580 Sections 5.2.2 and 5.2.3 are provided and perimeter walls are designed to carry the ceiling lateral forces. The connections between the ceiling grid, wall angle, and the wall shall be designed to resist the ceiling lateral forces. Horizontal restraint spacing shall be justified by analysis or testing, except that the first horizontal restraint shall be no greater than 6 feet (1.8 m) from the perimeter support in each orthogonal direction and horizontal restraint spacing shall not exceed 12 feet (3.7 m) on center. Bracing wires shall be secured with four tight twists in 1 $\frac{1}{2}$ inches, or an alternate connection.
5. Ceiling support and bracing wires shall be spaced at a minimum of 6 inches (15 cm) from all pipes, ducts, conduits, and equipment that are not braced for horizontal forces. Where sway bracing is provided for such pipes, ducts, conduits, and equipment, the minimum spacing shall be permitted to be determined from rational analysis or in-situ validation and shall be permitted to be taken less than 6 inches (15 cm).

For structures assigned to Functional Recovery Category A and B, lighting fixtures and other suspended items weighing more than 20 lbs (89 N) shall be designed to prevent damage, which could affect basic intended function to fixtures, suspended items, or ceiling caused by relative movement.

A.10.11.2 Partitions Connection strength at the top and bottom of partition walls shall be sufficient to resist an allowable stress demand corresponding to the greater of 10psf (0.48 kN/m²) applied uniformly over the wall area out-of-plane and the seismic demands imposed by the wall and all connected items.

For structures assigned to Functional Recovery Category A or B, the exception of Section 13.5.8.1 shall not be permitted.

A.10.11.3 Adhered Veneer For structures assigned to Functional Recovery Category A and B, adhered veneer shall not be permitted in hallways, corridors, stairs, lobbies, over building exits, or in other locations where fallen veneer would obstruct exiting.

EXCEPTION: Adhered veneer shall be permitted where the functional recovery story drift does not exceed 0.3% or where the backup walls are isolated in the in-plane direction from the functional recovery story drift. Partitions supporting adhered veneer shall be designed for a maximum of $L/600$ deflection under seismic loading.

A.10.11.4 Window Wall, Curtain Wall, and Storefront Systems For structures assigned to Functional Recovery Category A and B, seismic tests in accordance with AAMA 501.4 shall be performed. The elastic design displacement of AAMA 501.4 shall be taken as I_{pfr} times the functional recovery story drift. At a minimum, the pass/fail criteria from AAMA 501.4 Section 11 for the elastic design displacement test, including air infiltration and water penetration resistance, shall apply.

A.10.12 Mechanical, Electrical, Plumbing and Fire Protection Components The requirements of this section shall apply to all mechanical, electrical, plumbing and fire protection elements, components, equipment, and distribution systems required to be considered by Section A.4.

A.10.12.1 Distribution Systems Distribution system bracing design and connections of distribution systems to equipment shall consider the required forces including those corresponding to displacements in accordance with Section A.10.10 and shall explicitly consider differential building movement, and movement between components, equipment, and distribution systems. Distribution systems, including those in rooftop conditions, shall be positively attached at all lateral restraints. Flexible connections or flexible configurations of distribution systems shall be provided, at a minimum, at all of the following conditions:

1. For vertical riser pipes spanning between structural levels, oversized holes at each structural level, with connections detailed to accommodate the seismic rotations resulting from seismic relative displacements and/or flexible connections or flexible configurations of distribution, systems shall be provided.

2. For pipes, ducts with cross sectional area $\geq 6 \text{ ft}^2$ (0.55 m^2), and conduit of 2.5 in (64 mm) or greater trade size crossing seismic separation joints, flexible connections or flexible configurations of distribution systems shall be provided at the location of the seismic separation joint.
3. For pipes, ducts with cross sectional area $\geq 6 \text{ ft}^2$ (0.55 m^2), and conduit of 2.5 in (64 mm) or greater trade size with lateral restraint at a structural level different than the component or equipment to which the pipe, duct or conduit attaches, flexible connection(s) or flexible configuration(s) of distribution systems shall be provided between the point of attachment to the component or equipment and the first lateral restraint point at the other structural level.

Where provided, flexible connections and/or flexible configurations of distribution systems shall be detailed to accommodate the seismic relative displacements and rotations.

A.10.12.2 Fire Sprinkler Drops Flexible fire sprinkler drops shall be required where sprinkler heads are located in suspended acoustic tile ceilings or other suspended ceilings with wire bracing.

A.10.12.3 Elevators

Retainer plates are required at the top and bottom of the car and counterweight, except where safety devices are provided which meet all requirements of the retainer plates, including full engagement of the machined portion of the rail.

The design of the car, cab stabilizers, counterweight guide rails, counterweight frames, guide rail support bracket fastenings, and supporting structural framing for seismic forces shall be based on the following requirements:

1. For structures in Seismic Design Category D, E or F, the seismic force shall not be taken less than 0.5g allowable stress design load.
2. The component weight, W_p , shall equal the weight of the counterweight or the maximum weight of the car plus not less than 40 percent of its rated load.
3. For the design of guide rail support bracket fastenings and the supporting structural framing, the seismic forces shall be assumed to be distributed one-third to the top guiding members and two-thirds to the bottom guiding members of cars and counterweights, unless other substantiating data are provided.
4. With the car or counterweight located in the most adverse position, the stress in the rail shall not exceed the limitations in this standard, nor shall the deflection of the rail relative to its supports exceed the deflection listed in Table A.10-3.
5. Where guide rails are continuous over supports and rail joints are within 2 ft (610 mm) of their supporting brackets, a simple span shall be permitted to be assumed.

6. The use of spreader brackets shall be permitted.
7. For structures in Seismic Design Category D, E or F, cab stabilizers and counterweight frames shall be designed to withstand computed lateral load with a minimum horizontal acceleration of 0.5g allowable stress design load.

For hydraulic elevators, reference to requirements for counterweights above shall not apply.

For structures assigned to Functional Recovery Category A or B, traction elevators shall meet all of the following additional requirements:

1. For elevators operating with a speed of 150 ft/min (46 m/min) or greater, a seismic switch, as required by ASME A17.1, shall be provided and connected to the essential electrical system.
2. Where a seismic switch is present, a visible or audible annunciator shall be connected to the essential electrical system and located in the elevator machine room. The annunciator shall indicate if the seismic switch is inoperative due to a loss of power. If a visual annunciator is used, it shall be clearly visible in the room.
3. After a seismic switch has been triggered, the elevator shall have the ability to operate at a “go slow” speed until the elevator can be inspected. “Go slow” speed is defined as a travel speed of not more than 150 ft/min (46 m/min).
4. A sensor shall be installed on the governor rope/sheave. The sensor shall prevent cab movement when the governor tail sheave is dislodged from its normal position.

Table A.10-3. Elevator Guide Rail Allowable Deflection

Rail Size (weight per foot of length, lbs)	Width of Machined Surface (inches)	Allowable Rail Deflection (inches)
8	1 ¼	0.20
11	1 ½	0.30
12	1 ¾	0.40
15	1 31/32	0.50
18 ½	1 31/32	0.50
22 ½	2	0.50
30	2 ¼	0.50

A.11 SEISMIC CERTIFICATION REQUIREMENTS EQUIPMENT AND COMPONENTS

A.11.1 General For structures designed for functional recovery, equipment and/or components of that equipment required to support the basic intended function of the structure shall be seismically certified by the manufacturer as operable and, when containing hazardous substances, shall be certified by the manufacturer as maintaining containment when subjected to an acceleration at the point of attachment to the structure not less than the seismic floor acceleration of Section A.10.8 multiplied by I_{qualfr} in accordance with Table A.11-1. Seismic certification of equipment and/or components of that equipment in accordance with this section shall be in addition to, not in lieu of, the unmodified requirements of Chapter 13.

EXCEPTIONS:

1. Rugged components as defined in Section 11.2, with the design earthquake level replaced by the functional recovery earthquake level.
2. Components and equipment weighing less than or equal to 50 lbs (23 kg) supported directly on the structure or surface-mounted on equipment or components that are not required to have seismic certification by this section.
3. Mobile units and equipment, including components that are moved from one point in the structure to another during ordinary use.
4. Distribution systems, excluding in-line equipment and components.
5. Underground tanks.
6. Electric motors, base-mounted horizontal pumps, and compressors.
7. Base-mounted vertical pumps up to 20 hp.
8. Emergency generators up to 25 kilowatts.
9. Elevator cabs, machines, and governors.
10. For structures assigned to Functional Recovery Categories C or D, components and equipment without hazardous substances are not required to be seismically certified where the required seismic floor acceleration of Section A.10.8 is not greater than the seismic certification floor acceleration threshold in Table A.11-2.
11. For structures assigned to Functional Recovery Categories C or D, components and equipment are not required to be seismically certified where the requirements of Section A.17 are satisfied.

A.11.2 Seismic Certification Methods Equipment and/or components of that equipment shall be seismically certified by one of the following methods, except that active parts or energized components shall be certified exclusively based on shake-table testing:

1. Shake-table testing in accordance with Section 13.2.6. Where shake-table testing is used to seismically certify equipment, the connection of equipment to the structure shall be similar to the connection used in the shake-table test.
2. Experience data in accordance with Section 13.2.7
3. For nonactive components only, analysis of the forces determined in accordance with Section 13.3 as modified by Section A.10. The values of C_{AR} and R_{po} shall be taken for the equipment of which the component is part.

A.11.2.1 Seismic Certification by ICC-ES AC156 Shake-Table Testing Where seismic certification is demonstrated by shake-table testing in accordance with Section A.11.2 Item 1 and ICC-ES AC156 is used, the required response spectrum for ICC-ES AC156 shall be taken in accordance with the following:

1. The parameters A_{FLX-H} and A_{RIG-H} shall be taken as the seismic floor acceleration of Section A.10.8 multiplied by $2.5 \cdot I_{qualfr}$ and $1.0 \cdot I_{qualfr}$, respectively, except that A_{FLX-H} shall be permitted to not exceed $1.6 \cdot I_{qualfr} \cdot S_{DSfr}$.
2. The parameters A_{FLX-V} and A_{RIG-V} shall be taken as $0.67 \cdot S_{DSfr}$ and $0.27 \cdot S_{DSfr}$, respectively.

All components and equipment required by this section shall meet the post-test functional compliance verification of ICC-ES AC156 Section 6.8.2 regardless of the assigned seismic functional recovery importance factor, I_{pfr} , or seismic functional recovery qualification importance factor, I_{qualfr} .

Table A.11-1. Functional Recovery Nonstructural Qualification Importance Factors by Functional Recovery Category for Earthquake Loads

Functional Recovery Category	Seismic Functional Recovery Qualification Importance Factor, I_{qualfr}
A, B and C	1.5
D	1.25

Table A.11-2. Seismic Certification Floor Acceleration Threshold

Equipment or Component	Seismic Certification Floor Acceleration Threshold (g) by Functional Recovery Category	
	C	D
MECHANICAL		
Chillers, including associated evaporators and condensers	0.3	0.4
Cooling Towers	0.6	0.7
Compressors	0.5	0.6
HVAC Fans	0.9	1.0
Variable/Constant Air Volume Boxes	0.7	0.9
Air Handling Units	0.4	0.6
Control Panels	1.1	1.3
Other Mechanical Equipment	0.3	0.4
ELECTRICAL		
Transformers	1.0	1.2
Motor Control Centers	1.0	1.1
Low Voltage Switchgear and Switchboards	1.0	1.1
Distribution Panels	1.2	1.3
Battery Racks	1.1	1.2
Battery Chargers	0.8	1.0
Generators	1.0	1.1
Other Electrical Equipment	0.8	1.0

INTERIOR ARCHITECTURAL AND LIGHTING

Independent Hanging Lighting	0.6	0.7
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A.12 DOCUMENTATION AND REVIEW

A.12.1 Scope The requirements of this section shall apply to structures assigned to Functional Recovery Category A, B, C or D.

EXCEPTION: A functional recovery coordinator in accordance with Section A.12.6 and the requirements specific only to the functional recovery coordinator in Section A.12.2 through A.12.5 are not required for structures assigned to Functional Recovery Category D.

A.12.2 Documentation The following information related to functional recovery shall be shown on the construction documents, regardless of whether functional recovery governs the design:

1. The basic intended function of the structure identified in accordance with Section A.4.
2. All elements, components, equipment, and distribution systems required by Section A.4. For nonstructural elements, components, equipment, and distribution systems, construction documents shall include layout of the system and/or location of the component or equipment, locations of the gravity and seismic restraints, and details of seismic restraints. Light-frame (wood) walls not specifically designated as part of the seismic force-resisting system but required to achieve functional recovery in accordance with Section A.8.4.5 Exception 2 shall be indicated.
3. Delegated design elements, components, equipment, and distribution systems.
4. The assigned Functional Recovery Category in accordance with Section A.5. Where multiple Functional Recovery Categories are assigned to a single structure in accordance with the exception to Section A.5.1, the construction documents shall clearly identify the elements, components, equipment, and distribution systems in Section A.12.2 Item 2 required to support each portion of the structure and their independence from each other.
5. Description or plan drawing of the system boundary.
6. Functional recovery spectral response acceleration parameters, S_{DSfr} and S_{D1fr} .
7. Functional recovery response modification coefficient, R_{fr}
8. Functional recovery allowable story drift, Δ_{afr}
9. Seismic functional recovery component importance factor, I_{pfr} , and seismic functional recovery qualification importance factor, I_{qualfr} .

10. Special inspections and tests required for all elements, components, equipment, and distribution systems in accordance with Section A.14.3. Where tests of post-installed anchors are required in accordance with Section A.14.8, the test load(s) shall be shown.

For delegated design elements, components, equipment, and distribution systems required to support the basic intended function of the structure, the registered design professional delegating the design and the functional recovery coordinator shall additionally specify the functional recovery criteria on the construction documents.

For buildings assigned to Functional Recovery Category A, a written statement acknowledging the elements, components, equipment, and distribution systems required to support the basic intended function shall be signed by the functional recovery coordinator, in addition to the registered design professionals responsible for the structural, architectural, mechanical, electrical, plumbing, and fire suppression design.

A.12.3 Delegated Design Delegated design elements, components, equipment, and distribution systems shall be designed by a registered design professional. Review and acceptance of delegated design shall be performed by the registered design professional responsible for delegating the design and the functional recovery coordinator. Review and acceptance of delegated design by the registered design professional delegating the design shall be for general conformance with the functional recovery criteria and for impacts on structure, if applicable. Review and acceptance of delegated design by the functional recovery coordinator shall be for general conformance with the functional recovery criteria. In addition, for structures assigned to Functional Recovery Category A or B, review and acceptance of delegated design by the functional recovery coordinator shall include review of modeling, analyses, design, detailing, and documentation associated with design for functional recovery.

A.12.4 Seismic Certification Evidence demonstrating compliance with the requirements of Section A.11 shall be reviewed and accepted by the registered design professional responsible for the structural design and the functional recovery coordinator and submitted to the Authority Having Jurisdiction.

A.12.5 Design Changes During Construction The registered design professional responsible for the design, that has been altered or changed, shall notify the functional recovery coordinator and Authority Having Jurisdiction of elements, components, equipment, and distribution systems which are substantially altered or changed after review by the functional recovery coordinator, where the alteration or change affects the way in which the element, component, equipment, and distribution system supports the basic intended function of the structure. Additionally, for structures following the requirements of Section A.17 or for structures assigned to Functional Recovery Category A following the requirements of Section A.1.6, the independent reviewer(s) shall be notified. For delegated design, the registered design professional who delegated the design shall also be notified. Re-review and re-acceptance shall be performed by the registered design professional who delegated the design, as applicable for delegated design, and the functional recovery coordinator and re-submitted to the Authority Having Jurisdiction.

A.12.6 Functional Recovery Coordinator A registered design professional possessing knowledge of the following items shall act as the functional recovery coordinator. A registered design professional responsible for any portion(s) of the design shall be permitted to also act as the functional recovery coordinator.

The functional recovery coordinator shall review, accept, and coordinate the documentation, observation, quality assurance, and recovery plan for all elements, components, equipment, and distribution systems required by Section A.4 including the following:

1. Review and acceptance of functional recovery requirements contained in construction documents in Section A.12.2.
2. Review and acceptance of functional recovery requirements contained in delegated design documents in Section A.12.3.
3. Review and acceptance of seismic certification in accordance with Section A.12.4.
4. Preparation of the statement of special inspections in Section A.14.3.
5. For structures assigned to Functional Recovery Category A or B, review and acceptance of design for the on-site backup and external hookup requirements of Section A.16.
6. Review and acceptance of functional recovery observation reports in Section A.13.2.
7. Leading the pre-construction functional recovery conference(s) in Section A.13.3.
8. Participation in the post-construction functional recovery walk-through(s) in Section A.13.4.
9. Review and acceptance of special inspection and test reports in Section A.14.2.
10. For structures assigned to Functional Recovery Category A and B, review and acceptance of the nonstructural system coordination drawings in Section A.14.5.2.
11. For structures assigned to Functional Recovery Category A and B, review and acceptance of the functional recovery compliance statements prepared by the special inspector and general contractor in Section A.14.6.
12. For structures assigned to Functional Recovery Category A or B, review and acceptance of the post-earthquake functional recovery plan in Section A.15.

At the conclusion of the preparation or review and acceptance of Items 1 through 5 above, the functional recovery coordinator shall submit to the Authority Having Jurisdiction an interim written statement that the review has been made and identify any non-compliant items that, to the best of the functional recovery coordinator's knowledge, have not been resolved.

At the conclusion of the review and acceptance of the remaining items above, the functional recovery coordinator shall submit to the Authority Having Jurisdiction a final written statement that the remaining review has been made and identify any additional non-compliant items that, to the best of the functional recovery coordinator's knowledge, have not been resolved.

A.13 OBSERVATION, CONFERENCES AND WALK-THROUGHS

A.13.1 Scope The requirements of this section shall apply to structures assigned to Functional Recovery Categories A, B, C, or D.

A.13.2 Observation Responsibility and Reports One or more registered design professional(s) shall perform functional recovery observations and prepare functional recovery observation reports of the structural and nonstructural elements, components, equipment, and distribution systems, including delegated design, required by Section A.4. Functional recovery observations shall be performed visually of representative locations for general conformance to the construction documents. Functional recovery observation does not include or waive the responsibility for special inspections in accordance with Section A.14.

Functional recovery observation reports shall be reviewed and accepted by the functional recovery coordinator and submitted to the Authority Having Jurisdiction.

A.13.3 Pre-Construction Functional Recovery Conference(s) Pre-construction functional recovery conference(s) shall be held. For structures assigned to Functional Recovery Categories A, B, or C, the functional recovery coordinator shall arrange conference(s) to describe the performance objectives, documentation, review, observation, and quality assurance requirements for the design in accordance with this appendix. For structures assigned to Functional Recovery Category D, a registered design professional shall arrange the conference(s). Conference(s) shall be held before commencement of the associated phase of construction. The functional recovery coordinator (where required), registered design professional(s) required by Section A.13.2, and contractor shall attend. Where required by the Authority Having Jurisdiction, a representative of the Authority Having Jurisdiction shall also attend. At the conclusion of the meeting, each of the required attendees shall sign a statement of responsibility acknowledging awareness of the special requirements for functional recovery to be submitted to the Authority Having Jurisdiction.

A.13.4 Post-Construction Functional Recovery Walk-Through(s) Post-construction functional recovery walk-through(s) shall be made by the functional recovery coordinator, registered design professional(s) required by Section A.13.2, and contractor. Where required by the Authority Having Jurisdiction, a representative of the Authority Having Jurisdiction shall also attend. The walk-through(s) shall occur at substantial completion of construction of the systems required for functional recovery and shall include representative visual observation of such systems.

A.13.5 Final Written Statement of Functional Recovery Observations At the conclusion of the work included in the building permit, the registered design professional(s) required by Section A.13.2 shall submit to the functional recovery coordinator and Authority Having Jurisdiction a final written statement that the functional recovery observations have been made. Any non-compliant items shall be identified that, to the best of the registered design professional's knowledge, have not been resolved.

A.14 QUALITY ASSURANCE

A.14.1 Scope The quality assurance requirements of this section shall apply to structures assigned to Functional Recovery Category A, B, C or D. In addition to this section, the requirements of Section 11.1.5 and the applicable sections of the applicable building code shall be satisfied.

A.14.2 Special Inspection and Test Responsibility The owner or owner's authorized agent shall employ one or more approved agencies to provide special inspections and tests during construction. Such special inspections and tests shall follow the statement of special inspections in accordance with Section A.14.3. For structures assigned to Functional Recovery Category A and B, the agency providing special inspection and tests shall be approved by the functional recovery coordinator.

A.14.2.1 Identified Non-Compliant Items The approved agency shall be responsible for maintaining a log of items non-compliant with the statement of special inspections and closing items on this log. The log shall be available to the general contractor, the entity that prepared the statement of special inspection in accordance with Section A.14.3, the special inspector, and, upon request, the Authority Having Jurisdiction.

The entity that prepared the statement of special inspection in accordance with Section A.14.3 shall notify the AHJ of non-compliant items that have not been resolved.

A.14.3 Statement of Special Inspections For structures assigned to Functional Recovery Category A, B or C, the functional recovery coordinator shall prepare a statement of special inspections. For structures assigned to Functional Recovery Category D, the statement of special inspections shall be prepared by a registered design professional.

The registered design professional responsible for the design of each element, component, equipment, and distribution system required by Section A.4 shall be responsible for developing the required special inspections and tests applicable to that element, component, equipment, and distribution system, including those items that are delegated design, and showing them on the construction documents.

Where a statement of special inspection is required by the applicable building code, the special inspection and testing required by this Section shall be added to those required by the applicable building code.

The statement of special inspections required by this Section shall identify the following:

1. The elements, components, equipment, and distribution systems required to support the basic intended function of the structure in accordance with Section A.4.
2. The type and extent of each special inspection.
3. The type and extent of each test.
4. For each type of special inspection, identification as to whether it will be continuous special inspection, periodic special inspection, or performed in accordance with the notation used in the referenced standard where the inspections are defined.
5. The required qualifications of the individual(s) performing the special inspection and testing for elements, components, equipment, and distribution systems identified in Item 1 and Table A.14-1.
6. The frequency and distribution of special inspection and test reports.

The statement of special inspection shall be submitted to the Authority Having Jurisdiction. Where the statement of special inspection is changed after submission to the Authority Having Jurisdiction, the changes to the statement of special inspection shall be reviewed by the registered design professional who prepared the statement of special inspections in accordance with this section. The revised statement of special inspections shall be resubmitted to the Authority Having Jurisdiction.

A.14.4 Continuous and Periodic Special Inspections Continuous and periodic special inspections for functional recovery shall be defined in accordance with the applicable building code. Both continuous and periodic inspections shall include inspection of all completed elements of the work or portions thereof prior to acceptance or covering.

A.14.5 Contractor Responsibility

A.14.5.1 Contractor Functional Recovery Statement of Responsibility The general contractor shall submit a written contractor's functional recovery statement of responsibility to the Authority Having Jurisdiction and to the owner or the owner's authorized agent before the commencement of work. The contractor's functional recovery statement of responsibility shall contain acknowledgements of all of the following:

1. Awareness of the special requirements for functional recovery contained in the statement of special inspections.
2. That the special requirements for functional recovery have been communicated to appropriate subcontractor(s).
3. That control will be exercised to obtain conformance with the construction documents approved by the Authority Having Jurisdiction.

A.14.5.2 Schedule and Log An up-to-date schedule corresponding to all special inspections and tests identified in Section A.14.3 shall be maintained on site by the contractor and made available to the entity who prepared the statement of special inspection in accordance with Section A.14.3, the approved agency, and, upon request, the Authority Having Jurisdiction.

The general contractor is responsible for identifying the path to resolution of the non-compliant items identified in non-compliant item log according to Section A.14.2.

A.14.6 Functional Recovery Compliance Statements

A.14.6.1 General For structures assigned to Functional Recovery Categories A and B, functional recovery compliance statements shall be prepared by the functional recovery coordinator, special inspector, and general contractor in accordance with this section. Functional recovery compliance statements shall verify that the work during the period or portion of the construction covered by the compliance statement is in accordance with the construction documents, the applicable building code, and this appendix.

A.14.6.2 Submission Functional recovery compliance statements shall be submitted for the following:

1. At substantial completion of foundation construction.
2. At substantial completion of the structural part of construction.
3. At substantial completion of construction of the systems identified in accordance with Section A.4.
4. At intervals as established by the functional recovery coordinator.
5. At the conclusion of the work included in the building permit.

Functional recovery compliance statements prepared by the special inspector or general contractor shall be reviewed and accepted by the functional recovery coordinator. All functional recovery compliance statements shall be submitted to the Authority Having Jurisdiction.

A.14.6.3 Functional Recovery Coordinator Compliance Statements Functional recovery compliance statements prepared by the functional recovery coordinator shall be based on knowledge gained by performing or directly supervising personnel performing observations in accordance with the requirements of the applicable building code and this appendix. Functional recovery compliance statements prepared by the functional recovery coordinator shall detail the functional recovery observations which have been made. Any non-compliant items shall be identified that, to the best of the functional recovery coordinator's knowledge, have not been resolved. Where non-compliant items have been resolved, a record of the resolution shall be included.

A.14.6.4 Special Inspector Compliance Statements Functional recovery compliance statements prepared by the special inspector shall be based on knowledge obtained by performing or directly supervising personnel special inspections and tests in accordance with the requirements of the applicable building code and this appendix. Functional recovery compliance statements prepared by the special inspector shall detail the special inspections and tests which have been made. Any non-compliant items shall be identified that, to the best of the special inspector's knowledge, have not been resolved. Where non-compliant items have been resolved, a record of the resolution shall be included.

A.14.6.5 Contractor Compliance Statements Functional recovery compliance statements prepared by the general contractor shall be based on knowledge obtained from the construction of the structure, or parts thereof, in accordance with the requirements of the applicable building code and this appendix. Functional recovery compliance statements prepared by the general contractor shall detail the work performed, as well as the functional recovery observations and special inspections and tests which have been made. Any non-compliant items shall be identified that, to the best of the general contractor's knowledge, have not been resolved. Where non-compliant items have been resolved, a record of the resolution shall be included.

A.14.7 Required Additional Special Inspections for Functional Recovery For elements, components equipment, and distribution systems identified in the statement of special inspections in accordance with Section A.14.3, special inspections for functional recovery shall be in accordance with this section and the applicable sections of the applicable building code.

A.14.7.1 Special Inspection of Structural and Nonstructural Elements, Components, Equipment and Distribution Systems Special inspections of structural and nonstructural elements, components, equipment and distribution systems shall be in accordance with Table A.14-1.

Table A.14-1. Special Inspection of Structural and Nonstructural Elements, Components, Equipment and Distribution Systems

Item	Special Inspection Type ^a		Reference	Functional Recovery Category			
	C	P		A	B	C	D
Structural System							
Main gravity force-resisting system	As required by applicable building code		A.14.7.5	X	X	X	X
Main lateral force-resisting system			A.14.7.5	X	X	X	X

Item	Special Inspection Type ^a		Reference	Functional Recovery Category			
	C	P		A	B	C	D
Anchorage							
Post-installed anchors in concrete or masonry		X ^c	A.14.7.2	Special inspection of anchors required only when special inspection of anchored nonstructural element, component, equipment, and distribution system is also required.			
Cast in place anchors in concrete or masonry		X	A.14.7.2				
Power-actuated/driven fasteners		X	A.14.7.2				
Epoxy dowels, rods, or anchors in mass timber		X	A.14.7.2				
Other anchorage ^b		X	A.14.7.2				
Means of egress systems							
Exit stairs and ramps		X	A.14.7.5	X	X	X	X
Exit doors		X	A.14.7.11	X	X	X	X
Exit corridors		X	A.14.7.6 A.14.7.8	X	X	X	X
Other egress components ^b		X	A.14.7.11	X	X	X	X
Fire protection systems							
Sprinkler systems – distribution piping		X	A.14.7.3	X	X	X	X
Sprinkler systems – sprigs and drops		X	A.14.7.3	X	X	X	X
Sprinkler systems - pumps, valves		X	A.14.7.4	X	X	X	X
Sensors and controls		X	A.14.7.4	X	X	X	X
Smoke control - ducting		X	A.14.7.3	X	X	X	X
Smoke control - valves and actuators		X	A.14.7.4	X	X	X	X
Other fire suppression systems ^b		X	A.14.7.11	X	X	X	X

Item	Special Inspection Type ^a		Reference	Functional Recovery Category			
	C	P		A	B	C	D
Electrical power systems							
Panels		X	A.14.7.4	X	X	X	
Switchgear		X	A.14.7.4	X	X	X	
Transformers		X	A.14.7.4	X	X	X	
Inverters		X	A.14.7.4	X	X	X	
Battery racks		X	A.14.7.7	X	X	X	
Motor starters		X	A.14.7.4	X	X	X	
Motor control centers		X	A.14.7.4	X	X	X	
PV systems		X	A.14.7.4	X	X	X	
Distribution Systems - conduits		X	A.14.7.3	X	X	X	
Distribution Systems - raceways/cable trays		X	A.14.7.3	X	X	X	
Distribution Systems - bus ducts		X	A.14.7.3	X	X	X	
Other electrical power equipment and distribution systems ^b		X	A.14.7.11	X	X	X	
Exterior enclosure systems							
Wall framing		X	A.14.7.6	X	X		
Wall panels		X	A.14.7.6	X	X	X	X
Masonry walls/infill		X	A.14.7.6	X	X		
Veneer ^d		X	A.14.7.6	X	X		
Glazing		X	A.14.7.6	X	X	X	X
Curtain wall		X	A.14.7.6	X	X	X	X
Movement joints		X	A.14.7.6	X	X	X	X
Appendages and ornamentation		X	A.14.7.6	X	X		

Item	Special Inspection Type ^a		Reference	Functional Recovery Category			
	C	P		A	B	C	D
Waterproofing		X	A.14.7.6	X	X	X	X
Other exterior enclosure components and systems ^b		X	A.14.7.11	X	X	X	X
Potable water and sanitary drainage systems							
Potable water distribution piping		X	A.14.7.3	X	X	X	X
Pumps, valves		X	A.14.7.4	X	X	X	
Sewer distribution and vent piping		X	A.14.7.3	X	X	X	X
Drain distribution and vent piping		X	A.14.7.3	X	X	X	X
Wet wells, sumps, pumps		X	A.14.7.4	X	X	X	
Other potable water and sanitary drainage systems ^b		X	A.14.7.11	X	X	X	X
Heating, ventilation, and air-conditioning systems							
Boilers and furnaces		X	A.14.7.4	X	X	X	X
Heat pumps		X	A.14.7.4	X	X	X	X
Water heaters		X	A.14.7.4	X	X	X	X
Chillers		X	A.14.7.4	X	X	X	X
Cooling towers		X	A.14.7.4	X	X	X	X
Heat exchangers		X	A.14.7.4	X	X	X	X
Expansion tanks and other pressure vessels		X	A.14.7.4	X	X	X	X
Compressors		X	A.14.7.4	X	X	X	X
Air handlers/fans		X	A.14.7.4	X	X	X	X
Motors		X	A.14.7.4	X	X	X	X
Pumps		X	A.14.7.4	X	X	X	X

Item	Special Inspection Type ^a		Reference	Functional Recovery Category			
	C	P		A	B	C	D
Condensers		X	A.14.7.4	X	X	X	X
Furnaces		X	A.14.7.4	X	X	X	X
Evaporators		X	A.14.7.4	X	X	X	X
Air separators		X	A.14.7.4	X	X	X	X
Coils		X	A.14.7.4	X	X	X	X
Cooling		X	A.14.7.4	X	X	X	X
Variable Air Volume Box		X	A.14.7.4	X	X	X	X
Suspended equipment		X	A.14.7.4	X	X	X	X
Vibration isolation		X	A.14.7.10	X	X	X	
Shut off valves		X	A.14.7.3	X	X	X	X
Distribution systems - heated water piping		X	A.14.7.3	X	X	X	X
Distribution systems - chilled water piping		X	A.14.7.3	X	X	X	X
Distribution systems - steam piping		X	A.14.7.3	X	X	X	
Distribution systems - gas piping		X	A.14.7.3	X	X	X	X
Distribution systems - gas piping seismic switch		X	A.14.7.3	X	X	X	X
Distribution systems –hazmat piping		X	A.14.7.3	X	X	X	X
Medical/laboratory gas piping		X	A.14.7.3	X	X	X	X
Ductwork - air supply and exhaust		X	A.14.7.3	X	X	X	
Ductwork - hazardous materials		X	A.14.7.3	X	X	X	X
Pipe risers		X	A.14.7.3	X	X	X	X
Air distribution boxes/diffusers		X	A.14.7.4	X	X	X	

Item	Special Inspection Type ^a		Reference	Functional Recovery Category			
	C	P		A	B	C	D
Other heating, ventilation, and air-conditioning equipment and systems ^b		X	A.14.7.11	X	X	X	X
Interior architectural components and lighting systems							
Masonry partitions		X	A.14.7.6	X	X		
Other partitions		X	A.14.7.6	X	X		
Fire rated partitions		X	A.14.7.6	X	X	X	
Suspended ceilings		X	A.14.7.8	X	X		
Lighting fixtures		X	A.14.7.8	X	X		
Access floors		X	A.14.7.9	X	X	X	
Laboratory equipment		X	A.14.7.4	X	X	X	
Medical equipment		X	A.14.7.4	X	X	X	
Storage cabinets and shelving/racks > 6 feet tall		X	A.14.7.7	X	X	X	
Other interior architectural components and lighting systems ^b		X	A.14.7.11	X	X	X	X
Elevator systems							
Elevator machine		X	A.14.7.4	X	X	X	X
Hydraulic system pumps and tanks		X	A.14.7.4	X	X	X	X
Rail connections and splices		X	A.14.7.5	X	X	X	X
Cable guides		X	A.14.7.5	X	X	X	X
Elevator framing		X	A.14.7.5	X	X	X	X
Seismic switch		X	A.14.7.4	X	X		
Escalators		X	A.14.7.11	X	X	X	
Material handling conveyors		X	A.14.7.11	X	X	X	X

Item	Special Inspection Type ^a		Reference	Functional Recovery Category			
	C	P		A	B	C	D
Other elevator systems ^b		X	A.14.7.11	X	X	X	X
Telecommunication and data systems							
Servers, routers, and switches		X	A.14.7.4	X	X	X	
Equipment rack systems		X	A.14.7.7	X	X	X	
Cable trays		X	A.14.7.3	X	X	X	
Battery racks		X	A.14.7.7	X	X	X	
Communication antennas		X	A.14.7.11	X	X		
Alarm system		X	A.14.7.11	X	X		
Building access and safety system		X	A.14.7.11	X	X	X	X
Other telecommunication and data systems ^b		X	A.14.7.11	X	X	X	X
Backup systems							
Emergency generators		X	A.14.7.4	X			
Water tank		X	A.14.7.11	X			
Sewage tank		X	A.14.7.11	X			
Other backup systems ^b		X	A.14.7.11	X			

^a C = continuous; P = periodic

^b The registered design professional responsible for the design of each element, component, equipment, and distribution system, shall provide a list of other elements, components, equipment, and distribution systems and their special inspection requirements.

^c See Section A.14.8.1.1 for requirements on test requirements for periodic and continuous special inspection

^d Veneer shall be inspected for any height. TMS 602 requirement for special inspection of only veneer higher than 60ft is not applicable.

A.14.7.2 Special Inspection of Anchors Special inspections of post-installed anchors to concrete or masonry shall include all of the following:

1. Confirm anchor type including mechanism, material, diameter, and embedment.
2. Confirm conformance with the construction documents including edge distance, anchor spacing, and hole diameter in anchored component is appropriate for anchor diameter,
3. Confirm installation procedure is in conformance with Manufacturer's Printed Installation Instructions (MPII) including hole size, hole drilling method, hole cleaning method, installation and tightening, use of washers.
4. Confirm that anchors that were initially not installed properly are addressed correctly.

Special inspections of cast-in-place anchors in concrete or masonry shall include all of the following:

1. Prior to concrete placement or during the masonry layup, confirm anchor type including mechanism, material, diameter, and embedment.
2. Prior to concrete placement or during the masonry layup, confirm conformance with the construction documents, evaluation report and manufacturer's instructions including edge distance, anchor spacing, hole diameter in anchored component is appropriate for anchor diameter, and use of washers.
3. After concrete placement or following masonry grouting, confirm anchor being normal relative to concrete or masonry surface as appropriate, extent of anchor projection above surface is appropriate, threads are undamaged, and, for internally-threaded anchors, threads are clean and functional.

Special inspections of power-actuated/driven fasteners in concrete shall include all of the following:

1. Confirm pin type including material, diameter, and embedment.
2. Confirm conformance with the construction documents, evaluation report, and manufacturer's instructions, including edge distance, anchor spacing, and use of washers.
3. Identify fasteners that fail to hold and that they are addressed properly.

Special inspections of epoxy dowels, rods, or anchors in mass timber shall include the following:

1. Confirm anchor type including, mechanism, material, diameter, and embedment.
2. Confirm conformance with the construction documents including edge distance, anchor spacing, and hole diameter in anchored component is appropriate for anchor diameter,

3. Confirm installation procedure conforms with MPII, including hole size, hole drilling method, hole cleaning method, installation and tightening, use of washers, and that fail-to-sets are addressed correctly.

For other types of anchors, the registered design professional responsible for the anchor design shall develop special inspection requirements.

A.14.7.3 Special Inspections of Distribution Systems Special inspections of distribution systems shall include all of the following:

1. Inspect anchors in accordance with Section A.14.7.2.
2. Confirm distribution system size, material, connection type, and installation level/elevation are in conformance with the construction documents.
3. Confirm that the vertical supports and seismic bracing systems are installed at the locations, elevations, spacing, clearances, and physical interrelationships specified on the construction documents.
4. Confirm that the vertical support and seismic bracing systems are installed in accordance with the manufacturers, materials, and methods specified on the construction documents. This shall include all of the following: approved angles of bracing systems, distance between rod drop and rod stiffener material where required, spacing of rod stiffener restraints on rod stiffener materials, size and construction of bracing arms and brackets, diameter of rod drops, trapeze materials and spans, and in-plane lateral restraint devices.
5. Confirm beam clamps, wall attachments, and other connectors, including manufacturer, system, and size.
6. Confirm location and installation of water and gas shut-off valves and access to them.
7. Confirm clearances to other structural and nonstructural elements, components, equipment, and distribution systems, including penetrations.
8. Confirm type of pipe joints and their bracing are in conformance with the construction documents.
9. Confirm flexible connections or other measures to accommodate relative displacements are installed in accordance with the construction documents and accommodate relative displacement in all directions of anticipated movement including at seismic joints.
10. Any additional special inspection required in accordance with Section A.14.7.5.

A.14.7.4 Special Inspection of Equipment Special inspections of equipment shall include all of the following:

1. Inspect anchors in accordance with Section A.14.7.2.
2. Where seismic certification is required in accordance with Section A.11, confirm that the label, anchorage, and mounting conforms to the seismic certification.
3. Confirm equipment bracing and/or strapping details for conformance with construction documents, including welding, screw attachments, bolting, anchorage, and other fastening methods.
4. Confirm equipment installation for conformance with construction documents.
5. Where distribution systems are connected to the equipment, confirm flexible connection or other measures to accommodate the relative displacements are installed in accordance with the construction documents and accommodate relative displacement in all directions of anticipated movements.

A.14.7.5 Inspection of Elements Made of Structural Materials

A.14.7.5.1 Inspection of Main Gravity and Lateral Force-Resisting System Inspection of the main gravity and lateral force-resisting system shall be performed in accordance with requirements of applicable building code.

A.14.7.5.2 Special Inspection of Other Elements Made of Structural Materials Where referenced in A.14.7, special inspections of structural and nonstructural elements or components made of structural materials shall be in accordance with Table A.14-2.

Table A.14-2. Special Inspection of Structural Elements or Components Not Part of the Main Gravity or Lateral Force-Resisting System

Material ^a	Special Inspection Requirements ^b
Steel	AISC 360, ANSI/SDI QA/QC
Concrete	ACI 318, Applicable building code for seismic resistance
Masonry	TMS 602, Level 2
Wood	Applicable building code for seismic resistance
Light-frame cold-formed steel	Applicable building code for seismic resistance

^a For materials not listed, refer to appropriate material standards for special inspection requirements. The requirements shall at minimum match requirements for gravity force resisting systems.

^b Where special inspection requirements reference material standards, it is permitted to adopt special inspection notation as defined in those standards. Regardless of definition, all components shall be inspected as required in A.14.4.

A.14.7.6 Special Inspection of Nonstructural Walls Special inspections of nonstructural walls shall include the following:

1. Inspect anchors in accordance with Section A.14.7.2 and other fastening between framing members and primary structure.
2. Confirm location, spacing, and arrangement of kickers or bracing, and framing members including studs, top and bottom plates, sills, headers, and mullions are in accordance with construction documents.
3. Confirm connections, bracing, backing plates, penetrations, and connection of finishes to framing are in accordance with construction documents.
4. For penetrations, confirm that clearance specified in the construction documents is achieved.
5. Where displacement testing is required in accordance with Section A.10.8.3, confirm that the tested mock-up is representative of the constructed product, including framing and/or panels, connections between panels and the structure, and movement joint size and location.
6. Confirm that connections or other measures to accommodate the story drift and vertical deflections are installed in accordance with the construction documents.
7. For exterior enclosure, confirm the size of required movement joints and waterproofing are in accordance with construction documents.
8. Any additional special inspection required in accordance with Section A.14.7.5.

A.14.7.7 Special Inspection of Racks Special inspections of racks shall include all of the following:

1. Inspect anchors in accordance with Section A.14.7.2.
2. Confirm frame size, material, and connection type are in accordance with construction documents.
3. For fabricated items used in racks, it shall be permitted to provide inspection in accordance with the applicable building code or industry standard.
4. Where cable tray is connected to top of racks, confirm flexible connection and other measures to accommodate the relative displacements are installed in accordance with the

construction documents and accommodate relative displacement in all directions of anticipated movements.

5. Confirm straps on individual battery units to prevent overturning and spilling of hazardous materials are in accordance with construction documents.
6. Any additional special inspection required in accordance with Section A.14.7.5.

A.14.7.8 Special Inspection of Ceilings Special inspections of ceilings shall include all of the following:

1. Inspect anchors in accordance with Section A.14.7.2.
2. Confirm framing member size, type and material, splices, hangers, braces, fasteners, connections, and wall clearances are in accordance with construction documents.
3. Confirm that the vertical supports and seismic bracing systems are installed at the locations, elevations, spacing, clearances, and physical interrelationships specified on the construction documents.
4. Confirm that the vertical support and seismic bracing systems are installed in accordance with the manufacturers, materials, and methods specified on the approved contract documents. This shall include, but is not limited to: approved angles of bracing systems, distance between rod drop and rod stiffener material where required, spacing of rod stiffener restraints on rod stiffener materials, size and construction of bracing arms and brackets, diameter of rod drops, trapeze materials and spans, and the presence of in-plane lateral restraint devices attached to the utilities

A.14.7.9 Special Inspection of Access Floors Special inspection of access floors shall include all of the following:

1. Inspect anchors in accordance with Section A.14.7.2.
2. Confirm that the vertical supports and seismic bracing systems are installed at the locations, elevations, spacing, clearances, and physical interrelationships specified on the construction documents.
3. Confirm that the vertical supports and seismic bracing systems are installed in accordance with the manufacturers, materials, and methods specified on the construction documents.

A.14.7.10 Special Inspection of Vibration Isolation Special inspection of vibration isolation shall include the following:

1. For equipment requiring seismic certification in accordance with Section A.11, confirm that the seismic certification permits vibration isolation.

2. Confirm that vibration isolation has properly installed seismic restraints.

A.14.7.11 Special Inspection of Other Nonstructural Elements, Components, Equipment and Distribution Systems Special inspection requirements of other nonstructural elements, components, equipment, and distribution systems not addressed in other sections shall be developed by the registered design professional responsible for the design of the respective element, component, equipment, or distribution system.

A.14.8 Required Tests of Post-Installed Anchors for Functional Recovery When special inspections of post-installed anchors are required in accordance with Section A.14.7.1, post-installed anchors shall also be tested in accordance with either Section A.14.8.1 or A.14.8.2. Where failure testing to qualify the installer in accordance with Section A.14.8.2. is selected, the general contractor shall provide to the special inspector all of the following:

1. Name of qualified installers present on the site each day.
2. General installation area of each qualified installer.
3. For adhesive post-installed anchors, identification of the first anchor installed by each qualified installer each day.

Tension testing shall be performed in accordance with ASTM E3121.

All post-installed anchors shall have ready access for proof load testing at the first inspection visit of the special inspector after anchor installation regardless of whether the anchors are tested in accordance with Section A.14.8.1 or A.14.8.2.

Each type of post-installed anchor, diameter, embedment length, installation procedure, installation position, and subcontractor installing the anchors shall be tested as a separate category.

The test loads for each category shall be defined in contract documents according to Section A.12.2.

EXCEPTIONS:

1. The individual(s) who prepared the statement of special inspections shall be permitted to group similar post-installed anchors into fewer categories. The post-installed anchors in each resulting category shall be tested using appropriate loads for that respective category.
2. Where a connection is made by post-installed anchors and all post-installed anchors within that connection have a demand-to-capacity ratio less than or equal to 0.25, the testing requirements of this section shall be permitted to be neglected for the post-installed anchors for that connection. These connections shall be identified on the construction documents by the registered design professional responsible for the component.
3. Testing is not required for power-actuated fasteners.

A.14.8.1 Proof Load Tests Proof load tests shall be performed as tension tests except, for torque-controlled expansion anchors only, it shall be permitted to use torque-tests. The test frequency for each category of anchors shall be in accordance with Section A.14.8.1.1. For tension tests of adhesive anchors, there shall be no restriction on distance between the anchor and the loading equipment reactions.

The tension proof load shall be taken as 1.25 x design strength, where design strength is determined in accordance with ACI 318, but shall not exceed 0.8 times the nominal yield capacity of the anchor or 67% of characteristic bond stress in uncracked concrete for adhesive anchors. The tension proof load test shall be deemed acceptable when the proof load is applied for 15 seconds, and the anchor does not exhibit movement exceeding 0.1 inches.

The torque proof load shall be established in accordance with the manufacturer's recommended installation torque based on an approved evaluation report. Torque-controlled post-installed expansion anchors shall attain the specified torque in MPa within $\frac{1}{2}$ turn of the nut except for 3/8-inch sleeve anchors only, which shall be attained within $\frac{1}{4}$ turn of the nut.

For adhesive post-installed anchors, all of the following shall apply:

1. Proof load tests shall be performed only after minimum cure time specified in MPa has elapsed.
2. When required by Section A.14.8.2, the first anchor installed by each qualified installer each day shall be considered preferential for proof load tests.

A.14.8.1.1 Test Frequency Testing of post-installed anchors shall be performed at random intervals throughout the construction period. Test frequency shall be in accordance with Table A.14-3 based on the total number of items of anchors in each category as defined in 14.8. The contractor shall be responsible for identification of the total number of anchors in each category.

Table A.14-3 Required Test Frequency

Total Number of Anchors in Each category	Testing Frequency
≤ 100	80%
101-1000	50%, but not less than 80
1001-5000	25%, but not less than 500
> 5000	10%, but not less than 1250

EXCEPTION: Test frequency shall be permitted to be lowered to 25% for total number of anchors up to 1000, when continuous inspection for these anchors is performed.

A.14.8.1.2 Failed Test Where a post-installed anchor does not meet the test acceptance criteria, it shall be deemed a failed test. The mechanism(s) leading to failure shall be identified, and all

anchors within the failed anchor category since the last acceptable test shall be identified. A remedy addressing the mechanism(s) leading to failure shall be approved by the individual(s) who prepared the statement of special inspections and the registered design professional responsible for the design of the element, component, equipment, or distribution system. The test frequency shall be 100% for anchors within that category after the failed test.

EXCEPTION: The individual(s) who prepared the statement of special inspections and the registered design professional responsible for the design of the element, component, equipment, or distribution system shall be permitted to lower the test frequency to the initial rate in accordance with Table A.14-3 when all of the following are satisfied:

1. A minimum of 100 items shall be installed and tested after the failed test without another failed test.
2. The contractor shall provide a written description of the mechanism(s) leading to the failed test and steps taken to mitigate the issue(s).

A.14.8.2 Failure Tests Anchor installers for each category of post-installed anchors shall be qualified using failure tests. The qualified installers shall be the only personnel permitted to install anchors in the given category.

An installer shall install three post-installed anchors of each category for which they are qualified. The special inspector shall be present during the installation and shall inspect the anchors.

Each test shall be deemed acceptable when the anchor develops the lesser of nominal strength, where nominal strength is determined in accordance with ACI 318 for uncracked concrete and nonseismic condition, 1.1 times the nominal steel yield capacity of the anchor or 100% expected ultimate bond stress in uncracked concrete.

For the installer to be considered qualified, all three anchors shall pass the failure test. If an installer fails the qualification, one of the following shall be implemented:

1. It shall be permitted to re-qualify the installer following the requirements of this section, except that the number of anchors installed and tested shall be raised to five.
2. The installer shall not be permitted to install anchors within this category.
3. For anchors of this category installed by the installer, proof load tests in accordance with Section A.14.8.1 shall be performed.

Where failure tests are selected for the given anchor type, proof load testing is permitted to be included for this anchor category in the statement of special inspection. In such cases, the proof load testing frequency shall not exceed 5% of the total number of anchors in a given category. Where an anchor fails proof load testing, the requirements of Section A.14.8.1 shall apply for the remaining anchors.

A.15 POST-EARTHQUAKE FUNCTIONAL RECOVERY PLAN

A.15.1 Scope The requirements of this section shall apply to structures assigned to Functional Recovery Category A or B.

A.15.2 Responsibility A registered design professional shall prepare a post-earthquake functional recovery plan.

A.15.3 Functional Recovery Plan The post-earthquake functional recovery plan shall include all of the following:

1. Identification of structural and nonstructural elements, components, equipment, and distribution systems per Section A.4, which require post-earthquake functional recovery evaluation. The subset of elements, components, equipment, and distribution systems required to be evaluated by a registered design professional shall be identified separately from the subset not required to be evaluated by a registered design professional.
2. Instructions for evaluation of structural and nonstructural elements, components, equipment, and distribution systems identified in Item 1, including potential damage mechanisms. Where hazardous substances may be encountered, special instructions shall be included describing what actions must be taken before evaluation can occur.
3. Description of the minimum qualifications and arrangements necessary for individual(s) who may perform post-earthquake functional recovery evaluation of the subset of elements, components, equipment, and distribution systems identified in Item 1 that are not required to be evaluated by a registered design professional.
4. Description of the minimum qualifications and arrangements necessary for individual(s) who may perform post-earthquake functional recovery evaluation of the subset of elements, components, equipment, and distribution systems identified in Item 1 that are required to be evaluated by a registered design professional.
5. Description of the minimum qualifications and arrangements necessary for entities who may perform post-earthquake repair and replacement of elements, components, equipment, and distribution systems identified in Item 1.
6. Access to as-built architectural, structural, mechanical, electrical, plumbing, and fire protection drawings maintained by the owner.
7. Instructions for rapid measures to address jammed egress doors if discovered during the post-earthquake functional recovery evaluation.
8. Instructions for rapid measures to address damage to fire protection systems if discovered during the post-earthquake functional recovery evaluation.

9. If elevators are deemed necessary to support the basic intended function in accordance with Section A.4, instructions for rapid alternatives to the use of elevators if damage to elevators is discovered during the post-earthquake functional recovery evaluation.
10. For structures assigned to Functional Recovery Category A or B, description of the operation of the on-site backup or external hookup systems required by Section A.16.
11. For structures assigned to Functional Recovery Category B, description of means of providing temporary power, water and sewage to enable temporary repairs and cleanup during loss of normal service due to external factors.
12. Instructions for regular inspection, maintenance and repair, when required, of structural and nonstructural elements, components, equipment, and distribution systems identified in Item 1 as well as on-site backup and external hookup systems.

The post-earthquake functional recovery plan shall be reviewed by the functional recovery coordinator and submitted to the Authority Having Jurisdiction. Access to the post-earthquake functional recovery plan shall be provided.

A.16 ON-SITE BACKUP AND EXTERNAL HOOKUP REQUIREMENTS

A.16.1 Scope For structures assigned to Functional Recovery Category A, on-site backup requirements for electrical power, potable water, and sewage storage shall be provided in accordance with Section A.16.2. For structures assigned to Functional Recovery Category B, external hookup requirements for electrical power shall be provided in accordance with Section A.16.3.

A.16.2 On-Site Backup Requirements for Functional Recovery Category A

A.16.2.1 Electrical Power Sufficient on-site backup electrical power shall be provided to support the basic intended function of the structure for a period not less than 72 hours. Where fuel is required for on-site backup electrical power, an on-site supply of fuel to support not less than 72 hours of operation shall be required.

A.16.2.2 Potable Water Sufficient on-site backup potable water shall be provided to support the basic intended function of the structure for a period not less than 72 hours. The on-site backup water system shall allow for replenishment from transportable water sources and a means to dispense water to transportable containers

A.16.2.3 Sewage Storage Sufficient on-site sewage storage shall be provided to support the basic intended function of the structure for a period not less than 72 hours. The on-site sewage storage system shall allow for a means to dispose of the stored sewage and liquid waste.

A.16.3 Electrical Power External Hookup Requirements for Functional Recovery Category B An electrical power external hookup shall be provided, consisting of an electrical transfer switch that permits the hookup of a temporary generator.

A.17 EXPLICIT FUNCTIONAL RECOVERY ASSESSMENT PROCEDURE

A.17.1 Acceptable Procedures An acceptable explicit functional recovery assessment procedure shall be a probabilistic methodology that addresses, at a minimum, all of the following:

1. Engineering demand parameters for structural and nonstructural systems, elements, and components.
2. Fragility of structural and nonstructural systems, elements, and components.
3. Repair time at the component level and building repair time, addressing sequencing among multiple components and systems.
4. Functional interactions between multiple systems, elements, and/or components.
5. Impeding factors including inspection, financing, engineering mobilization and design, permitting, and contractor mobilization as well as long lead time items, and surge factors.
6. Inclusion of all structural and nonstructural systems, elements, and components required to support the basic intended function of the structure.

A.17.2 Structural Analysis Requirements For structures assigned to Functional Recovery Category A or B, nonlinear response history analysis of a nonlinear analytical model developed in accordance with Section 16.3 and assessed for the seismic ground motion hazard of Section A.6 shall be performed. The provisions of Section 16.4.1.1 shall apply, and no unacceptable responses shall be permitted, without exception.

EXCEPTIONS:

1. Where a linear structural design is performed in accordance with Section A.8 with R_{fr} taken not greater than 3 and C_{dfr} taken equal to R_{fr} , linear analysis shall be permitted. Where a seismic force-resisting system from Table A.8-1 is used, the R_{fr} and Δ_{afr} in accordance with Table A.8-1 need not apply. For the purposes of this exception, the seismic force-resisting system shall not be restricted to those provided in Table A.8-1.
2. Where the structure meets the requirements of Section A.8.4.3, a structural analysis using rational methods based on accepted principles of engineering mechanics acceptable to the independent reviewer(s) shall be permitted. An acceptable structural analysis shall produce engineering demand parameters including peak story drift,

residual story drift, and peak floor acceleration as well as any local element demands, if required, and shall address nonlinear response including story drift localization.

A.17.3 Acceptance Criteria The probability that the functional recovery time calculated by the explicit functional recovery assessment exceeds the target functional recovery time acceptance criteria specified in Table A.1-1 shall not exceed 25% for the seismic ground motion hazard of Section A.6. The functional recovery time calculated by the explicit functional recovery assessment for comparison against the target functional recovery time acceptance criteria shall be taken as the time at which at least 80% of the structure's area supporting its basic intended function is restored.

A.17.4 Independent Review

A.17.4.1 Review Scope and Reviewer Qualifications An independent review of the explicit functional recovery assessment shall be performed by one or more individuals acceptable to the Authority Having Jurisdiction and possessing knowledge of the following items, with a minimum of one reviewer being a registered design professional. Independent review shall include, but not be limited to, review of the appropriateness of the following:

1. Explicit functional recovery assessment procedure selected.
2. Determination of the basic intended function of the structure in accordance with Section A.4. This shall include review of the determination of nonstructural systems, elements, components, equipment, and distribution systems deemed not required to support the basic intended function of the structure.
3. Structural analyses used in determination of the engineering demand parameters.
4. Determination or estimation of collapse fragility.
5. Structural and nonstructural fragility functions and their relationship to enhanced design interventions (if any are pursued).
6. Component damage states that trigger impact on functional recovery and the translation of those component damage states from component test data.
7. Elements, components, equipment, and distribution systems deemed rugged.
8. Structural and nonstructural repair time consequence functions and repair sequencing, including long-lead-time effects.
9. Functional consequences between multiple components and/or systems.
10. Temporary repair measures and alternatives.
11. Assumptions made in selecting and implementing impeding factors.

12. Structural and nonstructural component quantities.

The independent review shall not be performed by the functional recovery coordinator. Where independent review of other aspects of the analysis or design are required by Chapters 1, 16, 17, or 18, the independent review in accordance with this section shall be permitted to be performed by the same individual(s), provided the individual(s) meet the qualification requirements above.

A.17.4.2 Review Letter Upon completion of the review, the reviewer(s) shall provide the Authority Having Jurisdiction with a letter attesting to the following:

1. Scope of review performed.
2. Whether the reviewer(s) concur with the explicit functional recovery assessment and its applicability to design.
3. Whether the reviewer(s) concur that the explicit functional recovery assessment and design achieves the target functional recovery time.
4. Any items relating to the functional recovery analysis or design that require further resolution by the Authority Having Jurisdiction.

A.18 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

See Chapter 23 for the list of consensus standards and other documents that shall be considered part of this standard to the extent referenced in this appendix.

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2026 NEHRP RECOMMENDED SEISMIC PROVISIONS FOR NEW BUILDINGS AND OTHER STRUCTURES: PART 2, COMMENTARY

Part 2, Commentary provides commentary for the recommended changes in Part 1, Provisions. It is comprised of the new commentary to each proposed change contained in Part 1. The commentary of ASCE/SEI 7-22 for unchanged sections is not reproduced. Therefore, Part 2, Commentary should be used side-by-side with the ASCE/SEI 7-22 Commentary.

COMMENTARY TO CHAPTER 11, SEISMIC DESIGN CRITERIA

Revise Section C11.2 as follows:

C11.2 DEFINITIONS

BASE: The level at which the horizontal seismic ground motions are considered to be imparted to the structure. See section C12.7.3.1 for additional information.

Add the following to the end of Section C11.4:

C11.4 SEISMIC GROUND MOTION VALUES

In addition to the revised theoretical basis of the mapped values of the MCE_R ground motions, the 2026 NEHRP Recommended Provisions introduces changes to the geometric mean peak ground acceleration parameters intended for specific use to assess liquefaction hazards. The new liquefaction-targeted MCE_L peak ground acceleration, PGA_L , is defined in Section 11.2 and Section 11.3. Its use in investigation of geological hazards is defined in Section 11.8.3, with details on site-specific procedures for determining PGA_L added in Section 21.5.4. The new PGA_L is intended to be used in place of the MCE_G PGA_M as defined in previous editions of the NEHRP Recommended Provisions when assessing liquefaction triggering and its consequences. The mapped liquefaction-targeted MCE_L PGA_L ground motions differ considerably from the mapped MCE_G PGA_M ground motions in a number of ways. These differences include: (1) the use of probabilistic ground motions that are based on uniform liquefaction triggering hazard, rather than uniform ground shaking hazard; (2) the inclusion of the effects of magnitude M_w and ground motion duration on liquefaction triggering in the calculation of the mapped of PGA_L , rather than requiring the selection of a single controlling magnitude “consistent with the MCE_G peak ground acceleration”; and (3) no deterministic considerations for computing the MCE_L PGA_L . For other geotechnical considerations not related to seismic liquefaction, such as seismic slope stability assessments and evaluation of permanent ground deformations not caused by liquefaction, the existing PGA_M should continue to be used.

Revise Sections C11.4.3 and C11.4.5 as follows:

C11.4.3 Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

“Mapped” values of seismic parameters S_s , S_1 , S_{MS} , and S_{M1} are archived in the USGS Seismic Design Geodatabase at gridded locations across United States regions of interest and provided online by the USGS Seismic Design Web Service, for user-specified site location (latitude and longitude) and site class. The USGS web service spatially interpolates between the gridded values of

these parameters based on site location (latitude and longitude). Chapter 22 provides print copies of seismic parameters S_{MS} and S_{M1} for default site conditions. Seismic parameters S_{MS} and S_{M1} (and S_{DS} and S_{D1}) incorporate site effects, eliminating the need for the tables of site factors F_a and F_v of ASCE 7-16.

With the introduction of the 2018 USGS National Seismic Hazard Model in ASCE 7-22, the Central and Eastern United States (CEUS) seismic parameters are derived using different ground motion models than the Western United States (WUS) active crustal region (ACR). The WUS ACR uses the NGA-West 2 ground motion models (Bozorgnia et al, 2014) while the CEUS uses the NGA-East ground motion models (Goulet et al, 2018, 2021). The NGA-East (Goulet et al. 2021) project demonstrated that WUS ACR ground motion models were inappropriate for the CEUS because of the different seismological and geological settings in that region compared to the WUS ACR. NGA-West 2 ground motion models indicate that seismic waves amplify significantly on soft soils under linear (weak shaking) conditions, with the observed trend being that the softer the soils, the more significant the ground motion amplification will be, especially at long-periods. This has been historically represented in the S_{D1} term by the long-period site factor F_v . The NGA-East ground motion models also show that seismic waves are amplified at soft soil sites, but to a much lesser extent than there would be for a site with a similar 30 m average shear wave velocity (V_{s30}) in the western US. For very soft sites, Site Class E, nonlinearity reduces site amplification for short-period spectral ordinates, which produces de-amplification with respect to Site Class D spectral ordinates, as shown in Figure C11.4-X.

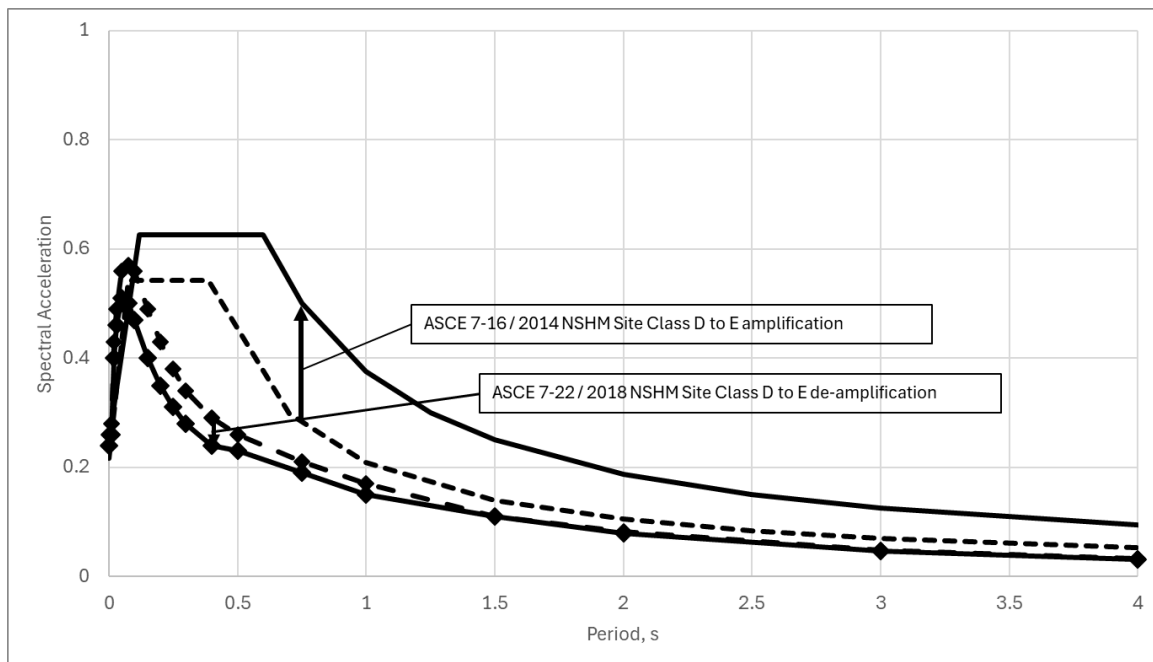


Figure C11.4-X: ASCE 7-16 vs. ASCE 7-22 Site Class D and E Response Spectra Comparison for Knoxville, TN

The seismic parameters for the CEUS are derived from the NGA-East ground motion models with elastic (Stewart et al, 2020) and inelastic (Hashash et al, 2020) site amplification models. Hashash et al (2020) recommended that the nonlinear site amplification applicability be limited to sites with $200 \text{ m/s} < V_{s30} < 2,000 \text{ m/s}$ due to a sparsity of simulated data and the limitations inherent in 1-D simulations outside of that V_{s30} range. The lower end of the Site Class D range is 210 m/s, meaning that the nonlinear site amplification model is not recommended for many Site Class DE sites and all Site Class E sites. The NSHM was developed for all site classes, which led to extrapolating the nonlinear model beyond this recommended limit.

In light of concerns regarding deamplification of the seismic hazard parameters for Site Classes DE and E with respect to Site Class D for CEUS sites, a re-examination was performed of how the CEUS site amplification models (Stewart et al. 2020; Hashash et al. 2020) were implemented. As part of this re-examination, the impact of increasing the site amplification for Site Class DE and E sites by a half standard deviation from the nonlinear site amplification model was evaluated. A half standard deviation above the mean bias was applied previously in the 2015 NEHRP Provisions and ASCE 7-16 when the site factor tables were updated for Site Class E due to “the relatively modest amount of data for this site condition [E].” This was not included in the 2018 USGS hazard model because site amplification was explicitly considered in the hazard calculations, including the effects of site condition on ground motion standard deviation, as opposed to the prior practice of incorporating site amplification through a single factor.

On average, the impact of a half standard deviation bias adjustment to the nonlinear site amplification produced ground motion parameters in the CEUS that were similar to those produced by the models when Site Class D is used. Given that result and the recommended limitation on the nonlinear site amplification mode for soft soil sites from Hashash et al. (2020), a floor has been added to the seismic parameters for Site Classes DE and E. In effect, this floor conservatively estimates seismic parameters in the CEUS for Site Class DE and E by using seismic parameters for Site Class D. This was seen as a reasonable method to address uncertainty. The impact to CEUS sites varies depending on the level of seismic hazard. These impact variations occur because the influence of the nonlinear component of the amplification model increases as seismic hazard increases. Accordingly, the impact is somewhat limited for sites where the seismic hazard parameters are less than those required to classify as Seismic Design Categories D. For those sites, the seismic hazard parameters typically do not increase by more than 10% for Site Class DE sites and 20% for Site Class E sites. Rarely does this change increase the Seismic Design Category assigned. However, for regions where the seismic hazard parameters are larger than those required to assign Seismic Design Category D, such as Memphis and Charleston, the effect of this floor is more significant, where spectral parameters can increase by more than 50%. Nevertheless, the Seismic Design Category does not change in these regions.

C11.4.4 Design Spectral Acceleration Parameters Design in ASCE 7 (e.g., Chapter 12) is performed for earthquake demands that are $2/3$ of MCE_R ground motions. As set forth in Section 11.4.4, two additional parameters S_{DS} and S_{D1} are used to define design spectral accelerations.

Values of the seismic parameters S_{DS} and S_{D1} ($2/3S_{MS}$ and $2/3S_{M1}$) are provided online by the USGS Seismic Design Web Service for user-specified site location (latitude and longitude) and site class. Values of the seismic parameters S_{DS} and S_{D1} provided by the USGS are based on the multi-period design response spectrum (Section 11.4.5.1) of the site of interest and the requirements of Section 21.4 (for determining values of S_{DS} and S_{D1} from a site-specific design response spectrum).

C11.4.5 Design Response Spectrum

The design response spectrum, (and MCER response spectrum of Section 11.4.6) are defined by either (1) a multi-period response spectrum (Section 11.4.5.1) or (2) a two-period response spectrum (Section 11.4.5.2), unless the design is based on site-specific ground motions (Section 21.3). The multi-period design response spectrum (MPRS) provides a “best available science” representation of spectral response accelerations for design use for structure fundamental periods of 0.2 seconds and higher. If “best available science” representation of spectral response accelerations was the only consideration, the MPRS would also be the preferred characterization for structures with periods less than of 0.2 seconds. However, it is also recognized that the designer is generally using very simplistic and low accuracy methods to estimate effective structure periods, ignoring a wide range of boundary conditions, modeling choices and non SFRS contributions to strength and stiffness. This is particularly true for short period structures. When the considerations of representative response spectral acceleration and representative estimation of period are combined, the MPRS is thought to be a best representation for mid-range and long period structures, while a two-period spectrum is the preferred method for very short period structures.

One of the reasons that the S_{DS} plateau and not the MPRS is preferred for very-short-period structures and nonstructural components is because the MPRS represents free-field motion, which can provide considerably conservative estimates of base-of-structure motion at periods less than 0.2 s. Factors that can reduce the free-field motion include base slab averaging, foundation damping, foundation sliding and foundation uplift whose reduction effects are believed to be more pronounced at periods less than 0.2 s. Furthermore, because of anticipated nonlinear behavior, most actual structures subject to strong earthquake motions have effective secant periods that are greater than 0.2 s, even though the procedures of Chapter 12 might result in an estimated period of less than 0.2 s.

Studies focused on collapse of short period structures (FEMA P-2139) brought forth another reason for disregarding computed response accelerations at periods below 0.2 s. The spectral displacements associated with those very short periods are on the order of small fractions of an inch for spectral accelerations of 100% gravity, and those small displacements are not associated with collapse of gravity load resisting systems, which is the limit state of interest for structures.

Due to these reasons, high accelerations spikes at periods less than 0.2 s are permitted to be capped at S_{DS} and are not included as part of seismic design ground motions when using the MPRS, similar to the treatment when using the two-period spectrum approach. It is the judgement of the Provisions Update Committee that at periods equal to or greater than 0.2 s, it is considered acceptable to treat the MPRS as representing the base-of-structure motion because it has been

observed that base-of-structure earthquake strong motion records for spectral periods greater than 1 s reasonably match nearby free-field ground motion records and that for periods between 0.2 and 1.0 s, the MPRS free-field motions are generally greater than the base-of-structure motions. The differences between them is primarily attributed to base slab averaging and embedment reductions that are permitted to be accounted for in accordance with the procedures of Chapter 19 when Chapter 16 type analyses are performed. The two-period design response spectrum is the same as that of ASCE 7-16, which relies on a simpler characterization of the frequency content of design ground motions (Figure 11.4-1) based on the values of seismic parameters S_{DS} and S_{D1} (and T_L). The two-period design response spectrum is retained in ASCE 7-22 as an alternative characterization of ground motions of nonstructural components, for seismic design using the equivalent static force procedure of Section 12.8 and for scaling of base shears when using the modal response spectrum method of Section 12.9.1.

For both the multi-period spectra and the two-period spectra, a requirement is added that in the Central and Eastern United States, the seismic design parameters for Site Classes DE and E shall not be taken less than those values obtained for Site Class D for reasons discussed in Section C11.4.3.

Revise Sections C11.4.5.1 and C11.4.5.2 as follows:

C11.4.5.1 Multi-Period Design Response Spectrum

Sets of multi-period MCE_R response spectra (5% damping) at 22 response periods (0.0 s, 0.01 s, 0.02 s, 0.03 s, 0.05 s, 0.075 s, 0.1 s, 0.15 s, 0.2 s, 0.25 s, 0.3 s, 0.4 s, 0.5 s, 0.75 s, 1.0 s, 1.5 s, 2.0 s, 3.0 s, 4.0 s, 5.0 s, 7.5 s, and 10 s) are archived in the USGS Seismic Design Geodatabase at gridded locations across United States regions of interest. A USGS Seismic Design Web Service, for the site location and site class of interest, spatially interpolates between the gridded sets of multi-period MCE_R response spectra based on site location (latitude and longitude). Multi-period design response spectrum is constructed from two-thirds of these values by linear interpolation for response periods less than 10 s and by extrapolation for response periods greater than 10 s.

At response periods beyond 10 s, values of the multi-period design response spectrum are assumed to decrease from the value of the design response spectrum at 10 s as the inverse of the period, T , where T is less than T_L and/or as inverse of the square of the period, T^2 , where T is greater than T_L , essentially following the same approach as that used to construct the two-period spectrum (Section 11.4.5.2) at very long-periods. For periods greater than 10s, procedures from Assadollahi, Pezeshk, and Campbell (2023) are recommended. For response periods greater than 10 s, if the long-period transition period, T_L , is greater than 10 s, the spectral acceleration S_a , is assumed to decrease from the value of the design response spectral acceleration S_a at 10s as the inverse of the period, T , up to T_L , and for longer periods than T_L , as the inverse of the square of the period, T^2 . For response periods greater than 10 s, if the long-period transition period, T_L , is less than 10 s, the spectral acceleration, S_a , for periods greater than 10 s will decrease as the inverse of the square of the period, T^2 .

C11.4.5.2 Two-Period Design Response Spectrum

The two-period design response spectrum (Figure 11.4-1) consists of several segments. The constant-acceleration segment covers the period band from T_0 to T_s ; response accelerations in this band are constant and equal to S_{DS} . The constant-velocity segment covers the period band from T_s to T_L , and the response accelerations in this band are proportional to with the response acceleration at a 1-s period equal to S_{D1} . The long-period portion of the design response spectrum is defined on the basis of the parameter T_L , the period that marks the transition from the constant-velocity segment to the constant-displacement segment of the design response spectrum. Response accelerations in the constant-displacement segment, where $T \geq T_L$, are proportional to $1/T^2$. Values of T_L are provided on maps in Figures 22-14 through 22-17.

The T_L maps were prepared following a two-step procedure. First, a correlation between earthquake magnitude and T_L was established. Then, the mean magnitude from deaggregation of the ground-motion seismic hazard at a 2 s period was mapped.

Details of the procedure and the rationale for it are found in Assadollahi, Pezeshk, and Campbell (2023). The procedure in Assadollahi, Pezeshk, and Campbell (2023) was used; however, the disaggregation used to develop the maps were updated with finer grid point spacing of 0.05 degrees for Conterminous US and 0.02 for Hawaii using the probabilistic MCE_R (risk-targeted) ground motions.

Revise Section C11.4.7 as follows:

C11.4.7 Site-Specific Ground Motion Procedures

Site-specific ground motions are permitted for design of any structure and are required for design of certain structures and certain site soil conditions. The objective of a site-specific ground motion analysis is to determine ground motions for local seismic and site conditions with higher confidence than is possible using the general procedure of Section 11.4.

As noted earlier, the site-specific procedures of Chapter 21 are the same as those used by the US Geological Survey (USGS) to develop the mapped values of MCE_R ground motions. Unless significant differences in local seismic and site conditions are determined by a site-specific analysis of earthquake hazards, site-specific ground motions would not be expected to differ significantly from those of the mapped values of MCE_R ground motions prepared by the USGS.

Revise Section C11.6 as follows:

C11.6 SEISMIC DESIGN CATEGORY

Seismic design categories (SDCs) provide a means to step progressively from simple, easily performed design and construction procedures and minimums to more sophisticated, detailed, and costly requirements as both the level of seismic hazard and the consequence of failure escalate. The SDCs are used to trigger requirements that are not scalable; such requirements are either on or off. For example, the basic amplitude of ground motion for design is scalable—the quantity simply increases in a continuous fashion as one moves from a low hazard area to a high hazard area. However, a requirement to avoid weak stories is not particularly scalable. Requirements such as this create step functions. There are many such requirements in the standard, and the SDCs are used systematically to group these step functions. (Further examples include whether seismic anchorage of nonstructural components is required or not, whether particular inspections will be required or not, and structural height limits applied to various seismic force-resisting systems.)

In this regard, SDCs perform one of the functions of the seismic zones used in earlier US building codes. However, SDCs also depend on a building's occupancy and, therefore, its desired performance.

In developing the ground-motion target limits and design requirements for the various seismic design categories, the equivalent modified Mercalli intensity (MMI) scale was considered. There are now correlations of the qualitative MMI scale with quantitative characterizations of ground motions. The reader is encouraged to consult any of a great many sources that describe the MMIs. The following list is a coarse generalization:

MMI V: No real damage

MMI VI: Light nonstructural damage

MMI VII: Hazardous nonstructural damage

MMI VIII: Hazardous damage to susceptible structures

MMI IX: Hazardous damage to robust structures

When the historical design philosophy was adopted from the 1997 NEHRP provisions and commentary (FEMA 1997a, b), the upper limit for SDC A was set at roughly one-half of the lower threshold for MMI VII, and the lower limit for SDC D was set at roughly the lower threshold for MMI VIII. However, the lower limit for SDC D was more consciously established by equating that design value (two-thirds of the MCE) to one-half of what had been the maximum design value in building codes over the period of 1975 to 1995. As more correlations between MMI and numerical representations of ground motion have been created, it is reasonable to make the following correlation between the MMI at MCE ground motion and the seismic design category (all this discussion is for ordinary occupancies):

MMI V: SDC A

MMI VI: SDC B

MMI VII: SDC C

MMI VIII: SDC D

MMI IX-X: SDC E

An important change was made to design philosophy for the determination of SDC relative to the philosophy used to determine the predecessor seismic performance categories (SPC). Earlier editions of the NEHRP provisions used the peak velocity-related acceleration, A_v , to determine a building's seismic performance category. However, this coefficient does not adequately represent the damage potential of earthquakes on sites with soil conditions other than rock. Consequently, the 1997 NEHRP provisions (FEMA 1997a) adopted the use of response spectral acceleration parameters S_{DS} and S_{D1} , which include site soil effects for this purpose.

Except for the lowest level of hazard (SDC A), the SDC also depends on the Risk Categories. For a given level of ground motion, the SDC is typically one category higher for Risk Category IV structures than for lower risk structures in ASCE 7-98 through ASCE 7-22. This increase in SDC has the effect of increasing the confidence that the design and construction requirements can deliver the intended performance in the extreme event. As an exception, SDC D does not increase to SDC E, where the special requirements of SDC E (and SDC F) are deemed unnecessary for Risk Category IV structures unless located at sites of very high seismicity (i.e., $S_{DS} \geq 1.5g$, $S_{D1} \geq 1.0g$).

Note that the historical SDC thresholds are at the design level, defined as two-thirds of the MCE level. Also recall that the MMIs are qualitative by their nature and that the above correlation will be more or less valid, depending on which numerical correlation for MMI is used. The numerical correlations for MMI roughly double with each step, so correlation between design earthquake ground motion and MMI is not as simple or convenient.

In sum, at the MCE level, SDC A structures should not see motions that are normally destructive to structural systems, whereas the MCE level motions for SDC D structures can destroy vulnerable structures. The grouping of step function requirements by SDC is such that there are a few basic structural integrity requirements imposed at SDC A, graduating to a suite of requirements at SDC D based on observed performance in past earthquakes, analysis, and laboratory research.

The nature of ground motions within a few kilometers of a fault can be different from more distant motions. For example, some near-fault motions have strong velocity pulses, associated with forward rupture directivity, that tend to be highly destructive to irregular structures, even if they are well detailed. For ordinary occupancies, the boundary between SDCs D and E is set to define sites likely to be close enough to a fault that these unusual ground motions may be present. The additional

design criteria imposed on structures in SDCs E and F specifically are intended to provide acceptable performance under these very intense near-fault ground motions.

The values of S_{DS} ($2/3 S_{MS}$) and S_{D1} ($2/3 S_{M1}$) of the boundary between SDC D and SDC E (SDC F) sites define the strongest intensity of MCE_R shaking ($SDC D_{max}$) required for quantification of building seismic performance factors (e.g., R factor) of a new SFRS proposed for Table 12.2-1 and for verification of an alternative SFRS (i.e., Section 12.2.1.1). As such, the boundary between SDC D and SDC E (SDC F) effectively defines the strongest intensity of MCE_R ground motions for which Risk Category II structures are expected to meet the 10% target reliability (i.e., 10% probability of collapse given MCE_R ground motions) and for which Risk Category IV structures are expected to meet the 2.5% target reliability of Table 1.3-2. See also commentary to Section 12.2.1.1.

Since their introduction into the Provisions, in the 1997 edition, SDC has been determined on a site-specific (i.e. based on level of seismicity) basis, and has directly considered site class. For structures located in regions where high intensity ground motions were not expected, a benefit of this approach was to limit requirements for higher levels of seismic detailing and design only to those sites where site amplification of motion was likely to produce intense shaking. However, this approach led to significant variation in both design and construction requirements for projects on different sites but within the same communities. While an economical approach, limiting more restrictive criteria to only those structures that needed it, it resulted in non-uniform practice requirements within regions, making it difficult both for designers and contractors to adopt appropriate design and construction practices on those sites where it was truly needed. In addition, as the USGS developed periodic updates to the seismic hazard and design value maps, this created considerable instability in the required SDC in a region between one code edition and the next.

Starting in the 2028 edition, the SDC map in the standard was developed by determining the design category using the same traditional bounds (shown in Table C11.6-X) but with the default site class and then using geographic smoothing to adjust boundaries to ensure that SDC lines do not cross through major populated areas, with the intent to have consistency of design and construction within a community. Contours generated using the rules in Table C11.6-X are used as the starting point, but adjustments were made to follow county and state lines everywhere except Alaska. The adjustments to the maps were typically modest using a $\pm 15\%$ adjustment to the thresholds on the values in Table C11.6-x to advise how changing the SDC break lines on the map would compare to the baseline maps. In almost every case the resulting SDC in the provisions align with the SDC you would get by calculating using the table values $\pm 15\%$, with a few exceptions in areas of sparse population. The boundary between SDC D and E/F was the only boundary not adjusted to county and state lines. In Alaska the SDC categories were not adjusted by county due to the extreme size of some of the counties which have 3 or 4 different Seismic Design Categories within a single county. While the population in most of these counties is sparse, significant infrastructure exists in these remote locations.

Table C11.6-X: Baseline SDC thresholds used in maps**Table 11.6-1: Seismic Design Category Based on Short-Period Response Acceleration Parameter.**

Value of S_{DS}	Risk Category	
	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.50 \leq S_{DS} < 1.50$	D	D
$1.50 \leq S_{DS}$	E	F

Table 11.6-2: Seismic Design Category Based on 1 s Period Response Acceleration Parameter.

Value of S_{D1}	Risk Category	
	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1} < 1.00$	D	D
$1.00 \leq S_{D1}$	E	F

Since the introduction of SDC into ASCE 7-98 through ASCE 7-22, the SDC was determined without consideration of the building's period. Structures were assigned to a SDC based on the more severe condition determined from 1 s acceleration and short-period acceleration. For ASCE 7-28, a similar approach is used to determine the SDC maps. This assigning was done for several reasons. Perhaps the most important of these is that it is often difficult to estimate precisely the period of a structure using the default procedures contained in the standard. Consider, for example, the case of rigid wall-flexible diaphragm buildings, including low-rise reinforced masonry and concrete tilt-up buildings with either untopped metal deck or wood diaphragms. The formula in the standard for determining the period of vibration of such buildings is based solely on the structural height, h_n , and the length of wall present. These formulas typically indicate very short periods for such structures, often on the order of 0.2 s or less. However, the actual dynamic behavior of these buildings often is dominated by the flexibility of the diaphragm—a factor neglected by the formula for approximate fundamental period. Large buildings of this type can have actual periods on the order of 1 s or more. To avoid misclassifying a building's SDC by inaccurately estimating the fundamental period, the standard generally required that the more severe SDC determined on the basis of short- and long-period shaking be used.

Local or regional jurisdictions enforcing building regulations may desire to consider the effect of the maps, typical soil conditions, and seismic design categories on the practices in their jurisdictional areas. For reasons of uniformity of practice or reduction of potential errors, adopting ordinances could stipulate particular values of ground motion, or particular seismic design categories for all or part of the area of their jurisdiction. For example,

- An area with a historical practice of high seismic zone detailing might mandate a minimum SDC of D regardless of ground motion
- A jurisdiction with low variation in ground motion across the area might stipulate particular values of ground motion rather than requiring the use of maps.

This mapping was validated in 2025 for the criteria described above: coherence of practice in metropolitan areas, county Seismic Design Category corresponding to the predominant category that would be obtained on individual sites within the county, etc. While this validation is specific to the 2023 USGS seismic hazard model, it is expected that this mapping will remain acceptable with few or no changes deemed necessary for subsequent editions of ASCE 7. To ensure this acceptability, review in concert with the adoption of each new USGS seismic hazard model is appropriate. Such a review would entail the use of judgment as to whether any changes warrant a change in Seismic Design Category (with the corresponding change in practice).

Revise Section C11.8.1 as follows:

C11.8.1 Site Limitation for Known Active Surface Fault Zones

Local or state governments have the option of designating areas where the potential for surface fault rupture must be considered under these Provisions. Typically, these are areas where the fault is well enough defined and has large enough movement potential to affect the design of structures based on these Provisions (i.e., faults that can be characterized per Section 11.8.4; not buried or concealed faults that are difficult to trace at the ground surface). For example, in California, Alquist-Priolo Maps (CGS, 2018) are published indicating zones where a sufficiently well-defined hazard for surface fault rupture exists.

The Registered Design Professional (RDP) for the geotechnical investigation report should consult the maps adopted by the AHJ to determine if the potential for surface fault rupture needs to be considered at a structure. For the purposes of these Provisions, the requirement to consult maps of surface fault rupture hazard is limited to SDC D through F at the default site class because sufficiently hazardous active surface fault zones are not currently known to be present outside of these boundaries. The default site class is used because the boundary between SDC C and D at the default site class aligns well with the location of known active faults in the US and if new faults are found in the future, the boundaries could conform accordingly.

Generally, if a structure is within a Known Active Surface Fault Zone, the structure should not be constructed unless (1) the RDP can show through a site investigation that the surface fault rupture

hazard does not exist at the specific structure of interest, or (2) the structure and its foundation are designed to accommodate surface fault rupture in accordance with Section 12.13.10, either with or without a supporting site investigation. The MCFD at the foundation is that which will occur after accounting for any ground modification measures that are part of site preparation. Because surface fault rupture often results in many splays of distributed fault rupture over wide areas, it is not always necessary to avoid construction in areas where relatively minor movement is expected away from the principal fault or where a fault is very small. However, structures should not be placed on the principal traces of large faults (e.g., on the main trace of the San Andreas fault) because of the potential for very large fault displacements to occur.

The determination of the MCFD resulting from surface fault rupture is generally performed by the RDP in one of two ways, (1) by modeling the occurrence of the earthquake utilizing published information about the size of the fault, slip rate, other pertinent factors, and empirical fault displacement models (often called the “earthquake approach”), or (2) a site-specific geologic investigation to locate and measure the actual surface displacements that have previously occurred onsite (often called the “displacement approach”). In determination method 1, the actual location of surface fault traces onsite may be determined by geologic investigation, or it could be conservatively assumed that the structure is subject to the principal fault displacement. A structure-to-fault distance of 50 feet is often used to define the area where significant off-fault deformation could occur but can be more or less appropriate depending on site-specific conditions.

Use of a deterministic criteria was deemed appropriate because risk-based procedures have not yet been fully developed and accepted. Using the higher of a deterministic value and a risk-based design fault displacement was considered by the committee; however, the deterministic values were determined to almost always be greater than the risk-based values. However, a deterministic percentile that varies based on the fault slip rate was selected for consistency with the risk concepts of this Standard. In the future, the deterministic criteria may be replaced with a risk-based MCFD. The 50th- to 84th-percentile displacement for a rupture of the fault in question under a plausible scenario event is used due to the large uncertainty in fault displacements when using empirical fault displacement models (determination method 1 above). The slip rates and deterministic percentiles selected were originally developed by the Federal Energy Regulatory Commission for dam design. The median value may be used if a site-specific investigation, such as fault trenching, has measured previous fault displacements at the proposed structure (determination method 2 above).

While deterministic fault displacement has historically been evaluated for a rupture scenario involving the “full fault length,” this should not be construed to require the evaluation of unlikely multi-segment and multi-fault rupture scenarios that are increasingly considered in modern earthquake rupture forecasts such as ERF23. The committee considered a specific criterion to determine which multi-segment/fault scenarios need to be considered for the MCFD, but felt this was overly complicated to define at this time. For this reason, it is left to the RDP, with peer-reviewer concurrence, to identify a credible scenario event for use in the derivation of displacements. Some methods to evaluate whether multi-segment/fault scenarios should be considered include (1) querying an earthquake rupture forecast modified to include only scenario that rupture to the ground surface for the scenarios with the highest frequency of occurrence, (2) deaggregating a probabilistic

fault displacement hazard analysis for the rupture scenarios contributing the greatest to the hazard, and (3) using scenario earthquakes that contribute 10% or more of the largest probabilistic contributor from a ground motion probabilistic seismic hazard analysis.

Both principal and distributed fault movement can be estimated (with determination methods 1 and 2 above). The MCFD value is reported for a single fault trace, so if multiple fault traces are present at a structure, each trace may have an MCFD associated with it. If no principal faults are shown to be located at a structure, and distributed displacement is not expected, the MCFD is zero and the provisions of Section 12.13.10.1 will not require the structure to be designed for fault rupture. Various methods are available to perform site investigations (e.g., IAEA, 2022; California Geological Survey (CGS) Special Publication 42, 2018) and estimate the magnitude of ground deformation (e.g., Lavrentiadis and Abrahamson, 2023; Moss et al., 2024; Kuehn et al., 2024; Chiou et al., 2025). Distributed fault movement can also be estimated empirically (e.g., Petersen et al., 2011). Tools to assist in the evaluation of the MCFD are available from the Fault Displacement Hazard Initiative at the B. John Garrick Institute for the Risk Sciences of the University of California at Los Angeles (UCLA) and can be accessed at the following website:

<https://www.risksciences.ucla.edu/nhr3/fdhi/models>. The website opens to a page that provides direct links to these publications, which are also part of the Earthquake Spectra journal's "Fault Displacement Hazard Analysis Special Collection"; these papers and others in this special collection are available at https://journals.sagepub.com/topic/collections-eqs/eqs-1-fault_displacement_hazard_analysis_special_collection/eqs). An Excel workbook can be downloaded from the link at the bottom of this website. For specified user inputs, the workbook computes MCFD values for the 50th and 84th percentile levels for the four models.

The following commentary about surface fault rupture is from the California Geological Survey (CGS) Special Publication 42.

Surface fault rupture is the result of fault movement that breaks to the surface of the earth either suddenly during earthquakes, or slowly due to a process known as fault creep and is the result of tectonic movement that originates deep in the Earth. Surface fault rupture is different from other types of earthquake-related ground deformation, such as that caused by soil liquefaction or earthquake-triggered landslides. The energy released during an earthquake is a direct result of fault rupture at depth, and when that rupture extends to the ground surface it manifests as displacements expressed as fractures, fissures, and related tectonic deformation.

Surface fault rupture poses a hazard to structures because one side of the fault moves relative to the other, which can severely damage structures. In extreme cases, this damage can result in the structural collapse of a building, potentially resulting in injuries or loss of life. In less extreme cases, structural damage may render a building uninhabitable and require costly repairs. Earthquakes around the world have demonstrated the potential for extensive damage to structures due to surface fault rupture and the hazard it poses to life and property.

Once the potential for surface fault rupture ground deformation at a structure has been characterized, the Geotechnical Engineering RDP should work with the Structural Engineering RDP to

provide foundation design recommendations that mitigate the effects of surface fault rupture to the extent it is feasible based on the magnitude of the MCFD and the type of structure. Many methods of foundation design are feasible for mitigating the effects of small to modest levels of ground deformation. The most common foundation design is a strong, stiff mat foundation that causes a structure to respond to the fault displacement with essentially rigid body movement, thereby protecting the superstructure from significant damage and allowing for potential repair. Mitigation strategies with flexible connections to critical utilities and/or automatic utility shutoffs. Mitigation strategies are described in publications such as Oettle and Bray (2013a). While deep foundations are a common foundation design mitigation strategy for liquefaction-induced ground deformation, deep foundations are often not appropriate to mitigate surface fault rupture and may actually increase damage to the superstructure by tying the structure to both sides of the fault and transmitting the displacement into the superstructure. Note that while placement of fill over a fault may be a valid design recommendation to mitigate the effects of surface fault rupture, the hazard of surface fault rupture will still be regarded as existing even though the fill is presently unfaulted. If analysis of fault propagation through soil and its interaction with a structure and its foundation is necessary, methods of analysis are described in publications such as Oettle and Bray (2013b).

Note that certain local jurisdictions (e.g., California, Utah), have specific laws that address surface fault rupture under certain circumstances which may control.

Revise Section C11.8.3 as follows:

C11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

Provisions for computing peak ground acceleration (PGA) for geologic and geotechnical hazards evaluation were originally introduced in this section in ASCE 7-16. At that time, the explicit requirement was stated that liquefaction and other geologic hazards must be evaluated for maximum considered geometric mean (MCE_G) peak ground acceleration (PGA_M). In the *2026 NEHRP Recommended Provisions*, a new design parameter for liquefaction hazard evaluation is introduced, and liquefaction hazards must now be evaluated for the *liquefaction-targeted* maximum considered earthquake (MCE_L) geometric mean peak ground acceleration (PGA_L), while all other geohazards not related to seismic liquefaction are still evaluated using PGA_M . Values of both the PGA_L and PGA_M parameters are archived in the USGS Seismic Design Geodatabase at gridded locations across United States regions of interest. Values are provided online by the USGS Seismic Design Web Service for user-specified site location (latitude and longitude) and site class, by spatially interpolating between the gridded values of PGA_L or PGA_M based on site location.

PGA_L and PGA_M Provisions.

In addition to the fact that PGA_L is now based on uniform liquefaction hazard (rather than the uniform ground shaking hazard basis of PGA_M), PGA_L differs importantly from PGA_M in its use in many typical liquefaction triggering and consequence models, which requires selection of a representative magnitude M_w to capture the effects of duration on liquefaction potential. Recognizing the

considerable uncertainty in selecting just a single M_w value from the wide range of magnitudes that contribute to MCE ground shaking, the liquefaction-targeted PGA_L is determined and mapped to account for the contributions of all earthquake magnitudes on the liquefaction hazard evaluation. As a result, PGA_L should be used in most current liquefaction models with magnitude scaling or duration weighting factors set to their reference levels of 1.0, without the need for selecting a single representative M_w value. For all other geological and geotechnical hazards, such as the evaluation of seismic slope stability and soil strength loss, the MCE_G PGA_M shall continue to be used, with the definition of PGA_M as defined in Section 21.5 of these provisions. The continued use of the MCE_G PGA_M based on uniform ground shaking hazard is due to the fact that analogous probabilistic studies for other non-liquefaction geohazards have yet to be conducted or implemented in these provisions.

There is an important difference in the derivation of the PGA_L and PGA_M parameters and the MCE_R spectral response parameters that are derived in these provisions. As has been the case since ASCE 7-16, MCE_R parameters have been derived for “maximum direction shaking”, and are risk-based rather than hazard-based. However, the PGA_L and PGA_M parameters have been derived based on the geometric mean of the two horizontal components of motion. The geometric mean has been used to derive the PGA_M parameter since its definition in ASCE 7-10, rather than the PGA for the maximum direction shaking to ensure that there is consistency between the determination of PGA and the basis of the simplified empirical field procedure for estimating liquefaction potential based on results of standard penetration tests (SPTs), cone penetrometer tests (CPTs), and other similar field investigative methods.

When these liquefaction triggering correlations were originally derived, the geomean (or a similar metric) of peak ground acceleration at the ground surface was used to identify the cyclic stress ratio for sites with or without liquefaction. The resulting envelopes of data define the liquefaction cyclic resistance ratio (CRR). Rather than reevaluating these case histories for the “maximum direction shaking,” it was decided to develop maps of the geomean PGA and to continue using the existing empirical methods. To ensure continuity for the *2026 NEHRP Recommended Provisions*, the MCE_L PGA_L is also derived from the geometric mean PGA .

Liquefaction Evaluation Requirements. Beginning with ASCE 7-02, it has been the intent that liquefaction potential be evaluated at MCE ground motion levels. There was ambiguity in the previous requirement in ASCE 7-05 as to whether liquefaction potential should be evaluated for the PGA associated with the MCE or for the design earthquake (i.e., 2/3 of the MCE PGA). Paragraph 2 of Section 11.8.3 of ASCE 7-05 stated that liquefaction potential would be evaluated for the design earthquake; it also stated that in the absence of a site-specific study, peak ground acceleration shall be assumed to be equal to $S_s/2.5$ (S_s is the MCE short-period response spectral acceleration on Site Class B rock). There has also been a difference in provisions between ASCE 7-05 and the 2006 edition of the International Building Code, in which Section 1802.2.7 stated that liquefaction shall be evaluated for the design earthquake ground motions, and the default value of peak ground acceleration in the absence of a site-specific study was given as $S_{DS}/2.5$ (S_{DS} is the short-period site-adjusted design response spectral acceleration). Item 2 of Section 11.8.3 requires explicitly that liquefaction triggering and its consequences be evaluated based on the MCE_L peak ground acceleration PGA_L , and Item 2 of Section 11.8.3 requires explicitly that other geohazards, such as

seismically-induced permanent ground displacement not related to soil liquefaction, be evaluated based on the MCE_G peak ground acceleration.

The explicit requirement to evaluate liquefaction for MCE ground motion, rather than to design earthquake ground motion, addressed an issue that had existed and periodically been discussed since the design earthquake concept was first suggested in the 1990s. The design earthquake ground motion was obtained by multiplying the MCE ground motion by a factor of 2/3 to account for a margin in capacity in most buildings. Various calibration studies at the time of code development concluded that for the design earthquake, most buildings had a reserve capacity of more than 1.5 relative to collapse. This reserve capacity allowed the spectral accelerations for the MCE to be reduced using a factor of 2/3, while still achieving safety from collapse. However, liquefaction potential is evaluated at the selected MCE_L peak ground acceleration and is typically determined to be acceptable if the factor of safety is greater than 1.0, meaning that there is no implicit safety margin on liquefaction potential. By multiplying peak ground acceleration by a factor of 2/3, liquefaction would be assessed at a lower effective 50-year probability of liquefaction (or probability of non-exceedance of FS_L) than the MCE_L targets (1.5% over a 50-year period).

As noted at the start of this commentary, in the 2026 NEHRP *Recommended Provisions*, a new design parameter for liquefaction hazard evaluation is introduced, and liquefaction hazards must now be evaluated for the *liquefaction-targeted* maximum considered earthquake (MCE_L) geometric mean peak PGA_L , while all other geohazards not related to liquefaction are still evaluated using geometric mean PGA_M . Item 4 of Section 11.8.3 lists the various potential consequences of liquefaction, seismically induced permanent ground displacement, and soil strength loss that must be assessed; soil downdrag and loss in lateral soil reaction for pile foundations are additional consequences that have been included in this paragraph. This section of the new provisions, as in previous editions, does not present specific seismic criteria for the design of the foundation or substructure, but Item 4 does state that the geotechnical report must include discussion of possible measures to mitigate these consequences.

A resource document was prepared in support of the revisions to Section 11.8.3 in the 2009 NEHRP *Recommended Provisions*. The resource document “Evaluation of Geologic Hazards and Determination of Seismic Lateral Earth Pressures” includes a summary of methods that were being used to evaluate liquefaction potential and the limitations of these methods. This summary appears as Resource Paper RP 12 in the 2009 NEHRP *Recommended Provisions* (FEMA 2009). A more recent resource document, “State of the Art and Practice in the Assessment of Earthquake-Induced Soil Liquefaction and Its Consequences,” by the National Academies of Sciences, Engineering, and Medicine (2021), provides updated information regarding liquefaction triggering and consequences evaluation. This updated document includes supporting information for these provisions, including the need for and benefit of probabilistic liquefaction hazard analysis (PLHA). These resource documents summarize alternatives for evaluating liquefaction potential, methods for evaluating the possible consequences of liquefaction (e.g., loss of ground support and increased lateral earth pressures), and methods of mitigating the liquefaction hazard. These resource documents also identify alternate methods of evaluating liquefaction hazards, such as analytical and physical

modeling. Reference is made to the use of nonlinear effective stress methods for modeling the buildup in pore water pressure during seismic events at liquefiable sites.

Evaluation of Seismic Lateral Earth Pressures. The seismic lateral earth pressure on basement and retaining walls during earthquake ground shaking is considered to be an earthquake load, E , for use in design load combinations. This seismic lateral earth pressure is superimposed on the preexisting static lateral earth pressure. The preexisting static lateral earth pressure is considered to be an H load.

Embedded (or basement) walls and retaining walls must be designed for both inertial and kinematic seismic loading as appropriate. Two primary factors may contribute to seismic lateral earth pressures – inertial loading and kinematic loading. Inertial loading occurs when the walls are part of the lateral force resisting system for the above-ground portion of the structure and as a result, internal forces and deformations of the walls are influenced by the structural base shear and moments. The effects of inertial loading, including any attached structures, can be evaluated using the direct or substructure methods of soil-structure-interaction (SSI) analysis as discussed in NIST (2012).

Kinematic loading (often referred to as seismic lateral earth pressures or seismic soil increment) is present to some degree on all basement and retaining walls and involves the effects of wall interaction with surrounding soil movements during shaking. There are two general methods that can be used to evaluate the effects of kinematic loading:

- 1) Methods based on a pseudo-acceleration acting on the backfill (limit-state methods) that increase the lateral force on the wall. Such methods are often referred to using the names of authors of papers from the 1920s (Mononobe and Okabe). These methods do not accurately represent the mechanisms by which kinematic pressures develop, but they can be calibrated based on experiments to provide reasonable results. Calibration data has been developed in a series of testing programs (Al Atik and Sitar, 2010; Hushmand et al., 2016; Wagner and Sitar, 2016; Candia et al., 2016) and recommendations derived from those tests involve estimating the pseudo-acceleration using a depth-weighting approach presented in Wagner and Sitar (2016). Once the pseudo-acceleration is developed, the resulting wall pressures can be developed using design charts as given in Koseki et al. (1998) and reproduced as Figure 8.56 in Kramer and Stewart (2025) (Section 8.6.3.1). These methods may not be reliable outside of the calibration range of the aforementioned tests. It should be noted that these methods are not appropriate for evaluation of kinematic loading when large permanent soil displacements are anticipated such as liquefaction induced lateral spreading or slope instability.
- 2) Methods based on relative wall-soil displacement that consider characteristics of the input motion and wall (SSI methods). This method is presented in a Part 3 paper (RP4) (BSSC 2020), Durante et al. (2022), and Section 8.6.3.2 of Kramer and Stewart (2025). These methods are not empirical, although they have been validated against data, and hence are not subject to the same empirical constraints as limit state methods.

Either or both of the kinematic approaches can be applied for development of resultant forces against walls. Both approaches are simplified and have limitations. As listed in RP4, one limitation is related to soil layering within the backfill. For the case of soil layering, absent a very strong impedance contrast (sudden shear wave velocity step within the backfill greater than 2), the shear wave velocity averaging procedure implicit to the method accounts for these effects. For cases with stronger velocity contrasts, more advanced solutions involving direct SSI analyses may be required (NIST, 2012). The other main limitation listed in RP4 is related to non-planar walls, including walls with tiebacks and soil nails. Neither the simplified methods have been calibrated (pseudo-acceleration method) or validated (SSI method) for such conditions. The procedures could be applied if justification for their use in a given situation is provided in the project documentation or more advanced procedures (finite element analysis, finite difference analysis, or other appropriate analytical methods) could be applied.

The peak resultant force from kinematic response is unlikely to occur simultaneously with the peak resultant from inertial response. Where both forces need to be considered, they can be combined as square-root-sum-of-squares to estimate the total seismic force. That seismic force should be combined with the resultant of preexisting static lateral earth pressure without application of load combination factors. The static lateral earth pressure that should be considered depends on the restraint condition of the element. For restrained conditions at rest pressures are applicable, for cantilever or non-restrained elements active pressures are applicable.

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COMMENTARY TO CHAPTER 12, SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

Revise Section C12.2.1 by adding the following paragraphs:

The seismic performance factors for the Post-tensioned Timber Rocking Wall (PTRW) system, when paired with the associated design and detailing requirements, achieve adequate adjusted collapse margin ratios using the FEMA P-695 methodology based on recent research (Zimmerman et al. 2026). The PTRW system achieves high ductility and a high response modification coefficient ($R = 8$) through controlled, predictable rocking at a designated rocking interface. Furthermore, the design procedure for the PTRW system utilizes a section-based analysis method, incorporating an estimation of the nonlinear demands, which results in a close alignment between the calculated design capacity and the peak capacity from a pushover curve. Thus, compared to other $R = 8$ systems, the PTRW system exhibits a fairly low overstrength factor ($\Omega_0 = 2$).

Two PTRW systems are listed in Table 12.2-1, one for a bearing wall and the other for a building frame condition. Unlike many other lateral force-resisting systems in the table, the PTRW system uses the same seismic performance factors for both conditions because the distinction between a bearing wall and a building frame condition is less critical for the PTRW system. Notably, the vertical force in the walls is the sum of gravity load and post-tensioning force. In fact, two designs, one as a bearing wall and one as a building frame, end up having nearly identical vertical forces in the walls because, for the former, the post-tensioning force is reduced to account for the added gravity load.

Revise Section C12.2.1.1 as follows:

C12.2.1.1 Alternative Structural Systems

Historically, this standard has permitted the use of alternative seismic force-resisting systems subject to satisfactory demonstration that the proposed systems' lateral force resistance and energy dissipation capacity is equivalent to structural systems listed in Table 12.2-1, for equivalent values of the response modification coefficient, R , overstrength factor, Ω_0 , and deflection amplification coefficient, C_d . These design factors were established based on limited analytical and laboratory data and the engineering judgment of the developers of the standard.

Under funding from the Federal Emergency Management Agency (FEMA), the Applied Technology Council developed a rational methodology for validation of design criteria for seismic force-resisting systems under its ATC-63 project. Published as FEMA P-695 (2009b), this methodology recognizes that the fundamental goal of the seismic design rules contained in the standard is to limit collapse probability to acceptable levels. The FEMA P-695 methodology uses nonlinear response history (NLRH) analysis to predict an adjusted collapse margin ratio (ACMR) for a suite of archetypical

structures designed in accordance with a proposed set of system-specific design criteria and subjected to a standard series of ground motion accelerograms. The suite of archetypical structures is intended to represent the typical types and sizes of structures that are likely to incorporate the system. The ACMR relates to the conditional probability of collapse given MCE_R shaking and considers uncertainties associated with the record-to-record variability of ground motions, the quality of the design procedure, the comprehensiveness and quality of the laboratory data on which the analytical modeling is based, and uncertainties associated with the analytical modeling. Subsequent studies have been used to benchmark this methodology against selected systems in Table 12.2-1 and have demonstrated that the methodology provides rational results consistent with past engineering judgment for many systems. The FEMA P-695 methodology is therefore deemed to constitute the preferred procedure for demonstrating adequate collapse resistance for new structural systems not currently in Table 12.2-1.

Under the FEMA P-695 methodology, the archetypes used to evaluate seismic force-resisting systems are designed using the criteria for Risk Category II structures and are evaluated to demonstrate that the conditional probability of collapse of such structures conforms to the 10% probability of collapse goal stated in this section and also described in Section C1.3.1 of the commentary to this standard. It is assumed that application of the seismic Importance Factors and more restrictive drift limits associated with the design requirements for structures assigned to Risk Categories III and IV will provide such structures with the improved resistance to collapse described in Section C1.3.1 for those Risk Categories.

The requirements of Section 12.2.1.1 distinguish between generic and project-specific applications of the alternative structural system in terms of the MCE_R ground motions required for collapse evaluation and compliance with the 10% target reliability of Risk Category II structures. For a project-specific application (e.g., individual building), MCE_R ground motions are those at the site of the project-specific application. For generic applications, such as a new seismic force resisting system proposed for Table 12.2-1, MCE_R ground motions are defined as 1.5 times the design spectral accelerations of Tables 11.6-1 and 11.6-2 corresponding to the permitted Seismic Design Category of the alternative structural system. The MCE_R ground motions required for collapse evaluation of an alternative structural system are different from and replace those of the FEMA P-695 methodology (FEMA, 2009).

The FEMA P-695 methodology defines a generic set of MCE_R ground motions, which are capped at $S_{MS} = 1.5g$ ($S_{DS} = 1.0g$) and $S_{M1} = 0.9g$ ($S_{D1} = 0.6g$) for collapse evaluation of a new structural system proposed for use as Seismic Design Category D (i.e., referred to as SDC D_{max} ground motions in FEMA P-695). SDC D_{max} ground motions are used by the FEMA P-695 methodology for design of archetypical configurations of buildings with the structural system of interest, as well as, for collapse evaluation of nonlinear models of building archetypes. When FEMA P-695 was developed in 2009, SDC D_{max} ground motions represented the strongest intensity of shaking at the majority of sites of high seismicity, intentionally excluding only those sites located in “deterministic MCE ground motion regions near active faults.” The MCE_R ground motions of Section 11.4 have increased substantially since 2009 (ASCE/SEI 7-05), in particular, values of S_{M1} of ASCE/SEI 7-22 are about twice those of ASCE/SEI 7-05. For reference, the MCE_R ground motions of ASCE/SEI 7-22 are greater than the SDC

$D_{\max}$ ground motions of FEMA P-695 at approximately two-thirds of all sites in the western United States assuming default site conditions, as reported in FEMA P-2343 (FEMA, 2024).

Under funding from the Federal Emergency Management Agency (FEMA), the Applied Technology Council investigated “Improving Performance of Buildings in Very High-Seismic Regions” (FEMA P-2343) using the FEMA P-695 methodology. The FEMA P-2343 study revealed a potential shortcoming with the collapse performance of structures in very high seismic (VHS) regions, where VHS ground motions of that study are defined as those greater than the SDC $D_{\max}$ ground motions of FEMA P-695. The FEMA P-2343 report made a number of recommendations, including updating SDC $D_{\max}$ ground motions to better characterize earthquake shaking defined by the boundary between Seismic Design Category D and Seismic Design Category E, below which explicit collapse evaluation of a new structural system (proposed for use as Seismic Design Category D) is required, and above which Seismic Design Category E collapse performance is permitted implicitly to exceed the 10% target reliability of Table 1.3-2 of ASCE/SEI 7-22 for Risk Category II structures. Accordingly, the MCE_R ground motions required by Section 12.2.1.1.1 are defined as 1.5 times the strongest intensity of shaking of Table 11.6-1 and Table 11.6-2 for Seismic Design Category D, or, by exception, Seismic Design Category C for a structural system designated as Not Permitted in Seismic Design Category D, E or F.

The MCE_R ground motions of the Seismic Design Category D/E boundary, defined as 1.5 times the design earthquake accelerations of Table 11.6-1, $S_{MS} = 2.25g$ (i.e., $1.5 \times S_{DS} = 1.5 \times 1.5g$) and Table 11.6-2, $S_{M1} = 1.5g$ (i.e., $1.5 \times S_{D1} = 1.5 \times 1.0g$), replace the SDC $D_{\max}$ ground motions of the FEMA P-695 methodology, where the balance of methodology remains the same. The MCE_R ground motions for Seismic Design Category D/E boundary are considered sufficiently strong for collapse evaluation of a new structural system to limit Seismic Design Category E exposure. Whereas, approximately two-thirds of all sites in the western United States have MCE_R ground motions of ASCE/SEI 7-22 that are greater than the SDC $D_{\max}$ ground motions of FEMA P-695, less than about 25 percent have MCE_R ground motions greater than those of the Seismic Design Category D/E boundary assuming default site conditions.

In addition to providing a basis for establishment of design criteria for structural systems that can be used for design of a wide range of structures, the FEMA P-695 methodology also contains a building-specific methodology intended for application to individual structures. The rigor associated with application of the FEMA P-695 methodology may not be appropriate to the design of individual structures that conform with limited and clearly defined exceptions to the criteria contained in the standard for a defined structural system. Nothing contained in this section is intended to require the use of FEMA P-695 or similar methodologies for such cases.

Add Section C12.2.1.3 as follows:

C12.2.1.3 Uncategorized Structural Systems

Some types of building structures have functional requirements that do not allow compliance with one of the permitted system types in Table 12.2-1. In such circumstances, in the absence of code

provisions, designers may take what they believe are reasonable measures, such as designing for $R=1$, to assure the safety of the building. This approach may seem reasonable since it means that the building design base shear is calculated from the elastic design earthquake response spectrum. However, the fundamental safety criterion, from Section 1.3 (Table 1.3-2), is that the conditional probability of collapse is no greater than 10% given occurrence of the MCE_R ground motion, not the design earthquake. Typical building design uses ductile detailing and capacity-based proportioning to produce a structure that can reliably withstand demands exceeding the design strength of components. If such ductility and proportioning cannot be provided (or demonstrated), reliability can be achieved by means of overstrength, which provides elastic structural response. Thus, it is expected that buildings designed as Uncategorized Structural Systems must be proportioned with R less than unity.

The primary seismic design pathway provided in ASCE/SEI 7 requires system proportioning for a design base shear that is calculated from an elastic response spectrum divided by R , the response modification coefficient (often called the “ R factor”). Thus, a system with an R factor greater than unity is expected to undergo controlled inelastic response during a design earthquake, with the level of inelastic response dependent upon the type of system chosen (e.g., ordinary, intermediate, special). System R factors are defined based on expected ductility capacity and system overstrength, where a large R factor indicates a large ductility capacity. To ensure stable inelastic response, seismic design requirements include ductile detailing for yielding (deformation-controlled) elements and capacity-based design for nonyielding (force-controlled) elements. These requirements are stringent for special (high R factor) systems and somewhat less so for intermediate and ordinary systems.

The permitted systems of ASCE/SEI 7 (Table 12.2-1) provide a range of options for design where there is an inverse relationship between strength and ductility requirements. Systems with large ductility capacity are designed for smaller forces (larger R factor) and systems with limited ductility capacity are designed for larger forces (smaller R factor). Currently missing from ASCE/SEI 7 is the logical limiting case of systems with no ductility that are proportioned to remain elastic. This new section for uncategorized structural systems addresses this important design pathway.

There is a subset of structures where it is difficult or impossible to satisfy ductile detailing and capacity-based design requirements, and an alternate design pathway – involving only strength and elastic stiffness requirements – is essential. Such structures include freestanding elevator towers, canopies, carports, open shelters, artistic-purpose structures, domes, structures with geometrical configurations that cannot meet prescriptive ductility requirements, and other structures with systems not defined in ASCE/SEI 7. More generally, Table 12.2-1 provides a finite set of structural systems that do not encompass the full range of building types and functional requirements needed in the built environment. In addition, there are structures that possess significant intrinsic excess strength, and it would be beneficial to have a design pathway that would leverage the excess strength to obviate ductile detailing and capacity-based design requirements. Examples of such structures include hard wall buildings (i.e., tilt-up concrete or masonry) with few windows/doors, and lightweight structures located in high wind regions. For these scenarios, adequate reliability must be demonstrated through separation between the system capacity and the seismic demand. The

approach to demonstrate this reliability employs a component-based reliability framework, where the failure of the system is associated with the first component failure. This method makes no assumption about the system characteristics since response is elastic up to the first (brittle) limit state. No ductility is assumed, and any ductility that may be present provides no design benefit.

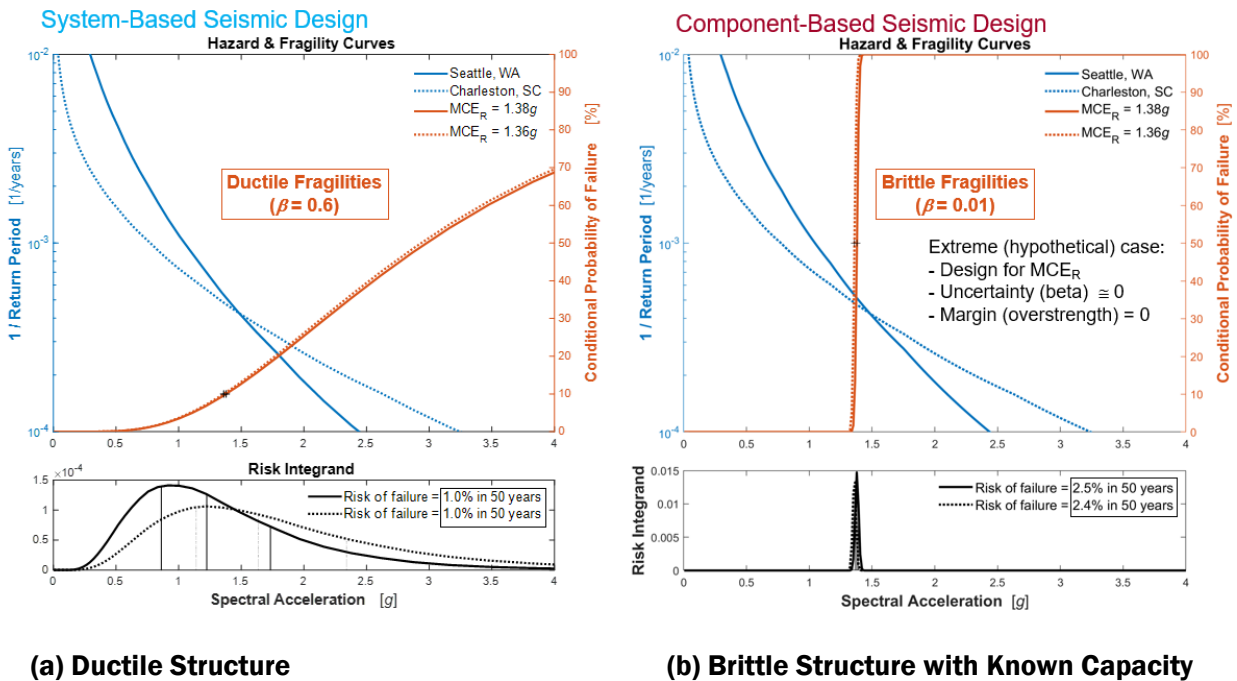


Figure C12.2.1.3-1 Collapse Risk for Hypothetical Ductile and Brittle Structures

The new seismic design pathway for Uncategorized Structural Systems is developed using a risk-targeted methodology consistent with the current methodology used for ductile structures. In the current system-based seismic design framework for ductile structures, hazard curves are defined from USGS (or site-specific PSHA, per ASCE/SEI 7 Chapter 21) by location, spectral period, and Site Class. A generic fragility curve based on FEMA P-695 and dependent on MCE_R spectral acceleration is defined with lognormal shape, standard deviation (total uncertainty) $\beta = 0.6$, and $P[\text{Collapse} | MCE_R] = 10\%$. [Note that this β is not the same as any of the betas (e.g., Table 1.3-1, Equation 12.8-19, Chapter 17) that occur in the body of ASCE/SEI 7.] The MCE_R -dependent fragility curve is iteratively integrated with each hazard curve to achieve a target collapse risk of 1% in 50 years. This framework is illustrated in Figure C12.2.1.3-1(a) for scenarios in Seattle, WA and Charleston SC where the MCE_R spectral accelerations are similar.

The concept of the new risk-targeted methodology for design of uncategorized elastic brittle structures is illustrated in Figure C12.2.1.3-1(b) for an extreme hypothetical case, where the fragility curve is very close to a step function defining a system that has known strength capacity exactly defined by the elastic MCE_R spectral acceleration. The system has no uncertainty and no overstrength. In this case, the collapse risk is approximately 2.5% in 50 years for both sites (in Seattle, WA and Charleston, SC). Although this is a hypothetical case, it is instructive since it represents a scenario where a seismic force-resisting system has strength exactly corresponding to

MCE_R elastic forces ($R = 2/3$), and collapse risk is approximately two and a half times the acceptable threshold. To adjust this hypothetical case to a realistic scenario, both uncertainty (β) and margin (overstrength) must be considered. Increasing uncertainty will have a negative effect on the collapse assessment, whereas increasing margin will have a positive effect, so the net influence of these counteracting parameters must be determined.

Since the new component-based elastic seismic design methodology requires only elastic response, uncertainty in response is significantly reduced compared to inelastic structures. Thus, β (total uncertainty) less than 0.6 is appropriate. Uncertainty in elastic seismic response still arises due to higher mode effects and ground motion record-to-record variability, along with material strength variability. FEMA P-695 states that record-to-record variability (β_{RTR}) is taken as 0.4 for structures with significant period elongation, but that systems with little, or no, period elongation have significantly less record-to-record variability. For these structures, the minimum permissible β_{RTR} is 0.2. Thus, for the current development where structural response must be elastic, $\beta = 0.2$ is taken as a minimum total uncertainty. Calculations with a range of β show that the final outcome (required R factor for achieving the target risk of 1% in 50 years) is relatively insensitive for the range of $\beta = 0.15$ to 0.45, and $\beta = 0.3$ is taken as the baseline for risk calculations used to develop the provisions of Section 12.2.1.3 for elastic Uncategorized Structural Systems.

Beyond uncertainty, the other parameter associated with the structural system that affects collapse risk is overstrength, also called margin. In the case of component-based elastic design for a system assumed to have brittle response, this margin is considered from the perspective of an intrinsic overstrength that will be present in all systems, regardless of the material type or system configuration. Lower-bound intrinsic overstrength (baseline margin) is approximated as (material overstrength) $\times$ (element oversize) / (material strength reduction factor) = $1.1 \times 1.1 / 0.9 \cong 1.3$.

Considering baseline $\beta = 0.3$ and margin = 1.3, collapse risk calculations are conducted as outlined above, following ASCE/SEI 7 Section 21.2.1 and Luco et al. (2007) but with $\beta = 0.3$ and $P[\text{Collapse} | MCE_R] = 19\%$, which corresponds to margin = 1.3. These calculations determine the required increase in MCE_R spectral acceleration for achieving the target risk of 1% in 50 years, and thereby the required R factor. In this context, the R factor is being determined for use in ASCE/SEI 7, thus it is operating on the design spectrum, which is $2/3 \times MCE_R$. Collapse risk calculations are conducted for 34 example locations (Table C22-2) that are commonly used for national benchmark studies, considering four periods (0.2, 1, 2 and 5 sec) and two site classes (BC and D). The calculations can be reproduced using the Python source code for the USGS Risk-Targeted Ground Motion Calculator (<https://doi.org/10.5066/P9A2F2B8>) or other means, including a spreadsheet. The statistics for these calculations are shown in Table C12.2.1.3-1, and they indicate that the R factor ranges from 0.46 to 0.64 depending on location and period, and the mean is around $R = 0.5$. Considering the tight banding of this data, location-, period-, and/or site class-specific R factors are not justified, and a single value of $R = 0.5$ is proposed for design of all elastic structural systems. The change to the target risk of collapse in the 2026 NEHRP Provisions from 1% to 1.5% in 50 years does not change the proposed $R = 0.5$, because the increase in MCE_R spectral accelerations required to achieve the 1.5% with $\beta = 0.3$ and margin = 1.3 is nearly the same as the increase

needed for 1% in 50 years. This is confirmed with calculations for the 34 example locations and an additional 132 locations across the conterminous United States.

Table C12.2.1.3-1 R factor required to achieve a Risk = 1% in 50 Years (Statistics from 34 example locations presented in Table C22-2)

		Site Class	
T (sec)		BC	D
0.2	Min	0.48	0.50
	Max	0.60	0.64
	Mean	0.54	0.58
1	Min	0.47	0.48
	Max	0.59	0.59
	Mean	0.53	0.54
2	Min	0.46	0.47
	Max	0.62	0.58
	Mean	0.53	0.53
5	Min	0.47	0.47
	Max	0.64	0.58
	Mean	0.53	0.53

Range
 $R = 0.46-0.64$
Mean
 $R = 0.54$

Margin = 1.3
beta = 0.3

12/12/2023, data from N. Luco, USGS

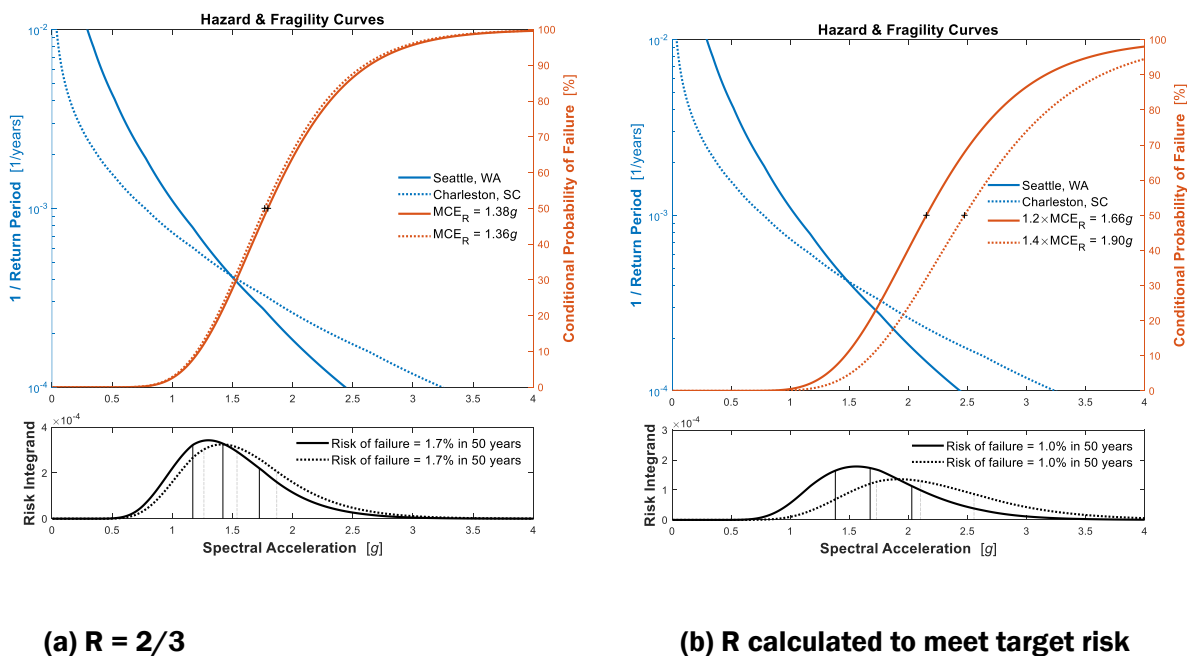


Figure C12.2.1.3-2 R Factors for Uncategorized Structural Systems in Seattle, WA and Charleston, SC with $\beta = 0.3$ and margin = 1.3.

Figure C12.2.1.3-2 illustrates the collapse risk calculations for 2 of the 272 cases that are summarized in Table C12.2.1.3-1. Compared to Figure C12.2.1.3-1(b), the influence of uncertainty

($\beta = 0.3$) is seen in the less steep fragility curves, and these fragility curves are shifted to the right through the intrinsic overstrength (baseline margin = 1.3). Figure C12.2.1.3-2(a) represents buildings proportioned for the MCE_R ($R = 2/3$), and the resulting collapse risk for both Charleston and Seattle is 1.7% in 50 years. Figure C12.2.1.3-2(b) represents buildings proportioned for the calculated R factors required to achieve the target Risk = 1% in 50 years. The resulting R factors are $(2/3) \cdot (1/1.4) = 0.48$ for Charleston and $(2/3) \cdot (1/1.2) = 0.56$ for Seattle.

Although $R = 1$ may commonly be viewed as design for elastic response at the Design Earthquake since it means that the building design base shear is calculated from the elastic design earthquake response spectrum, due to uncertainty in earthquake characteristics and structural response, this approach does not ensure elastic response under the design earthquake. To meet the risk target that is associated with providing adequate safety per ASCE/SEI 7, buildings designed as elastic Uncategorized Structural Systems must be proportioned with R less than unity.

The elastic Uncategorized Structural Systems approach provides a ceiling on design forces. The derivation of the required strength assumes only an overstrength of 1.3 (expected strength compared to design strength) and a variability of response of 0.3; no other potentially beneficial effects are included in the derivation. Such effects may include higher overstrength, displacement capacity beyond the elastic range, and beneficial effects of rocking and sliding.

The provisions allow the use of a $R = 2.0$ for conditions at the soil-foundation interface recognizing that controlled inelastic response at the soil-foundation interface – through lateral sliding, and/or uplift/rocking – does not compromise life safety and is associated with acceptable modes of behavior that possess deformation capacity. Figure C12.2.1.3-3 (from Gajan et al., 2008), shows idealized hysteretic behavior for uplift response (a), lateral passive response (b), and lateral sliding response (c), all of which exhibit significantly more ductility than is assumed for the reliability of $R=0.5$ structures.

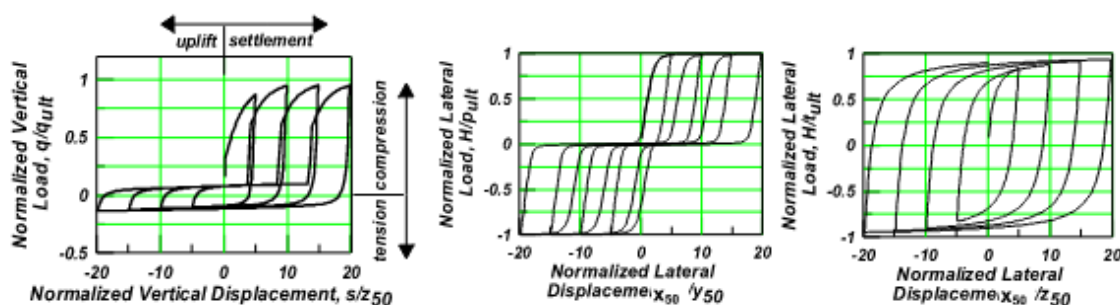


Figure C12.2.1.3-3 Hysteretic behavior for uplift response (a), lateral passive response (b), and lateral sliding response (c) (from Gajan et al., 2008)

Numerical simulations of nonlinear single degree-of-freedom (SDOF) models (e.g., Newmark and Hall 1982, Seo 2005) – with a wide range of hysteretic constitutive relationships – provide context on expected ductility demand as a function of design strength reduction (represented by the strength reduction factor, R). These SDOF response history results can be viewed as representing elastic

superstructures with nonlinear soil-foundation behavior approximated by the hysteretic models. Following Newmark and Hall (1982), the ductility component of the design force reduction (R factor) has been taken as equal to the element ductility. For the case of undefined systems, the other potential components of strength reduction (overstrength and redundancy) are neglected. Since sliding and uplift/rocking mechanisms at the soil-foundation interface are expected to have sufficient deformation capacity without appreciable degradation of load-carrying capacity such that a ductility demand of 2 is sustainable (as shown in Figure C12.2.1.3-3), $R = 2.0$ is selected as a reasonable and conservative strength reduction factor associated with proportioning the capacity of the soil-foundation interface. In addition, foundations of reinforced concrete, including footings, piles, pile caps, and mats, and well as piles of steel and concrete-filled steel designed to resist forces based upon ductile response in the superstructure have consistently performed well, even for superstructures that are significantly stronger than the minimum requirements. The structural response of such foundation elements is not brittle, thus the forces determined using $R = 0.5$ for uncategorized system superstructures, which could be of any material, are larger than needed for foundations, and $R = 2.0$ is acceptable. The requirements for strength of foundation elements of reinforced concrete and structural steel contained in this provision are established to prevent brittle foundation elements.

Because the required resistance to sliding and rocking is significantly less than the design base shear used for the structure, it is possible that these modes could be the primary source of nonlinearity in the structural response. Determining the displacements associated with these modes is necessary where only parts of the structure slide or rock; it may also be necessary for estimating P-Delta effects. Such determination may require significant investigation of soil-structure interaction (NIST, 2012). Where sliding or rocking only result in a rigid-body movement of the entire structure (such as for a foundation system consisting of a single mat) or rigid-body rotation of the entire structure (such as for a single tower on a mat), the drift limit should be applied to the deformation of the structure not including these rigid-body modes.

Future research may evaluate in more detail the beneficial effects of nonlinear response and deformation capacity at the soil-foundation interface and justify an increase in the associated R value. In addition, future research could evaluate an exception clause for Uncategorized Structural Systems on the basis that the required strength of elements need not exceed the maximum forces associated with sliding, rocking or uplift capacity of the foundation. Employing the capacity at the soil-foundation interface to reduce design force demands in the superstructure would require that: 1) a rational analysis shows that such behavior reliably controls the system response considering upper-bound conditions; and 2) the resulting displacement and deformation demands on the gravity-load-resisting system and nonstructural elements are determined using lower-bound conditions. In the analysis: 1) absolute and relative displacements corresponding to the force-limiting actions must be included in determination of drifts and displacements; 2) forces corresponding to the soil-foundation limit states must consider uncertainties in foundation sizes, soil properties, and loading, as well as participation of attached structural and non-structural elements; and 3) rational methods must be used to establish the minimum required strength for these limit states.

Since all potentially-beneficial system characteristics (other than intrinsic overstrength) are excluded in design of the superstructure with $R = 0.5$, there are no proportioning, detailing or analysis requirements beyond a simple elastic analysis to permit design for strength. This method is therefore applicable, in theory, to any system and any material for which an elastic analysis can be performed and for which design strengths for applicable limit states can be determined. It is envisioned that this method will be applied to uncategorized systems not defined in the standard, and to defined (categorized) systems used outside their defined range. However, ASCE/SEI 7 Chapter 12 requires conformance to material requirements in Chapter 14, which includes only steel, concrete, composite steel and concrete, masonry, and wood. To go beyond these materials, ASCE/SEI 7 11.1.4 governs, and the AHJ must approve based on sufficient substantiating evidence.

Historically, there are many examples of structures with limited ductility or no ductility that have performed poorly and collapsed during earthquakes. These brittle structures were loaded such that the force demands exceeded the elastic structural capacity, and undesirable inelastic response ensued. Although the procedure for design of uncategorized structural systems imposes no requirements that provide ductile response, brittle response is prevented through the elastic design framework that relies on system strength only.

It is recognized that very few, if any, buildings are at risk of collapse at their elastic limit. This upper-bound method is provided for cases where design for very high forces is preferable to the use of a defined system or investigating the influence of beneficial effects neglected in this method. Where a more economical solution is required and a defined system cannot be used, the engineer may consider ASCE 41, FEMA P695, or other documents addressing the performance of systems not defined in the standard.

The development of this elastic design methodology assumes that the demands that develop during dynamic structural response are distributed consistently with the force profile used to proportion the system. Thus, a structure proportioned using ELF must have dominant first-mode response. If a structure proportioned with ELF has dynamic response that includes significant higher mode contributions, the force demands may exceed the strength provided, particularly near the top of the structure. To prevent this type of response where demand could exceed capacity, a general height limit of 65 ft is imposed, for all SDC, so that response remains predominantly in the first mode. Since MRSA considers multiple structural modes in system proportioning and the strength distribution is adjusted accordingly, the potential for demand to exceed capacity is mitigated. As a result, when MRSA is used in proportioning a structure, the height limit is permitted to be relaxed, to 160 ft for all SDC.

For uncategorized structural systems designed according to this section, with system overstrength factor defined as 1.0, it is necessary to specify that the redundancy factor should be 1.0, so there is no confusion about using a redundancy factor of 1.3. The design of diaphragms cannot be conducted using the alternative provisions of 12.10.2 with an R_s factor more than 1, since this provision relies on elastic response throughout the full system. Elastic structural response is expected to cause larger accelerations than for an inelastic structure, so R_μ is taken as 1.0 in lieu of

Equation (13.3-6). In addition, structural and nonstructural wall and nonstructural component design must consider a force consistent with forces for diaphragms, chords, and collectors in Section 12.10.

The use of $R=0.5$ (in combination with the two-thirds factor in Section 11.4.4) can be conceptualized as the elastic design of a potentially non-ductile system for ground motions 1.33 times MCE_R . This value was determined in order to achieve collapse risk consistent with design of more ductile systems for two-thirds MCE_R ground motions (i.e., Design Earthquake ground motions). As such, the displacement determined using such forces correspond to the MCE_R level; application of the $C_d=0.5$ brings these displacements down to the Design Earthquake level, consistent with other systems.

Revise Section C12.2.3.2 as follows:

C12.2.3.2 Two-Stage Analysis Procedure

A two-stage equivalent lateral force (ELF) procedure is permitted where the lower portion of the structure has a minimum of 10 times the stiffness of the upper portion of the structure. In addition, the period of the entire structure is not permitted to be greater than 1.1 times the period of the upper portion considered as a separate structure supported at the transition from the upper to the lower portion. An example would be a concrete podium under a wood- or steel-framed upper portion of a structure. The upper portion may be analyzed for seismic forces and drifts using the values of R , Ω_0 , C_d , and ρ for the upper portion as a separate structure. The seismic forces resulting from horizontal seismic loading, F_x (e.g., shear and overturning), at the base of the upper portion are applied to the top of the lower portion and scaled up by the ratio of $(R/\rho)_{upper}$ to $(\frac{R}{\rho})_{lower}$. All other forces from the upper portion, including gravity forces and seismic forces resulting from vertical seismic loading, are directly applied to the lower portion without scaling. The lower portion, which now includes the seismic forces from the upper portion, may then be analyzed using the values of R , Ω_0 , C_d , and ρ for the lower portion of the structure. The lower portion of a structure qualifying for the two-stage analysis procedure is not exempted from meeting the requirements of 12.2.3.1. Item (f) clarifies that the structural height limits of Table 12.2-1 are applied to the upper portion individually, and not the combination of the two portions. This is permitted because the criteria are based on stiffness and period limit dynamic interaction between the lower portion and the upper portion.

Where horizontal structural irregularity Type 4 (out-of-plane offset) or vertical structural irregularity Type 3 (in-plane discontinuity) is present at the transition from the upper portion to the lower portion, the requirement for the overstrength effects is associated with the expected peak capacity of the upper portion. The ratio of R/ρ values (that of the lower portion to that of the upper portion) is associated with the effect of the upper portion mass on the lower portion, which is a separate consideration. These two effects are required to be used in combination where applicable. Overstrength effects include but are not limited to the requirements of Section 12.3.3.4 for elements supporting discontinued walls or frames and the requirements of Sections 12.10.1.1 and 12.10.2.3 for diaphragm transfer forces. See also Commentary Section C12.3.3.4.

Add the following to the end of Section C12.2.5.6.1:

After the development of the FEMA P-695 methodology, typical ordinary moment frames used in metal building applications were investigated to ascertain if the height and weight limits that were assigned post-Northridge pass the rigor of the FEMA P-695 methodology and if the results support any modifications.

A peer-reviewed FEMA P-695 methodology evaluation was more recently applied to a suite of archetypes representing both clear span and modular (interior column) configurations to explore the roof and wall weight limits at different building heights, as presented in Table C12-x.

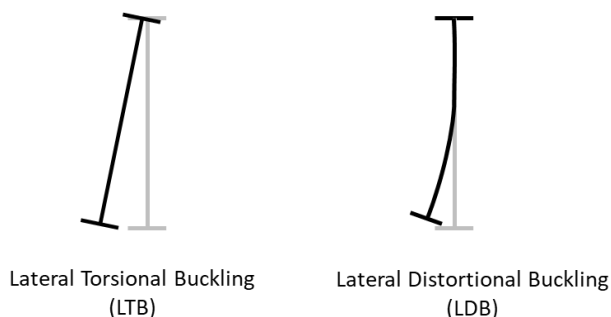
Table C12-x – Summary of seismic evaluation of metal building archetypes per FEMA P-695

Study Phase	Archetype No.	Archetype ID	Frame	Wall Weight	Roof Load	Span	Eave Height	Overstrength	Period-Based Ductility	h/b≤6*	A _w /A _r ≤2.5*	ACMR _i	ACMR _{10%}	ACMR _{20%}	Pass/Fail
						ft	ft	Q ₀	μ _T						
1 (2018)	1	A1	Clear span	Heavy	light	70	25	2.1	3.0	4.7	2.4	1.720	1.890	1.520	Pass
	2	A2	Clear span	Heavy	light	70	45	1.1	4.2	5.7	4.3	2.500	2.150	1.520	Pass
	3	A3	Clear span	light	light	70	25	1.9	1.4	3.1	1.9	1.640	1.670	1.400	Pass
	4	A4	Clear span	light	light	200	35	3.3	1.7	4.2	1.3	1.720	1.640	1.380	Pass
2 (2022)	5	AM1	Modular	light	light	50	25	2.3	1.9	3.5	1.5	1.543	1.699	1.417	Pass
	6	AM2	Modular	light	light	50	25	2.0	1.6	3.7	2.8	1.378	1.660	1.394	Pass
	7	AM3R	Modular	light	light	50	45	1.9	1.5	4.4	1.7	2.067	1.647	1.386	Pass
	8	AM4R	Modular	light	light	50	45	1.7	2.0	4.9	1.8	2.204	1.716	1.427	Pass
	9	AM5	Modular	light	light	100	25	2.3	1.5	4.8	1.8	1.483	1.652	1.389	Pass
	10	AM7	Modular	light	light	100	45	2.0	1.6	4.9	2.4	1.691	1.667	1.398	Pass
3 (2025)	11	M-HW-1	Modular	Heavy	light	50	45	1.7	1.9	3.9	1.3	2.086	1.700	1.418	Pass
	12	M-HW-2	Modular	Heavy	light	100	45	1.8	1.5	4.9	2.5	1.733	1.651	1.389	Pass
	13	M-HW-3	Modular	Heavy	Heavy	50	45	1.7	1.9	4.7	1.9	1.944	1.702	1.419	Pass
	14	M-HW-4	Modular	Heavy	Heavy	100	45	1.3	1.7	4.8	2.4	1.622	1.672	1.401	Pass
	15	C-HR-1	Clear span	light	Heavy	50	25	2.4	1.8	4.9	2.2	1.601	1.684	1.408	Pass
	16	C-HR-2R	Clear span	light	Heavy	100	45	2.1	>1.8	5.2	1.4	1.687	1.637	1.380	Pass
	17	C-HR-3	Clear span	light	Heavy	50	25	2.4	2.3	4.4	2.2	1.461	1.711	1.425	Pass
	18	C-HR-4R	Clear span	light	Heavy	100	45	2.7	>1.8	4.9	1.2	2.107	1.650	1.388	Pass
	Average								2.0	1.9	-	-	1.788	1.711	-

* Rafter proportions provided in second braced span from the knee.

These evaluations have been done in three separate studies. The baseline clear span buildings were studied in 2018 as Phase-1 (Meimand, Moen and Schafer, 2018). Subsequently, in Phase-2 of the study, a suite of modular buildings was evaluated in 2022, and the evaluations of clear span and modular archetypes with heavy wall weight and roof load were completed in 2025 (Phase-3) (Note: Reference forthcoming).

Phase-1 and Phase-2 of the study confirmed the suitability of the current seismic design parameters for the design of metal buildings as ordinary moment frames in accordance with AISC 360 and AISC 341. Phase-3 of the study specifically targeted the increase in wall and roof dead loads and the height limitations. The results showed that additional limits are required to justify an increase in the dead loads and height limitations of the OMFs. For short, braced spans under large gravity demands, lateral distortional buckling (LDB - Figure C12-x) was observed between braces when the member proportions violated the limits of $h/b_r \leq 6$ and $A_w/A_r \leq 2.5$. The LDB behavior resulted in limited ductility for the frame. When the proportioning limits were enforced, the LDB limit state was removed, and the usual checks on local buckling and lateral torsional buckling (Figure C12-x) were sufficient for the frame to develop substantial ductility.

**Figure C12-x**

The cross-section proportioning limits ($h/b_f \leq 6$ and $A_w/A_f \leq 2.5$) are the same as those used in shear to develop stable post-buckling through tension field action (TFA) in AISC 360, Section G2.2.

Stabilizing the compression flange to anchor shear post-buckling forces and stabilizing the compression flange to avoid web distortion and lateral-distortional buckling-related phenomena. To avoid the lateral deformation of the flange in compression, the width and area of the flanges need to satisfy the proposed criteria to ensure the flanges and the web stay in the same plane without lateral instabilities. The limitation on h/b_f ensures there is enough out-of-plane stiffness in the flange to control LDB. The limitation on A_w/A_f controls the force transferred into the flange from the web and ensures the flange has enough strength. It was observed in previous studies that, once the flange is adequately braced, LTB can provide a stable response. Note that all currently available hot-rolled W shapes satisfy the flange and web limits in Section 12.2.5.6.1(c)(2).

As shown in Table C12-x, all metal buildings with heavy wall weight and roof load passing FEMA P-695 evaluations satisfy the proposed limitations. However, it is worth noting that the first 10 archetypes that were designed without enforcing the proposed criteria almost always satisfied them as well. This observation confirms that the proposed criteria ensure sufficient ductility and over-strength, which was needed to fully pass the FEMA P-695 seismic evaluation.

The proposed limits were validated using metal building moment frame archetypes that were comprised of built-up shapes with slender cross-section proportions, a more strenuous test than steel OMFs constructed with locally-compact rolled shapes. Thus, the studies allow the extension of the results to other types of structural members permitted in steel OMFs.

Figure C12-xx compares the provisions of Section 12.2.5.6.1(a) (Figure C12-xx (A)) with the increase in eave height and roof and wall dead loads permitted in Section 12.2.5.6.1(c) (Figure C12-xx (B)).

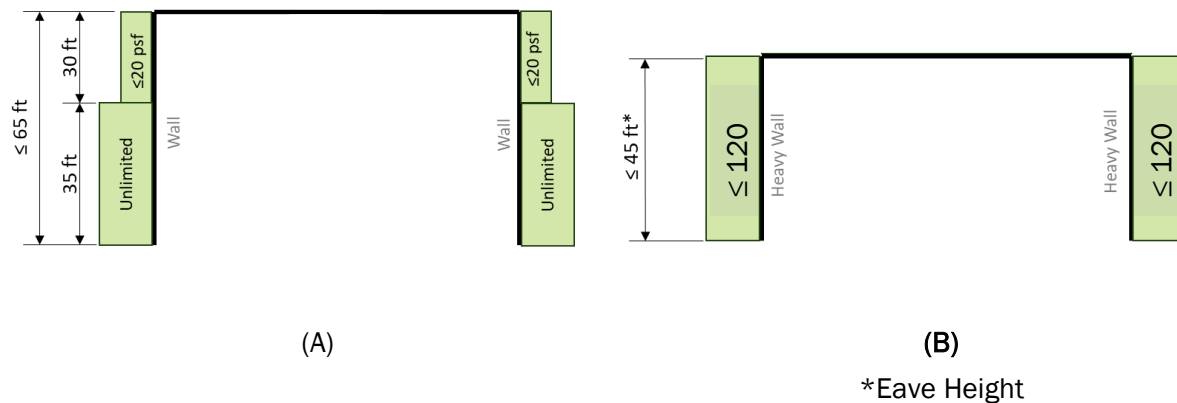


Figure C12-xx: Wall Weight Limits (A) Schematic presentation of 12.2.5.6.1(a); (B) Schematic presentation of 12.2.5.6.1(c)

Revise Section C12.3.1.3 as follows:

C12.3.1.3 Calculated Flexible Diaphragm Condition

A diaphragm is permitted to be idealized as flexible if the calculated diaphragm deflection (typically at midspan) between supports (lines of vertical elements) is greater than two times the average story drift of the vertical lateral force-resisting elements located at the supports of the diaphragm span.

The relative stiffness between the diaphragm and the vertical lateral force-resisting elements is important when characterizing a diaphragm as flexible; thus, the deflections and story drifts are from the same tributary loading without the use of deflection amplification factors. The analysis begins with a flexible diaphragm assumption, where δ_{MDD} is calculated under F_x forces from 12.3.3 using a load factor of 1.0 on E_h (in accordance with Section 12.3.3.6) acting on the diaphragm and Δ_{ADVE} is calculated under the resulting tributary lateral loading acting on the supporting lateral force-resisting elements. Note that the importance factor I_e remains included so that system stiffnesses are properly considered at the design load level.

Figure 12.3-1 depicts a distributed load, conveying the intent that the tributary lateral load be used to compute δ_{MDD} , consistent with the Section 11.3 symbols. A diaphragm opening is illustrated, and the shorter arrows in the portion of the diaphragm with the opening indicate lower load intensity because of lower tributary seismic mass.

Revise Section C12.3.3.5 as follows:

C12.3.3.5 Increase in Forces Caused by Irregularities for Seismic Design Categories D through F

Section 12.3.3.5 is applicable to diaphragm design forces determined in accordance with Section 12.10.1. Per Section 12.10.2, Item 2, diaphragm design forces determined in accordance with Section 12.10.2 are exempt from these provisions. Per Section 12.10.3.2.6, diaphragm design forces determined in accordance with Section 12.10.3 are exempt from these provisions. The listed irregularities may result in loads that are distributed differently than those assumed in the equivalent lateral force procedure of Section 12.8, especially as related to the interconnection of the diaphragm with vertical elements of the seismic force-resisting system. The 25% increase in force is intended to account for this difference. The variation in diaphragm loading is only expected to significantly affect the diaphragm(s) at the level(s) of irregularities as discussed in FEMA P-2012. Consequently, the 25% increase in diaphragm forces is limited to the diaphragm level(s) where irregularities are present. Where the force is calculated using amplified collector forces of Sections 12.10.1.2, no further increase is warranted. For many structures these sections already require amplified seismic forces be used for collectors and their connections to vertical elements, so the force increases of Section 12.3.3.5 Item 2 will not often be used. Similarly, Sections 12.10.1.1 will sometimes trigger amplification of diaphragm transfer forces by Ω_o , again meaning that the force increases of Section 12.3.3.5 Item 2 will not be used. Item 1 of Section 12.3.3.5 may trigger more often, however, because Section 12.10.1.2 does not require amplified forces for diaphragm connections directly to vertical element or to collectors.

Revise Section C12.4.2 as follows:

C12.4.2 Seismic Load Effect

The seismic load effect includes horizontal and vertical components. The horizontal seismic load effects, E_h , are caused by the response of the structure to horizontal seismic ground motions, whereas the vertical seismic load effects are caused by the response of the structure to vertical seismic ground motions. The basic load combinations in Chapter 2 were duplicated and reformulated in Section 12.4 to clarify the intent of the provisions for the vertical seismic load effect term, E_v .

C12.4.2.1 Horizontal Seismic Load Effect

Horizontal seismic load effects, E_h , are determined in accordance with Equation (12.4-3) as $E_h = \rho Q_E$. Q_E is the seismic load effect of horizontal seismic forces from V or F_p . The purpose of E_h is to approximate the horizontal seismic load effect from the design basis earthquake to be used in load combinations including E for the design of lateral-force-resisting elements including diaphragms, vertical elements of seismic force-resisting systems as defined in Table 12.2-1, the

design and anchorage of elements such as structural walls, and the design of nonstructural components.

C12.4.2.2 Vertical Seismic Load Effect

The vertical seismic load effect, E_v , is determined with Equation (12.4-4a) as $E_v = 0.2S_{DS}D$ or with Equation (12.4-4b) as $E_v = 0.3S_{av}D$. Vertical seismic effects may be small in most building-like structures. The use of the vertical spectrum has been found to be most impactful for three specific classes of nonbuilding structures including tanks and vessels, hanging structures, horizontal cantilevers, and log-span truss beams, which are sensitive to vertical ground motions. Equation (12.4-4a) uses horizontal spectral ordinates (independent of any vertical period) to approximate the vertical effects. The concept of using an equivalent static load coefficient applied to the dead load to represent vertical seismic load effects was first introduced in ATC 3-06 (1978), where it was defined as simply $\pm 0.2D$. The load combinations where the vertical seismic load coefficient was to be applied assumed strength design load combinations. Neither ATC 3-06 nor the early versions of the NEHRP provisions (FEMA 2009a) clearly explained how the values of 0.2 were determined, but it is reasonable to assume that it was based on the judgment of the writers of those documents. It is accepted by the writers of this standard that vertical ground motions do occur and that the value of $\pm 0.2S_{DS}$ was determined based on consensus judgment. Many issues enter into the development of the vertical coefficient, including phasing of vertical ground motion and appropriate R factors, which make determination of a more precise value difficult. Although no specific rationale or logic is provided in editions of the NEHRP provisions (FEMA 2009a) on how the value of $0.2S_{DS}$ was determined in Equation (12.4-4a), one possible way to rationalize the selection of the $0.2S_{DS}$ value is to recognize that it is equivalent to $(2/3)(0.3)S_{DS}$, where the 2/3 factor represents the often-assumed ratio between the vertical and horizontal components of motion, and the 0.3 factor represents the 30% in the 100%-to-30% orthogonal load combination rule used for horizontal motions. The 2/3 ratio between the vertical and horizontal components has been shown to be much more variable (higher and lower) in several studies (e.g., Bozorgnia and Niazi, 1993; Bozorgnia and Campbell, 2004; Gülerce and Abrahamson, 2011; Bozorgnia and Campbell, 2016; among others) and, although it is a good average approximation in the absence of information on vertical period, it is demonstrated not to be valid in many instances.

For situations where the vertical component of ground motion is explicitly included in design analysis, the vertical ground motion spectra definition provided in Section 11.9 should be used. Following the rationale described above, the alternate vertical ground motion component determined in Section 11.9, S_{av} , is combined with the horizontal component of ground motion by using the 100%-to-30% orthogonal load combination rule used for horizontal motions, resulting in the vertical seismic load effect determined with Equation (12.4-4b), $E_v = 0.3S_{av}D$.

Equation (12.4-4a), $E_v = 0.2 S_{DS} D$, is an approximate generalization for vertical effects and depending on vertical period, can result in very different motions from Equation (12.4-4b), $E_v = 0.3 S_{av} D$. The use of the more accurate period-dependent equation is not required in recognition that vertical periods are not readily available and unlike horizontal periods cannot be easily approximated at this time for all structures. However, when vertical periods are computed, the more accurate equation may be considered.

Depending on the vertical period, the two alternatives, $E_v = 0.3 S_{av} D$ and $E_v = 0.2 S_{DS} D$, can result in very different outcomes. For a vertical period range, T_v , from about 0.05 to 0.2 seconds, $E_v = 0.2 S_{DS} D$ can be much lower than $E_v = 0.3 S_{av} D$ based on analysis of many sites in the western U.S., as shown in the example presented in Figure C12.4-1. On the other hand, for sites in the central and eastern U.S. (east of about -105 degrees longitude), where Section 11.9.1 specifies the vertical component simply as 2/3 of the horizontal component, use of $E_v = 0.2 S_{DS} D$ can result in comparable or higher values with respect to $E_v = 0.3 S_{av} D$ (Peng et al., 2025).

E_v is permitted to be taken as zero in Equations (12.4-1), (12.4-2), (12.4-5), and (12.4-6) for structures assigned to Seismic Design Category B and in Equation (12.4-2) for determining demands on the soil–structure interface of foundations. E_v increases the load on beams and columns supporting seismic elements and increases the axial load in the axial-moment (P–M) interaction of walls resisting seismic load effects.

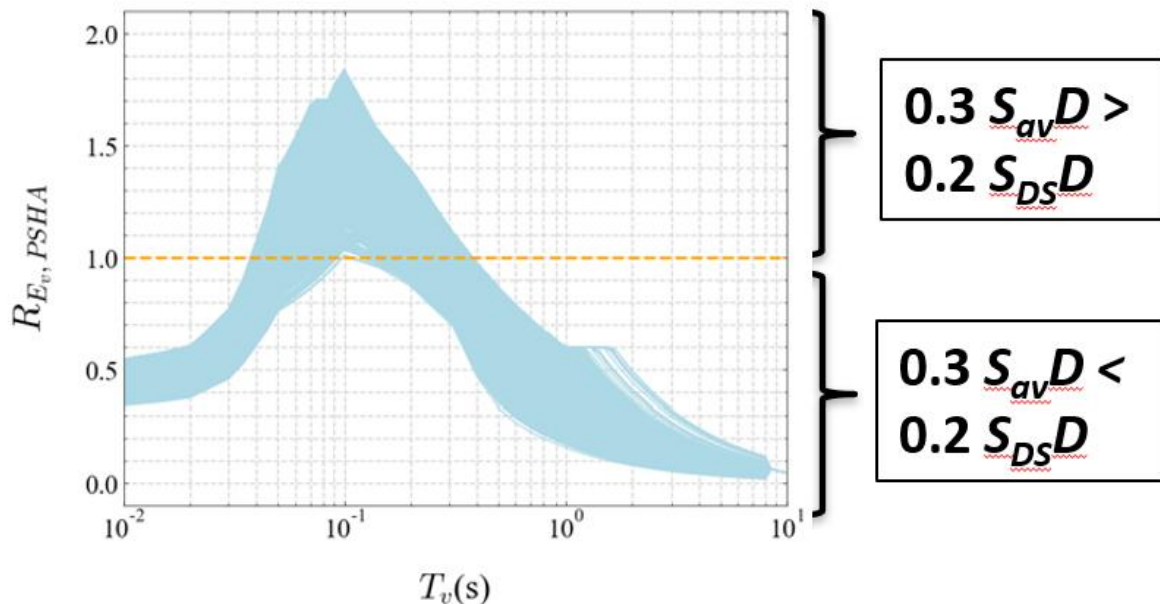


Figure C12.4-1. Ratio of $(0.3 S_{av} D)/(0.2 S_{DS} D)$ for over 190,000 sites in Seismic Design Category D in California (Bolourani et al., 2025).

Revise Section C12.7.1 as follows:

C12.7.1 Foundation Modeling

Structural systems consist of three interacting subsystems: the structural framing (girders, columns, walls, and diaphragms), the foundation (footings, piles, and caissons), and the supporting soil. The ground motion that a structure experiences, as well as the response to that ground motion, depend on the complex interaction among these subsystems.

Those aspects of ground motion that are affected by site characteristics are assumed to be independent of the structure–foundation system because these effects would occur in the free field in the absence of the structure. Hence, site effects are considered separately (Sections 11.4.3 through 11.4.5, Chapters 20 and 21).

Given a site-specific ground motion or response spectrum, the dynamic response of the structure depends on the foundation system and on the characteristics of the soil that supports the system. The dependence of the response on the structure–foundation–soil system is referred to as soil–structure interaction (SSI). Such interactions often, but not always, result in a reduction of seismic base shear. This reduction is caused by the flexibility of the foundation–soil system, associated lengthening of the fundamental period of vibration of the structure, and potentially by damping caused by soil–foundation interaction. However, the total lateral displacements typically increase when soil–structure interaction effects are considered.

If the foundation and soil are considered to be rigid, the computed base shears are usually conservative, and it is for this reason that the designer may often neglect soil–structure interaction. Nonetheless, consideration of soil–structure interaction is always allowed, and its effects can always be considered in accordance with Section 12.13.3 or implicitly in accordance with Chapter 19. There are, however, conditions where neglecting soil–structure interaction can be unconservative. The requirements in the provisions for including lateral soil flexibility in the analysis model are intended to address such conditions. These have been documented in studies such as FEMA P-2091/ATC-144, and NIST GCR 12-917-21/ATC-83. Of these potentially unconservative conditions, differences in the lateral stiffness of foundations have been identified by the PUC of being of highest concern and is addressed by these provisions. The designer may want to consider modeling soil–structure interaction to address other potentially unconservative conditions. There are two specific conditions identified in the provisions where differences in lateral soil flexibility of foundations need to be modeled.

The first condition occurs when the seismic force resisting system is supported by a mix of deep and shallow foundations. This condition causes the foundation system to have significant difference in lateral stiffnesses between SFRS elements, causing different lateral displacements. Piles in bending and shallow foundation base friction have very different load–deformation curves that lead to significantly different effective lateral stiffnesses. These differences in stiffness can affect distributions of internal demands within superstructure elements as well as global response of the

building (e.g., torsion). These complex effects often lead to some elements in the SFRS being under designed compared to those in the fixed base model.

The second condition occurs when there are significant differences in effective lateral deformation of pile caps. This could occur for a number of reasons such as pile caps having different length piles, varied quantity of piles, or if battered piles are used. The two times displacement trigger used is substantiated by the ATC 140 project studies on buildings on impacts to buildings with mixed foundation types.

The exception for flexible diaphragms recognizes that flexible diaphragms allow each line of resistance to move independently from one another. As a result, the relative stiffness of lines of resistance and their supporting foundations do not affect the distribution of forces to the SFRS, thus modeling the lateral soil flexibility would not change the force distribution and is not required.

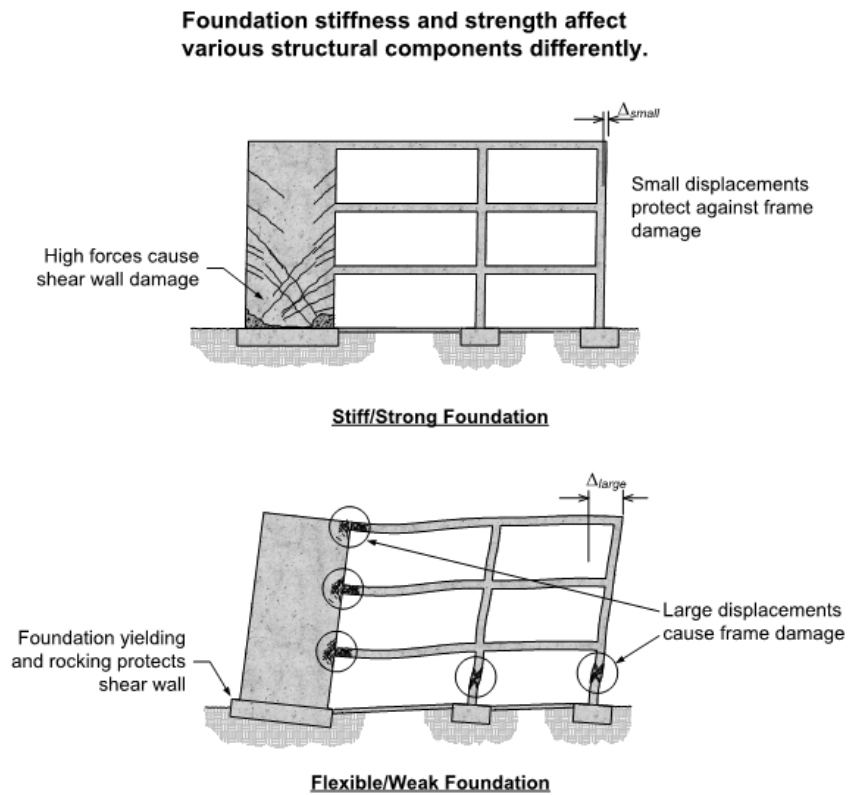
Beyond the requirements of the provisions there are several additional conditions where the designer might consider modeling the soil flexibility in the analytical model

Vertical or rotational soil flexibility

It should be noted that variability in vertical soil stiffness can also have significant impact on the structural performance, specifically with stiff SFRS that see larger overturning loads. This condition also is very common when there is a mix of deep and shallow foundations; however, this is highly dependent on the foundation layout and pile type. Modeling of vertical/rotational soil springs are not required by this section; however, the designer should be aware that the impact can be significant on the behavior of the structure if there is significant variability in the effective vertical/rotational stiffness across the foundation elements.

These conditions can also exist where there are elements of the SFRS supported on shallow foundations (individual or combined footings) which are not interconnected to other foundation elements of sufficient stiffness to prevent excessive foundation rocking. In such cases it is prudent to include vertical soil flexibility as well as any interconnecting foundation elements in the analytical model to capture the additional rotation. Examples of these conditions are shallow foundations supporting 1) SFRS consisting of shear walls or braced frames and 2) SFRS consisting of moment resisting frames,

- 1) **SFRS consisting of shear walls or braced frames:** Deformation caused by foundation rocking can be especially damaging when the stiff SFRS integral with a potentially rocking foundation is located adjacent to gravity frames that would not experience appreciable rotation under an assumed fixed-base condition that may not be designed to remain ductile when subjected to such drifts. Figure C12.7.1-1 is an illustration showing potential damage to the non-seismic force resisting elements, and Figure C12.7.1-2 illustrates how this condition should be modeled.



Stiff/strong is not always favorable; nor is flexible/weak always conservative.

Figure 2-6 The significant impact of soil flexibility on a reinforced concrete shear wall system (from ATC, 1996).

Figure C12.7.1-1: Example of a shear wall where potential rocking of walls results in deformations not adequately captured when the building is modeled as a fixed base.

Source: FEMA P-2091, Figure 2-6 (ATC 1996)

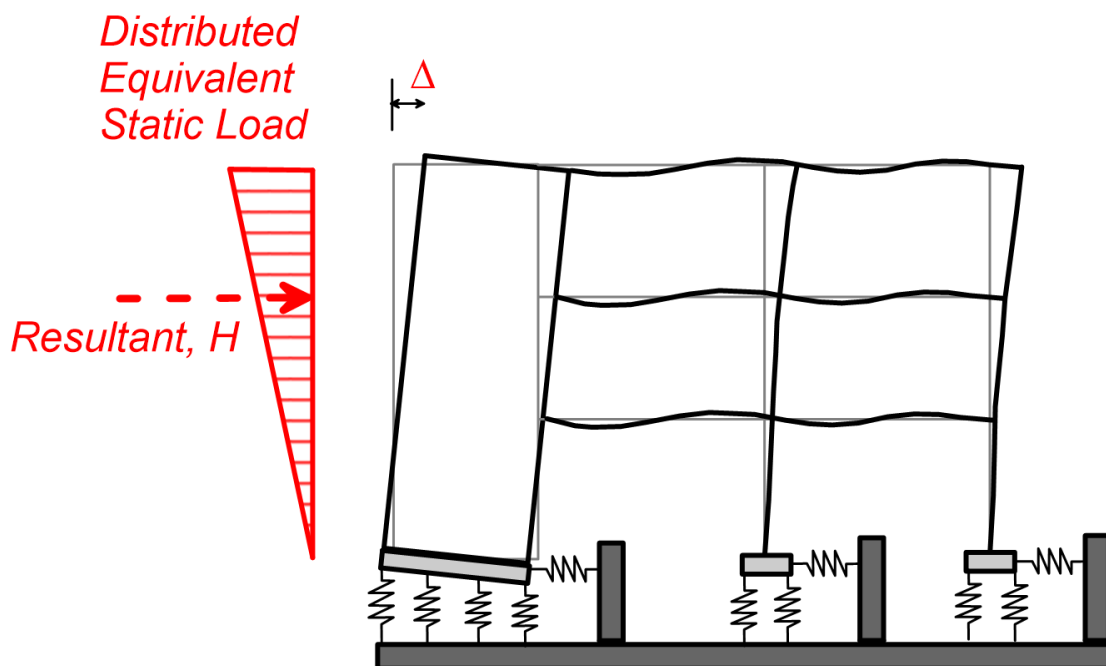


Figure C12.7.1-2: Example of a shear wall with soil springs support which permits consideration of rocking in the seismic response analysis. *Source: Kramer and Stewart (2025), Figure 8.35.*

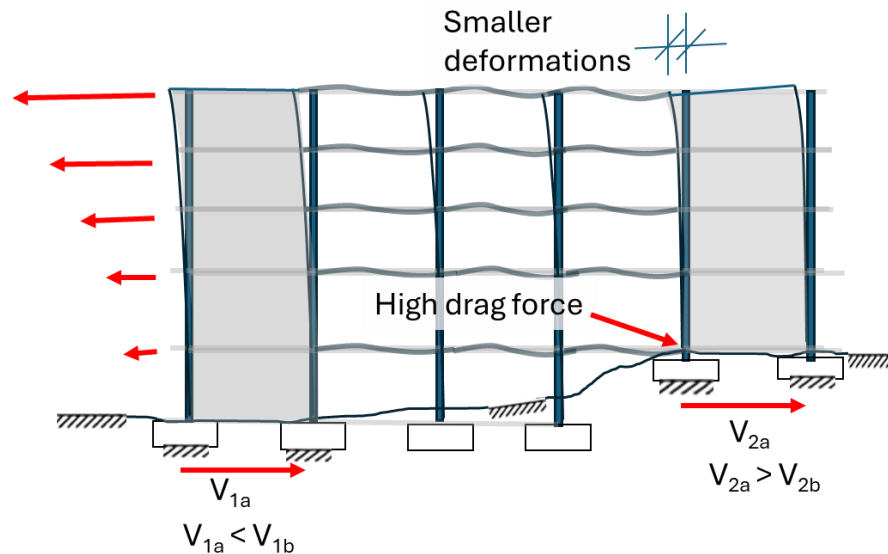
- 2) **SFRS consisting of moment frames:** Another condition where vertical soil flexibility is recommended to be included in the analytical model is where the SFRS consists of moment resisting frames on individual footings or the footings are interconnected by relatively flexible grade beams. Fixed base modeling of these moment frame columns underpredicts drifts and may not identify buildings with a potential soft or weak story. Special moment frame systems have an $R=8$ and are very likely to yield at the base to generate a yield mechanism in the superstructure. However, this requires the foundation support or grade beams to resist the flexural demands at the base of the columns with minimal rotation. Where such resistance is not provided, additional base rotations not accounted for in a fixed-base model increase the demands and drift at the top of the first-floor column and the connecting beams.

The combined vertical and rotational soil flexibility can be modeled either as single uncoupled axial and rotational springs under the vertical SFRS elements or using a finite-element representation of the foundation system with distributed vertical springs distributed over the geometry of the structural footing with an appropriate modulus of subgrade reaction.

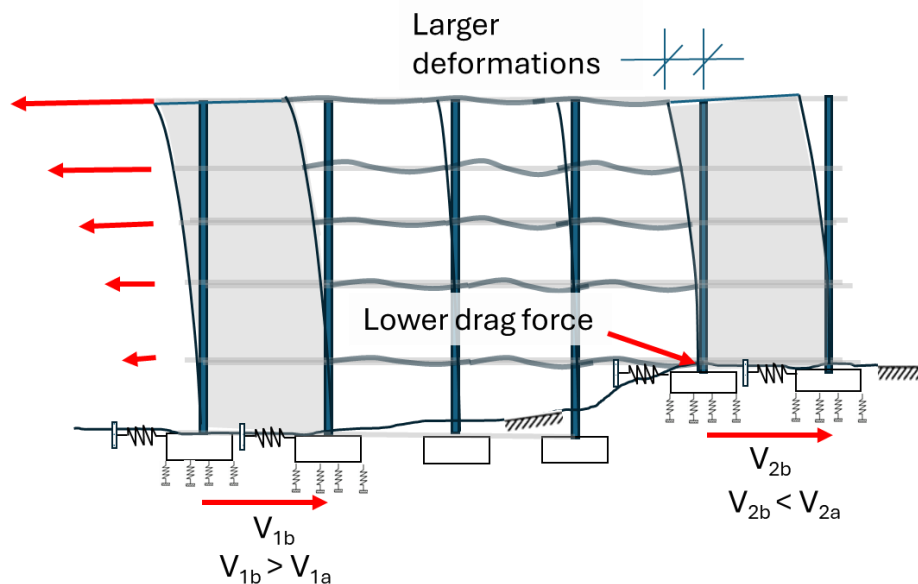
Lateral soil flexibility

In addition to the conditions where lateral soil flexibility is required to be modeled in the provisions, there are conditions where a fixed base model may not adequately capture the superstructure deformations where foundation lateral movement could be significant. Such cases include but are not limited to: 1) SFRS on foundations at different elevations and 2) SFRS where redistribution of force occurs at the base slab to non SFRS footings.

- 1) **SFRS on foundations at different elevations:** For buildings constructed on uneven ground where the tops of the footings are at different elevations, fixed-base modeling will cause more of the base shear to be resisted at the upper-level footings as shown in Figure 12.7.1-3. Modeling the lateral resistance with soil springs reduces the foundation demand at the upper levels and more accurately estimates the lateral resistance at each footing. Not including soil springs may underpredict the shear demands at the lower level and potentially cause the footings to be undersized. Additionally, if the force distribution is inaccurate, global building behavior (e.g., torsion) could also be misrepresented.



a)



b)

Figure C12.7.1-3: Example of a stepped foundation. a) Fixed base foundation supports, b) Foundations supported by soil springs.

- 2) **SFRS where redistribution of force occurs at the base slab to non SFRS footings:** It is often not practical to resist 100% of the lateral base shear from SFRS elements in a line resisting a significant percentage of the base shear (e.g., long shear wall) by only using the foundation element directly supporting the SFRS along that line. This is especially common for pile supported structures where the adjacent gravity piles are used to assist in resisting the lateral load (see Figure C12.7.1-4). When this occurs, the stiffness of the transfer mechanism (e.g., slab-on-grade), the stiffness of the individual lateral elements (e.g., piles in bending), and the geometry of the building all contribute to the distribution of the lateral forces in individual elements. Where the base shear is assumed to distribute uniformly to all foundations, not modeling the lateral soil stiffness could lead to lower than actual lateral demands at the SFRS foundations. This could lead to adverse impacts on foundation elements (e.g., additional pile bending). Several project studies have indicated if less than 80% of the load from a SFRS element is not resisted in line of that element, then modeling lateral soil stiffness along with the slab on grade stiffness is advised. The 80% threshold was set based on engineering judgement advised by some project examples to allow a small amount of load sharing without requiring the soil flexibility to be modeled.

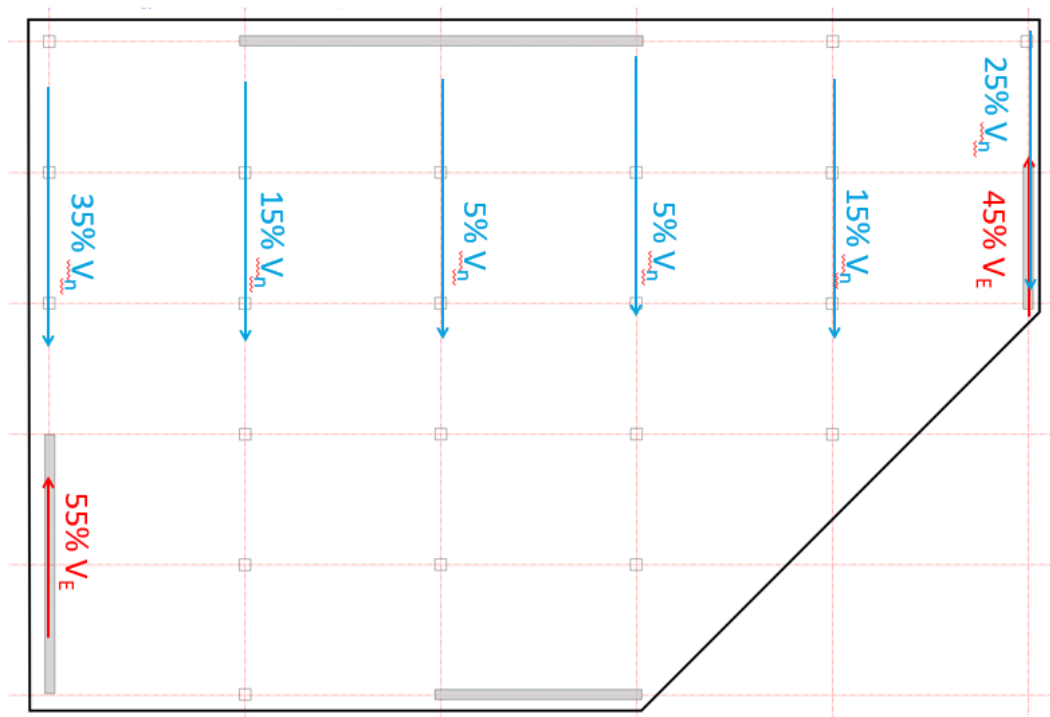


Figure C12.7.1-4: Example of condition where less than 80% of load (e.g., $35/55 = 64\%$. $25/45 = 55\%$) is resisted in line with the SFRS element.

Revise Section C12.7.3 and add Section C12.7.3.1 as follows:

C12.7.3 Structural Modeling

The development of a mathematical model of a structure is always required, because the story drifts and the design forces in the structural members cannot be determined without such a model. In some cases the mathematical model can be as simple as a free-body diagram as long as the model can appropriately capture the strength and stiffness of the structure.

The most realistic analytical model is three-dimensional (3D), includes all sources of stiffness in the structure and the soil–foundation system as well as P-delta effects, and allows for nonlinear inelastic behavior in all parts of the structure–foundation–soil system. Development of such an analytical model is time-consuming, and such analysis is rarely warranted for typical building designs performed in accordance with the standard. Instead of performing a nonlinear analysis, inelastic effects are accounted for indirectly in the linear analysis methods by means of the response modification coefficient, R , and the deflection amplification factor, C_d .

Using modern software, it often is more difficult to decompose a structure into planar models than it is to develop a full three-dimensional model, so three-dimensional models are now commonplace. Increased computational efficiency also allows efficient modeling of diaphragm flexibility. Three-dimensional models are required where the structure has horizontal torsional (Type 1), out-of-plane offset (Type 4), or nonparallel system (Type 5) irregularities.

Analysis using a three-dimensional model is not required for structures with flexible diaphragms that have horizontal out-of-plane offset irregularities. It is not required because the irregularity imposes seismic load effects in a direction other than the direction under consideration (orthogonal effects) because of eccentricity in the vertical load path caused by horizontal offsets of the vertical lateral force-resisting elements from story to story. This situation is not likely to occur, however, with flexible diaphragms to an extent that warrants such modeling. The eccentricity in the vertical load path causes a redistribution of seismic design forces from the vertical elements in the story above to the vertical elements in the story below in essentially the same direction. The effect on the vertical elements in the orthogonal direction in the story below is minimal. Three-dimensional modeling may still be required for structures with flexible diaphragms caused by other types of horizontal irregularities (e.g., nonparallel system).

The provisions require buildings with basements to be modeled down to their foundations. This recognizes the impact of the basement on the dynamics of the building and the impacts on the load path which often includes the basement walls. The structural model(s) is required to extend to the lowest level near the foundation since the elements of the structure below grade contribute to the transmission of the lateral loads into the ground. As a result, the dynamic properties and force distributions within the structure are affected by the basement. When basement walls transfer load out of the superstructure SFRS through the at-grade and below-grade diaphragms, the basement walls see forces which would be neglected if these were not included in the analysis model. In

addition, previous studies (Naeim et al. & Tileylioglu et al.) compared results between models with and without basements and showed that the models with basements (with their masses included) always produced results closer to observed responses from earthquake recordings. The provisions provide an exception for SDC A-C to not model below grade levels. This exception was added to align with the standard of practice for the typical geographic regions aligning with these Seismic Design Categories.

It is not the intent of the provisions that all minor slab depressions are captured in the analysis model (e.g. elevator pits, mechanical pits). The designer may consider including larger slab depressions in their analysis model, specifically when elements of the SFRS extend to these locations.

The provisions specifically note that the structural model(s) should include all above and below-grade elements, this acknowledges that the two stage analysis procedures allow for the building to be split into multiple models. This provision is not intended in any way to limit the application of the two-stage analysis.

In general, the same three-dimensional model may be used for the Equivalent Lateral Force (ELF), the modal response spectrum Analysis (MRSA), and the linear response history (LRH) analysis procedures. Modal Response Spectrum Analysis (MRSA) and linear response history (LRH) analyses require a realistic modeling of structural mass; the response history method also requires an explicit representation of inherent damping. Five percent of critical damping is automatically included in the MRSA approach. Chapter 16 and the related commentary have additional information on linear and nonlinear response history analysis procedures.

It is well known that deformations in the panel zones of the beam-column joints of steel moment frames are a significant source of flexibility. Two different mechanical models for including such deformations are summarized in Charney and Marshall (2006). These methods apply to both elastic and inelastic systems. For elastic structures, centerline analysis provides reasonable, but not always conservative, estimates of frame flexibility. Fully rigid end zones should not be used, because this method always results in an overestimation of lateral stiffness in steel moment-resisting frames. Partially rigid end zones may be justified in certain cases, such as where doubler plates are used to reinforce the panel zone.

Including the effect of composite slabs in the stiffness of beams and girders may be warranted in some circumstances. Where composite behavior is included, due consideration should be paid to the reduction in effective composite stiffness for portions of the slab in tension (Schaffhausen and Wegmuller 1977, Liew et al. 2001).

For reinforced concrete buildings, it is important to address the effects of axial, flexural, and shear cracking in modeling the effective stiffness of the structural elements. Determining appropriate effective stiffness of the structural elements should take into consideration the anticipated demands on the elements, their geometry, and the complexity of the model. Recommendations for computing cracked section properties may be found in Paulay and Priestley (1992) and similar texts.

When dynamic analysis is performed, at least three dynamic degrees of freedom must be present at each level consistent with language in Section 16.2.2. Depending on the analysis software and modal extraction technique used, dynamic degrees of freedom and static degrees of freedom are not identical. It is possible to develop an analytical model that has many static degrees of freedom but only one or two dynamic degrees of freedom. Such a model does not capture response properly.

C12.7.3.1 Base of Structure.

The “base” of structure is defined in section 11.2 which references Section 12.7.3.1 for use in determining seismic forces in accordance with Section 12.8.3.

The following factors affect the location of the seismic base:

- Location of the grade relative to floor levels,
- Depth of basement,
- Slope of grade.

Given these factors the provisions do not prescribe a specific location for where the base of structure shall be modeled, only an upper bound is provided. The following information is provided to assist the designer in selecting the proper base for their project. For structures without basements the vertical elements of the seismic force-resisting system are supported directly on footings or pile caps. If soil-structure interaction is not considered using the procedures in Chapter 19, the base is taken as the top of these foundation elements. Often vertical elements are supported at various elevations on the tops of footings, pile caps, and perimeter foundation walls. Where this occurs, the base is near the lowest elevation of the top of any element supporting the vertical elements of the seismic force-resisting system. However, it is common practice to neglect the distance from the top of foundation to the lowest slab elevation. This difference is likely negligible if that distance is the typical 1-2ft, however there are conditions where foundations are dropped increasing the height of specific walls/frames, which could have a measurable impact to the dynamics of the structure.

The provisions distinguish between the lowest level that is considered in the mathematical model of the structure, which is defined in Section 12.7.3, and level at which lateral forces are considered to be introduced, which is considered to be the base (Section 11.2). Figure C12.7-X shows for a nearly level-ground site a structure founded at the ground surface and a structure with a basement. For the first structure (Figure 12.7-Xa), the lowest level of the model and the base are equivalent. For the second structure (Figure 12.7-Xb), the model extends to the basement level but the base of the structure (i.e., the lowest level where seismic loads are applied in the ELF procedure) is allowed to be located at grade. Modeling the base of the structure at grade neglect seismic forces induced into the structure through inertia developed in basement levels. However, it is possible that these loads are offset by soil on the opposite side of the walls. Depending on the level of shaking and soil resistance, there could be a net lateral force at the below grade levels transmitted into the SFRS and down to the foundation. In these cases, modeling the base of the structure above the foundation level is not conservative.

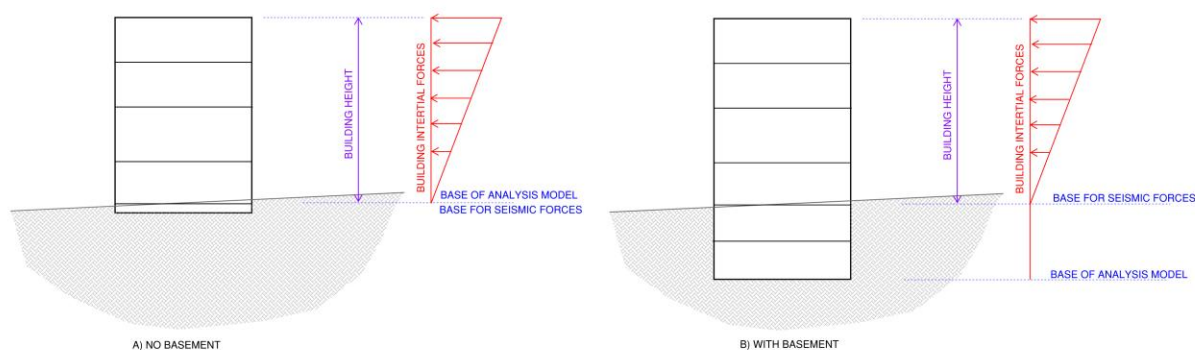


Figure C12.7-X. Base of structure and analysis model for a level site without (A) and with (B) basements.

Note that the base for seismic forces in equivalent lateral force procedures is at grade whereas the base of the analysis model is at the foundation level. This effectively means that the mass at or below the base is ignored in the analysis. While this mass exists, there is also soil directly resisting the movement and this mass is being excited at the peak ground acceleration (not the higher spectral acceleration of the building). Due to the complexity of modeling the below grade mass and lateral resistance of the soil it is permitted to be neglected in most cases.

For structures on sites with sloping grades, the base of the structure for seismic loads is limited based on the side of the structure with the lowest grade as shown in Figure C12.7-XX. The base of the structure aligning with the lowest ground level captures the impact of both the lack of resistance in one direction (allowing the structure to move freely in that direction) and the unbalanced lateral earth pressures including the seismic lateral earth pressures. These earth pressures can act in combination with the superstructure inertial forces without soil resistance on the side of the structure with the lower grade elevation, thus the base is required to be taken at the lower grade level. Where a basement extends beyond the lower level of grade, it is not required to consider a base at these levels. Similar to the level grade condition there are cases where not considering the base of the structure at the foundation level could be non-conservative. The most accurate modeling would be to model the non-linear impacts of soil on one side of the structure, however this is not required by the provisions.

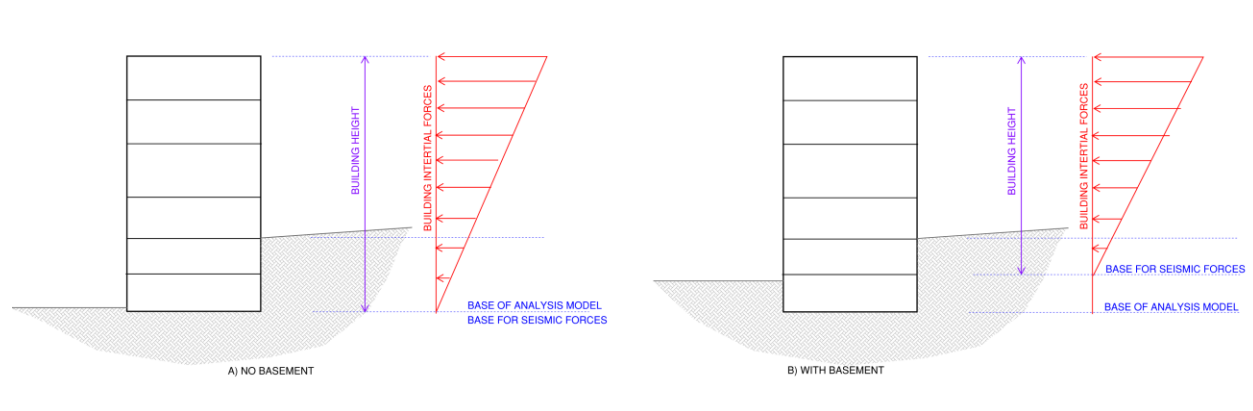


Figure C12.7-XX. Base of structure for sloped site with (A) and without (B) basements.

In the case where liquefiable soils are present it is required to consider the base of the structure at the lowest level below the depth of liquefaction (see Figure C12.7-XXX). This recognizes that the liquefiable soil is unable to provide appreciable resistance to lateral forces and the basement effectively acts as a podium structure. Research at UNR on the effects of liquefaction on ground shaking has generally shown that once liquefaction is triggered, it appreciably affects site response. For this reason, there is currently no technical justification for excluding sites that experience liquefaction under certain conditions (e.g., factor of safety only slightly less than unity, thinner vs thicker liquefiable layers). If the liquefaction extends below the lowest level, the base of the structure is taken as the lowest slab level of the building. Deep foundations would need to consider the liquefiable soil between the bottom of the foundation and the bottom of the liquefiable soil.

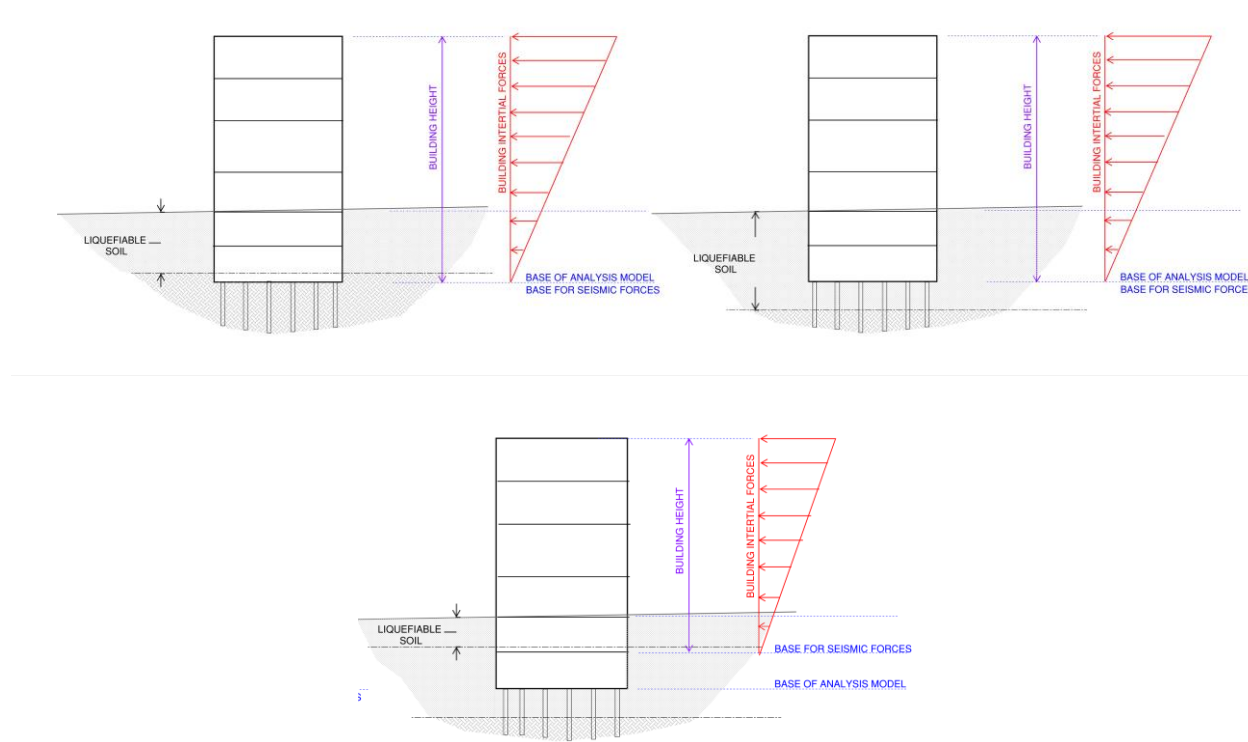


Figure C12.7-XXX: Base of Structure with liquefiable soils present

Other conditions may also necessitate establishing the base below grade for a building with a basement that is located on a level site. Such conditions include those where seismic separations extend through all floors, including those located close to and below grade; those where the floor diaphragms close to and below grade are not tied to the foundation wall; those where the floor diaphragms, including the diaphragm for the floor close to grade, are flexible; and those where other buildings are located nearby. For a building with seismic separations extending through the height of the building including levels close to and below grade, the separate structures are not supported by the soil against a basement wall on all sides in all directions. If there is only one joint through the building, assigning the base to the level close to grade may still be appropriate if the soils over the depth of the basement walls are stiff and the diaphragm is rigid. Stiff soils are required so that the seismic forces can be transferred between the soils and basement walls in both bearing and side friction. If the soils are not stiff, adequate side friction may not develop for movement in the direction perpendicular to the joint.

For large footprint buildings, seismic separation joints may extend through the building in two directions and there may be multiple parallel joints in a given direction. For individual structures within these buildings, substantial differences in the location of the center of rigidity for the levels below grade relative to levels above grade can lead to torsional responses. For such buildings, the base should usually be at the foundation elements below the basement or the highest basement slab level where the separations are no longer provided.

Where floor levels are not tied to foundation walls, the base may need to be located well below grade at the foundation level. An example is a building with tieback walls and post-tensioned floor slabs. For such a structure, the slabs may not be tied to the wall to allow relative movement between them. In other cases, a soft joint may be provided. If shear forces cannot be transferred between the wall and a ground level or basement floor, the location of the base depends on whether forces can be transferred through bearings between the floor diaphragm and basement wall and between the basement wall and the surrounding soils. Floor diaphragms bearing against the basement walls must resist the compressive stress from earthquake forces without buckling. If a seismic or expansion joint is provided in one of these buildings, the base almost certainly needs to be located at the foundation level or a level below grade where the joint no longer exists.

Add Section C12.7.5 as follows:

C12.7.5 Design of below-grade elements

Below-grade elements exposed to soil are subject to both static and seismic lateral earth pressures during an earthquake. This section requires that all below-grade elements are designed for the combined effects of static and seismic earth pressures, which aligns with the requirements for standalone retaining walls in Section 15.6. The section applies to SDC D-F which captures areas of high peak ground accelerations. Studies of relatively flexible walls with a pinned connection at their base that are subjected to ground motions for active tectonic regions (Al Atik and Sitar 2010) have shown that below $\sim 0.3\text{--}0.4g$ PGA there is enough factor of safety in the static earth pressures to account for the seismic increment. The $0.3g$ PGA roughly corresponds to a lower SDC D hazard, thus this provision only is required for SDC D-F. It should be noted that some of the areas of the lower SDC (e.g. B & C) are stable continental regions for which higher-frequency ground motions could be expected that would tend to produce higher seismic earth pressures and thus neglecting seismic earth pressures in SDC C could result in a design that doesn't have an adequate factor of safety.

To align the seismic earth pressures with those used on the rest of the structure the ground motions used to derive the resultant pressures should be based on Design Earthquake (DE) ground motions multiplied by the factors of 1.25 and 1.50 for Risk Category III and IV as described in Section 11.8.3. Since the relationship between ground motions and resultant pressures are not linear, the ground motions not the resultant pressures is what is required to be scaled. The factors of 1.25 and 1.50 to provide increased reliability associated with the higher risk categories. The intent is to align the effective forces being applied to the superstructure and below-grade elements. The factors used are not labeled as "importance factors" because importance factors are applied differently as a force increase factor by reducing R for higher risk categories. For Chapter 16 designs, forces including those resulting from seismic earth pressures, should be derived using the MCE_R ground motion level.

The terms "yielding" and "nonyielding" as applied to walls refers to whether an active wedge would be expected to form in the backfill. Where this occurs, the wall is considered to be "yielding" because of wall displacements associated with active wedge formation. Where no wall displacement occurs, corresponding to at-rest pressures, the wall is considered "nonyielding".

In addition to the out-of-plane forces on localized elements, this section also requires that seismic earth pressures, F_{SE} (often referred to as seismic increment) should be combined with the inertial forces on the superstructure on LFRS elements where there is a net force from seismic earth pressures. Elements that could see loading from both sources (seismic earth pressures and superstructure inertial forces) could include basement shear walls, diaphragms and the elements resisting the base shear. Additionally, this could include the elements resisting the base shear, such as deep foundations or walls resisting passive pressure. There are two primary conditions, one with level grade where passive pressure on the opposite side resists the seismic earth pressures and the other where there is unequal grade and the net forces are resisted at a lower level. The first condition the seismic earth pressures are translated from the basement walls through the diaphragms. These elements also see inertial forces from the superstructure. The second condition, the seismic earth pressure (in one or more directions) has to be resisted by the diaphragm and the vertical SFRS in addition to the inertial forces from the superstructure.

It is likely that the superstructure inertial forces may act out of phase with the seismic earth pressures, Therefore the SRSS combination of the inertial forces and the soil seismic forces are required to be used for design of the below-grade elements. These forces are also required to be combined with the non-seismic lateral earth pressures (defined in Chapter 3) through the load combinations of Section 2.3.6. It should be noted that the inertial forces, F_i , are only required to be considered from above the base of the structure as defined in 12.7.3.1. This section does not require the combination of out-of-plane forces per section 12.11.1 with the seismic earth pressures.

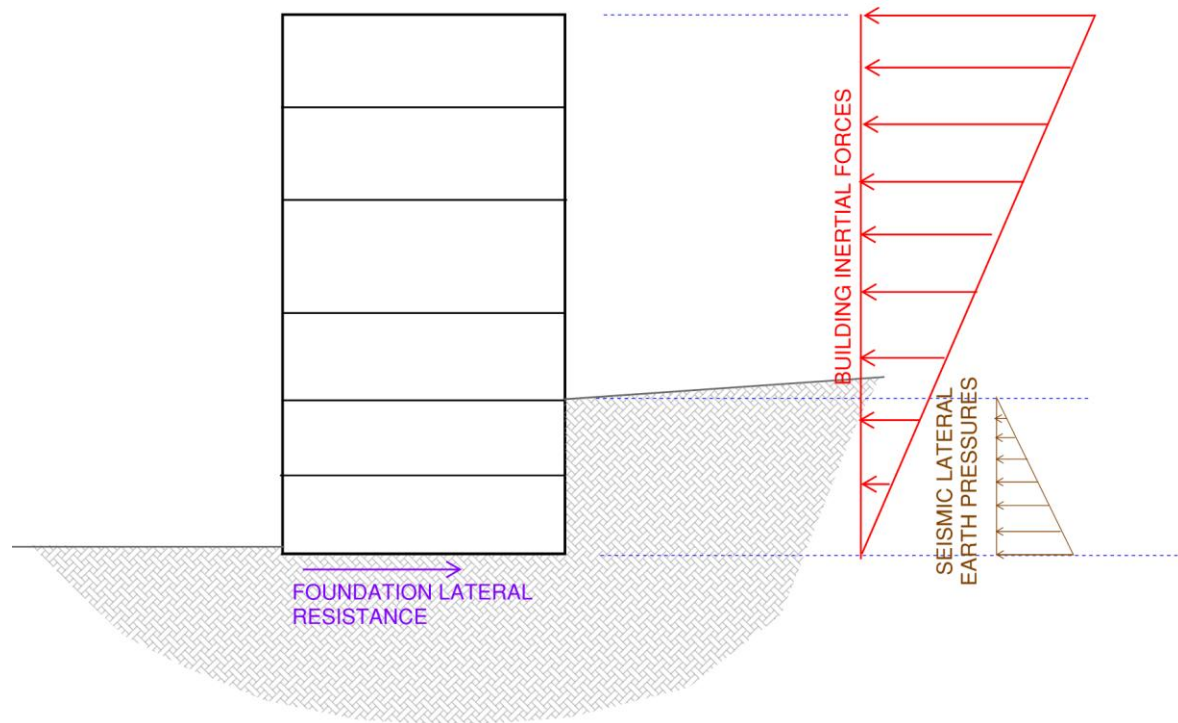


Figure C12.7-X: Hillside structure where seismic internal and seismic lateral earth pressures combine

Revise Section C12.8.3 as follows:

C12.8.3 Vertical Distribution of Seismic Forces

Equation (12.8-13) is based on the simplified first-mode shape shown in Figure C12.8-3. In the figure, F_x is the inertial force at level x , which is simply the absolute acceleration at level x times the mass at level x . The base shear is the sum of these inertial forces, and Equation (12.8-12) simply gives the ratio of the lateral seismic force at level x , F_x , to the total design lateral force or shear at the base, V .

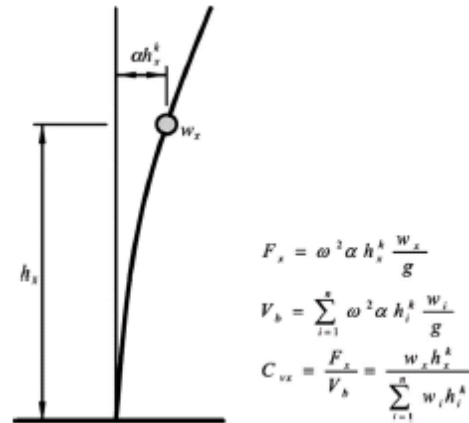


Figure C12.8-3. Basis of Equation (12.8-13).

The deformed shape of the structure in Figure C12.8-3 is a function of the exponent k , which is related to the fundamental period of the structure, T . The variation of k with T is illustrated in Figure C12.8-4. The exponent k is intended to approximate the effect of higher modes, which are generally more dominant in structures with a longer fundamental period of vibration. Lopez and Cruz (1996) discuss the factors that influence higher modes of response. Although the actual first-mode shape for a structure is also a function of the type of seismic force-resisting system (SFRS), that effect is not reflected in these equations. Also, because T is limited to $C_u T_a$ for design, this mode shape may differ from that corresponding to the statistically based empirical formula for the approximate fundamental period, T_a . A drift analysis in accordance with Section 12.8.6 can be conducted using the actual period. Thus, k changes to account for the variation between T and the actual period.

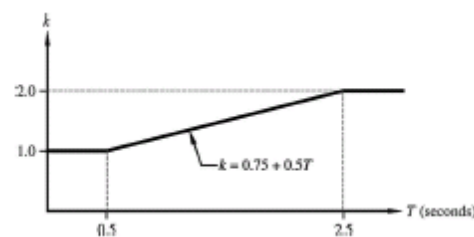


Figure C12.8-4. Variation of exponent k with period T .

The horizontal forces computed using Equation (12.8-12) do not reflect the actual inertial forces imparted on a structure at any particular point in time. Instead, they are intended to provide lateral seismic forces at individual levels that are consistent with enveloped results from more accurate analyses (Chopra and Newmark 1980).

The equations in Figure C12.8-3 apply for a deformed structure with uniform mass, stiffness, and strength over its full height. In addition to other features that lead to the structure having significant

changes in mass or stiffness between levels, basement levels often have a much higher story stiffness than above-grade levels. This will lead to differences in the force distribution between an ELF analysis and other analysis options (e.g. linear dynamic). However based on previous studies the ELF procedure provides conservative results. This is no different than a structure with a larger podium and tower where there is a significant change in stiffness between the two portions of the building.

Revise Section C12.8.6 as follows:

C12.8.6 Displacement and Drift Determination This section defines three types of displacement or drift: the Design Earthquake Displacement (δ_{DE}); the Maximum Considered Earthquake Displacement (δ_{MCE}); and the Design Story Drift (Δ).

The Design Earthquake Displacement corresponds to the design earthquake. It is used for structural separations and deformation compatibility and is computed at the location of the element being evaluated. (Previous editions referred to this quantity as the “maximum inelastic response displacement.”) There is thus a Design Earthquake Displacement at every point in the structure, although evaluations using this quantity are not required at every location. The Design Earthquake Displacement is used for structural separation (Section 12.6.2); deformation compatibility (Section 12.6.4); and nonstructural components (Section 13.3.2).

The Design Earthquake Displacement includes diaphragm deformation and rotation, as center-of-mass displacement could significantly underestimate displacement at the building corners and perimeter, and at non-rigid diaphragm locations away from the vertical elements of the seismic force-resisting system (SFRS, see Figure C12.8-1b). The diaphragm deformation corresponding to the design earthquake is required to be used. (The diaphragm deformation is represented by the term δ_{di} ; diaphragm deformation is therefore not included in the elastic displacement δ_e , which, however, does include displacement and diaphragm rotation effects of the seismic force-resisting system.) The engineer may determine that the diaphragm remains elastic under the expected demands (such as those corresponding to seismic loads including overstrength [Section 12.4.3] and considering diaphragm loading per Section 12.10) or may use rational methods of estimating its inelastic deformation. The engineer may determine that amplification of the elastic diaphragm deformation component by C_d/I_e , R_s/I_e , or $C_{d-diaph}/I_e$ is the appropriate method for diaphragms designed under Sections 12.10.1, 12.10.2 or 12.10.3 respectively. The 2020 NEHRP Provisions provide guidance on force-displacement characteristics of diaphragms (see the white paper, “Calculation of Diaphragm Deflections under Seismic Loading,” Part 3).

The deflection amplification factor, C_d , in Equation (12.3-17) amplifies the displacements computed from an elastic analysis using prescribed forces to represent the expected inelastic displacement for the design-level earthquake and is typically less than R (Section C12.1.1). It is important to note that C_d is a story-level amplification factor and does not represent displacement amplification of the elastic response of a structure, modeled either as an *effective* single-degree-of-freedom structure (fundamental mode) or a constant amplification to represent the deflected shape of a multiple-degree-of-freedom structure, implying, in effect, that the mode shapes do not change during inelastic

response. Furthermore, drift-level forces are different from the design-level forces used for strength compliance of the structural elements. Drift forces are typically lower, because the computed fundamental period can be used to compute the base shear (Section C12.3.3.6.2).

When conducting a drift analysis, the analyst should be attentive to the applied gravity loads used in combination with the strength-level earthquake forces so that consistency is maintained between the forces used in the drift analysis and those used for stability verification (P-delta) in Section 12.3.3.7, including consistency in computing the fundamental period if a second-order analysis is used. Further discussion is provided in Section C12.3.3.7.

The design forces used to compute the elastic deflection, δ_e , include the Importance Factor, I_e , so Equation (12.3-17) includes I_e in the denominator. This inclusion is appropriate because the allowable story drifts in Table 12.6-1 (except for masonry shear wall structures) are more stringent for higher Risk Categories.

Revise Section C12.10 as follows:

C12.10 DIAPHRAGMS, CHORDS, AND COLLECTORS

This section permits a choice of diaphragm design in accordance with Sections 12.10.1, 12.10.2, or 12.10.3. The diaphragm seismic design provisions in Section 12.10.1 are the non-system-based diaphragm design method. Section 12.10.2 is an alternative system-based diaphragm design method, first included in the 2016 edition of this standard. Section 12.10.3 provides another alternative design method, first introduced in the 2022 edition. The Section 12.10.3 method is permitted only for one-story structures employing flexible diaphragms with rigid vertical elements. For a given diaphragm, one of these three methods is to be selected and implemented. Where a group of diaphragms is similar enough in elevation that they would be anticipated to interact, the use of one diaphragm design method for the group of diaphragms is recommended.

Section 12.10.2 provides diaphragm seismic design provisions that specifically recognize and account for the effect of the diaphragm system ductility and displacement capacity on the diaphragm design forces. This is accomplished with the introduction of a diaphragm design force reduction factor, R_s . Neither the number of stories nor the building configuration is restricted by the Section 12.10.3 provisions; however, diaphragm construction is limited to the diaphragm systems specifically noted within those provisions.

Section 12.10.2 is mandatory for precast concrete diaphragms and welded bare steel deck diaphragms in structures assigned to Seismic Design Category C, D, E, or F and is optional for the following diaphragm systems in structures assigned to all Seismic Design Categories:

- Precast concrete diaphragms in Seismic Design Category B,
- Welded bare steel deck diaphragms in Seismic Design Category B,

- Cast-in-place concrete diaphragms,
- Cast-in-place concrete equivalent precast diaphragms,
- Wood sheathed diaphragms on wood framing,
- Bare steel deck diaphragms,
- Concrete-filled steel deck diaphragms, and
- Rod-braced diaphragms

Precast concrete diaphragms and cast-in-place concrete equivalent precast diaphragms are defined in Section 14.2. The requirement to use Section 12.10.2 is limited to precast concrete diaphragms and not cast-in-place concrete equivalent precast diaphragms. The required mandatory use of Section 12.10.2 for precast diaphragm systems in Seismic Design Categories C through F buildings is based on recent research that indicates that improved earthquake performance can thus be attained. Many conventional diaphragm systems designed in accordance with Section 12.10.1 have performed adequately. Continued use of Section 12.10.1 is considered reasonable for diaphragm systems other than those for which Section 12.10.2 is mandated.

New in the 2028 Edition, bare steel deck diaphragms fastened with welds are required to be designed per Section 12.10.2 or 12.10.3. Both of these sections permit lower diaphragm design forces for bare steel deck diaphragms that meet AISI special seismic detailing provisions and higher forces for diaphragms that do not. Bare steel deck diaphragms fastened with welds do not meet the special seismic detailing requirements, and as a result are required to be designed for the higher forces. By prohibiting use of the 12.10.1 non-system-based diaphragm forces, the less ductile behavior of these diaphragms will always be recognized in design. Further technical background on the differentiation of these diaphragm types is provided in the commentary to Sections 12.10.2 and 12.10.3.

Section 12.10.3 diaphragm seismic design provisions are permitted to be used for one-story structures combining flexible diaphragms with rigid vertical elements. The seismic design methodology specifically recognizes the dynamic response of these structures as being dominated by dynamic response of and inelastic behavior in the diaphragm. While the most common occurrences of this structure type are the concrete tilt-up and masonry “big box” structures, the “rigid vertical element” terminology of this section recognizes a wider range of vertical elements for which this methodology is permitted to be used. The primary use of Section 12.10.3 provisions is intended to be for structures where one or more spans of the diaphragm exceed 100 ft (30.5 m); however, use for structures in which all diaphragm spans are less than 100 ft (30.5 m) is not precluded.

These provisions are intended to address all diaphragm systems. Section 12.10 further emphasizes that its provisions apply to all portions of a diaphragm system. For purposes of Section 12.10, the terms diaphragm is intended to be consistent with the Chapter 11 definitions.

C12.10.1 Non-System-Based Diaphragm Design Method

The provisions of Section 12.10.1 are the method by which diaphragm seismic design forces have traditionally been determined. Section 12.10.2 also emphasizes that the diaphragm is to be designed for seismic forces in two orthogonal directions.

C12.10.1.1 Diaphragm Design Forces

Diaphragms must be designed to resist inertial forces, and in addition to transfer forces acting through the diaphragm between vertical seismic force-resisting elements. Inertial forces are those seismic forces that originate at the diaphragm level, whereas transfer forces originate from locations other than the diaphragm level under consideration. Transfer forces between vertical seismic force resisting elements may occur due to conditions such as, but not limited to, geometric offsets that cause the diaphragm to serve as an intermediary load path, differential stiffness between elements attached to a shared diaphragm through which deformation compatibility forces are induced, and interaction with external restraints. Examples of these conditions include horizontal out-of-plane offset irregularities in the vertical lateral-force-resisting system, shear-wall or brace-frame interaction effects with moment frames on separate lines in dual systems, and backstay effects. The redundancy factor, p (or the overstrength factor, Ω_0 , where required), that is used for design of the seismic force-resisting elements also applies to diaphragm transfer forces, thus completing the load path.

Buildings that contain below-grade levels may have either continuous SFRSs (as seen in figure C12.7.3) or discontinuous SFRSs. For vertical elements of continuous SFRS that extend below-grade down to the foundation (Figure C12.7.3), the below-grade structure can act as the backspan to what is typically idealized as a cantilevered lateral system. This interaction between the lateral system and below-grade structure is known as the backstay effect. Vertical elements of discontinuous SFRS that stop at grade typically shed all their shear load to the perimeter basement walls, thus not experiencing the backstay effect. The backstay effect can be illustrated in Figure 12.7.3.

The degree of interaction between the lateral system and below-grade structure depends on several factors, including the stiffness of the below-grade wall structure, depth of the below-grade structure, and stiffness of the transfer diaphragm at grade level, which includes the basement wall's contribution to stiffness and the continuity of the lateral system below-grade. In many cases, the backstay effect can increase the shear demand on the SFRS and the backstay diaphragms (Gardiner, et al. 2008 and Yacoubian, et al. 2017). In some cases, the shear demand on the SFRS can reverse direction if the below grade structure is stiff enough. Since the grade level transfer diaphragm plays the most significant role in contributing to the backstay effect, amplifying the transfer forces that occur due to this condition by a factor between 1.0 and Ω_0 of the SFRS may need to be considered.

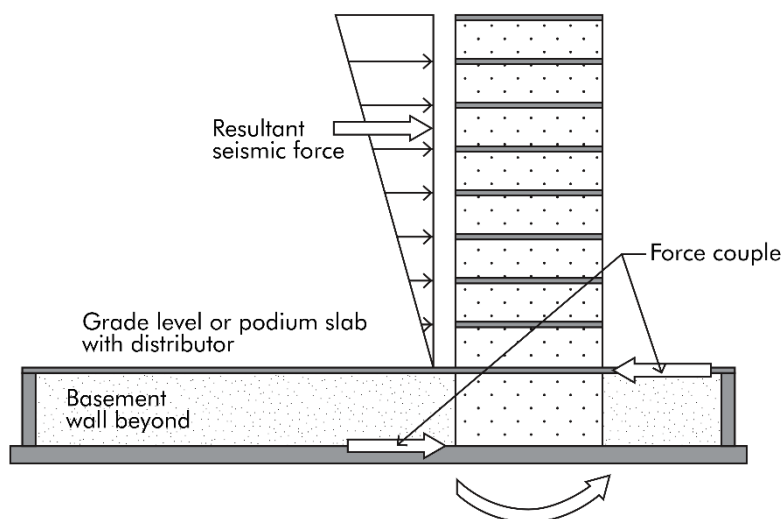


Figure C12.7.3. Illustration of backstay effect, showing the force couple between the grade level podium and below-grade levels to help resist overturning

Source: Adapted from Moehle et al. (2010)

Renumber Figures 12.7-3 through 12.7-9 as Figures 12.7-4 through 12.7-10. Update all references to these figures.

The weight tributary to a diaphragm, w_{px} , may conservatively be taken as the weight of the level, however only lateral masses that directly induce diaphragm forces need to be considered. Lateral mass whose inertial forces do not pass through the diaphragm (such as the self-weight of walls parallel to the considered direction that resist their own in-plane forces) need not be included. The weight tributary to a diaphragm may be different along each orthogonal direction.

The seismic forces given in Equations (12.10-1) through (12.10-3) are usually larger than those obtained for the design of the vertical elements of the seismic force resisting system. In particular, the ELF procedure is designed to envelope the story shear demands, and may not envelope peak story accelerations. This procedure reflects the expected dynamic horizontal amplification of floor and roof motion relative to the vertical system that has been observed in earthquakes. Transfer forces are generally independent of the acceleration of diaphragm mass; thus their magnitude is not affected by Equations (12.10-1) through (12.10-3). Note that while Equation (12.10-3) limits the maximum result of Equation (12.10-1), it does not limit the design forces determined from the structural analysis, should they be larger.

C12.10.1.2 Collector Elements for Structures in Seismic Design Categories C through F

The overstrength requirement of this section is intended to keep inelastic behavior in the ductile elements of the seismic force-resisting system (consistent with the response modification coefficient, R) rather than in collector elements.

C12.10.2 System-Based Diaphragm Design Method

This section was titled “Section 12.10.3 Alternative Design Provisions for Diaphragms, Including Chords and Collectors” in the 2015 and 2020 NEHRP Provisions and ASCE 7-16 and 7-22. It has been renamed as part of the rework of Section 21.10 to indicate it as a main-stream method and to provide a title that better indicates the method’s consideration of diaphragm system effects on diaphragm forces. The provisions of Section 12.10.2 are being mandated for precast concrete diaphragms and welded bare steel deck diaphragms in buildings assigned to Seismic Design Category C, D, E, or F and are offered as an alternative to those of Section 12.10.1 for a number of other diaphragm systems.

During initial development of the Section 12.10.2 diaphragm design provisions as part of the 2015 NEHRP Provisions update effort, studies found that the level of diaphragm design force from the non-system-based diaphragm design method (Section 12.10.1) may not ensure, that diaphragms have sufficient strength and ductility to mobilize the inelastic behavior of vertical elements of the seismic force-resisting system. Analytical and experimental results show that actual diaphragm forces over much of the height of a structure during the design-level earthquake may be significantly greater than those from the Section 12.10.1 the non-system-based design method, particularly when diaphragm response is near-elastic. There are diaphragm-system-specific factors that are related to overstrength and deformation capacity that may account for the adequate diaphragm performance in past earthquakes. The provisions of Section 12.10.2 consider both the significantly greater forces observed in near-elastic diaphragms and the anticipated overstrength and deformation capacity of diaphragms, improving the distribution of diaphragm strength over the height of buildings and among buildings with different types of seismic force-resisting systems.

Based on experimental and analytical data and observations of building performance in past earthquakes, changes appeared to be warranted to the procedures of the non-system-based design method of Section 12.10.1 for some types of diaphragms and for some locations within structures. Examples include the large diaphragms in some parking garages. The system-based diaphragm design method (currently Section 12.10.2) implements the warranted changes.

Section 12.10.2, Item 1, footnote b to Table 12.2-1, permits reducing Ω_0 for structures with flexible diaphragms. The lower Ω_0 results in lower diaphragm forces, which is not consistent with experimental and analytical observations. Justification for footnote b is not apparent; therefore, to avoid the inconsistency, the reduction is eliminated when using the Section 12.10.2 design provisions.

Section 12.10.2, Item 2. Section 12.3.3.5 provision requiring a 25% increase in design forces for certain diaphragm elements in buildings with several listed irregularities is eliminated when using the Section 12.10.2 design provisions, because the diaphragm design force level in this section is based on realistic assessment of anticipated diaphragm behavior. Under the Section 12.10.1 design provisions, the 25% increase for collectors and their connections from Section 12.3.3.5 Item 2 is invariably superseded by the requirement to amplify seismic design forces for certain diaphragm

elements by Ω_0 ; the only exception is wood diaphragms, which are exempt from the Ω_0 multiplier. See Section C12.3.3.5 for further discussion.

Section 12.10.2, Item 3. Sections 12.10.2.2 and 12.10.2.3 provide realistic seismic design forces for diaphragms. Section 12.10.2.4 requires that diaphragm collectors be designed for an amplified force level used for diaphragm in-plane shear and flexure. Based on these forces, the use of a p factor greater than 1 for collector design is not necessary and would overly penalize designs. This is reflected in Section 12.3.4.1, Item 8.

C12.10.2.1 Design

The phrase “diaphragms including chords, collectors, and their connections to the vertical elements” is used consistently throughout the provisions, to emphasize that its provisions apply to all portions of a diaphragm. It is also emphasized that the diaphragm is to be designed for motions in two orthogonal directions for inertial and transfer forces. Diaphragm design forces are the sum of inertial forces calculated in accordance with Section 12.10.2.2 and diaphragm transfer forces calculated in accordance with 12.10.2.3.

C12.10.2.2 Inertial Seismic Design Forces

Similar to Section 12.10.1.1, Section 12.10.2.2 requires that diaphragm inertial forces be taken as the greater of F_x forces used in design of the vertical elements and F_{px} inertial forces. This is new in ASCE 7-28 and intended to make the non-system-based design provisions of Section 12.10.1 and diaphragm system-based provision of Section 12.10.2 more consistent. Equation (12.10-4) makes the diaphragm inertial seismic design force equal to the weight tributary to the diaphragm, w_{px} , times a diaphragm design acceleration coefficient, C_{px} , divided by a diaphragm design force reduction factor, R_s , which is material-dependent and whose background is given in Section C12.10.3.5. The background to the diaphragm design acceleration coefficient, C_{px} , is given below.

The diaphragm design acceleration coefficient at any height of the building can be determined from linear interpolation, as indicated in Figure 12.10-2. The diaphragm design acceleration coefficient at the building base, C_{p0} , equals the peak ground acceleration consistent with the design response spectrum in Section 11.4.5.2, times the Importance Factor, I_e . Note that the term $0.4S_{DS}$ can be calculated from Equation (11.4-3) by making $T = 0$.

The diaphragm design acceleration coefficient at 80% of the structural height, C_{pi} , given by Equations (12.10-9) and (12.10-10), reflects the observation that at about this height, floor accelerations are largely, but not solely, contributed by the first mode of response. In an attempt to provide a simple design Equation, C_{pi} was formulated as a function of the design base shear coefficient, C_s , which may be determined from equivalent static analysis or modal response

spectrum analysis (MRSA) of the structure. Note that C_s includes a reduction by the response modification factor, R , of the seismic force-resisting system. It is magnified back up by the overstrength factor, Ω_0 , of the seismic force-resisting system because overstrength will generate higher first-mode forces in the diaphragm. In many lateral systems, at 80% of the building height, the contribution of the second mode is negligible during linear response, and during nonlinear response it is typically small, though nonnegligible. In recognition of this observation, the diaphragm seismic design coefficient at this height has been made a function of the first mode of response only, and the contribution of this mode has been factored by $0.9\Gamma_{m1}$ as a weighed value between contributions at the first-mode effective height (approximately 2/3 of the building height) and the building height.

Systems that make use of high R -factors, such as buckling-restrained braced frames (BRBFs), and moment-resisting frames (MRFs), show that in the lower floors the higher modes add to the accelerations, whereas the contribution of the first mode is minimal. For this reason, the coefficient C_{pi} needs to have a lower bound. A limit of C_{p0} has been chosen; it makes the lower-floor acceleration coefficients independent of R . Wall systems are unlikely to be affected by this lower limit on C_{pi} .

At the structural height, h_n , the diaphragm design acceleration coefficient, C_{pn} , given by Equation (12.10-7), reflects the influence of the first mode, amplified by system overstrength, and of the higher modes without amplification, on the floor acceleration at this height. The individual terms are combined using the square root of the sum of the squares. The overstrength amplification of the first mode recognizes that the occurrence of an inelastic mechanism in the first mode is an anticipated event under the design earthquake, whereas inelastic mechanisms caused by higher-mode behavior are not anticipated. The higher-mode seismic response coefficient, C_{s2} , is computed as the smallest of the values given by Equations (12.10-11), (12.10-12), and (12.10-13a) or (12.10-13b). These four Equations consider that the periods of the higher modes contributing to the floor acceleration can lie on the ascending, flat-top, or first descending branch of the design response spectrum shown in Figure 11.4-1. Users are warned against extracting higher modes from their modal analysis of buildings and using them in lieu of the procedure presented in Section 12.10.2.2.1, because the higher-mode contribution to floor accelerations can come from a number of modes, particularly when there is lateral-torsional coupling of the modes.

Note that Equation (12.10-7) makes use of the modal contribution factor, defined here as the mode shape ordinate at the building height times the modal participation factor and is uniquely defined for each mode of response (Chopra 1995). A building database was compiled to obtain approximate Equations for the first-mode and higher-mode contribution factors. The first and second translational modes, as understood in the context of two-dimensional modal analysis, were extracted from the mode shapes obtained from three-dimensional modal analysis by considering modal ordinates at the center of mass. These buildings had diverse lateral systems, and the number of stories ranged from 3 to 23. Equations (12.10-14) and (12.10-15) were empirically calibrated from simple two-

dimensional models of realistic frame-type and wall-type buildings and then compared with data extracted from the database (Figure C12.10-3). In Equation (12.10-7), C_{pn} is required to be no less than C_{pi} , based on judgment, to eliminate instances where the design acceleration at roof level might be lower than that at $0.8h_n$. This cap will particularly affect low- z_s systems such as BRBFs.

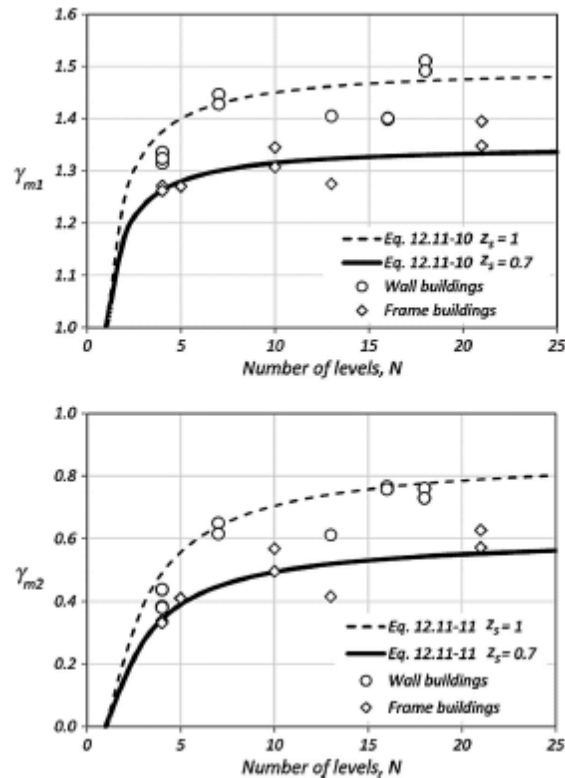


Figure C12.10-3. Comparison of factors Γ_{m1} and Γ_{m2} obtained from analytical models and actual structures with those predicted by Equations (12.10-14) and (12.10-15).

To validate Equation (12.10-1), C_{px} coefficients were calculated for various buildings tested on a shake table. Figures C12.10-4 and C12.10-5 plot the floor acceleration envelopes and the floor accelerations predicted from Equation (12.10-4), with $R_s = 1$, for two buildings built at full scale and tested on a shake table (Panagiotou et al. 2011, Chen et al. 2015), with C_{p0} defined as the diaphragm design acceleration coefficient at the structure base and C_{px} defined as the diaphragm design acceleration coefficient at level x . Measured floor accelerations are reasonably predicted by Equation (12.10-4). Research work by Choi et al. (2008) concluded that BRBFs are very effective in limiting floor accelerations in buildings arising from higher-mode effects. This finding is reflected in this proposal, where the mode shape factor, z_s , has been made the smallest for BRBF systems.

Figure C12.10-6 compares average floor accelerations obtained from the nonlinear time history analyses of four buildings (two steel BRBF systems and two steel special moment frame systems)

when subjected to an ensemble of spectrum-compatible earthquakes with floor accelerations computed from Equations (12.10-4) and (12.10-5). The design Equations predict the accelerations in the uppermost part of the building and in the lowest levels reasonably well.

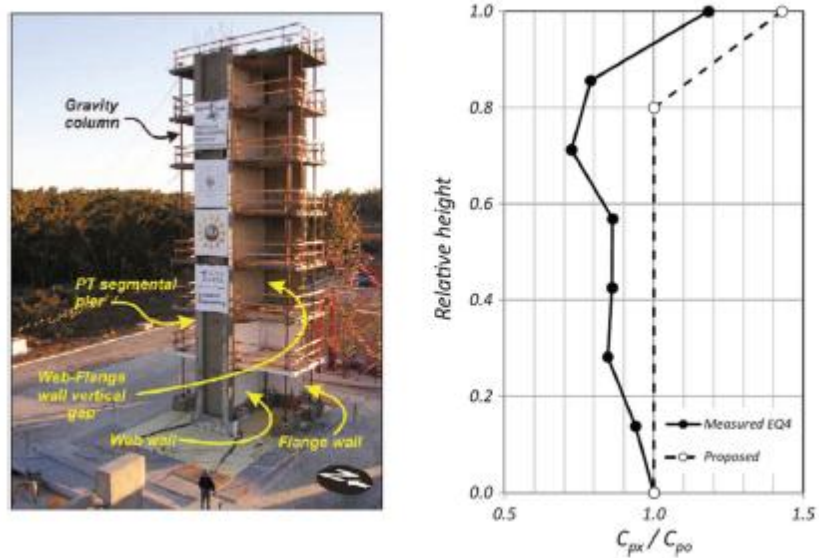


Figure C12.10-4. Comparison of measured floor accelerations and accelerations predicted by Equation (12.10-1) for a seven-story bearing-wall building.

Source: Panagiotou et al. (2011).

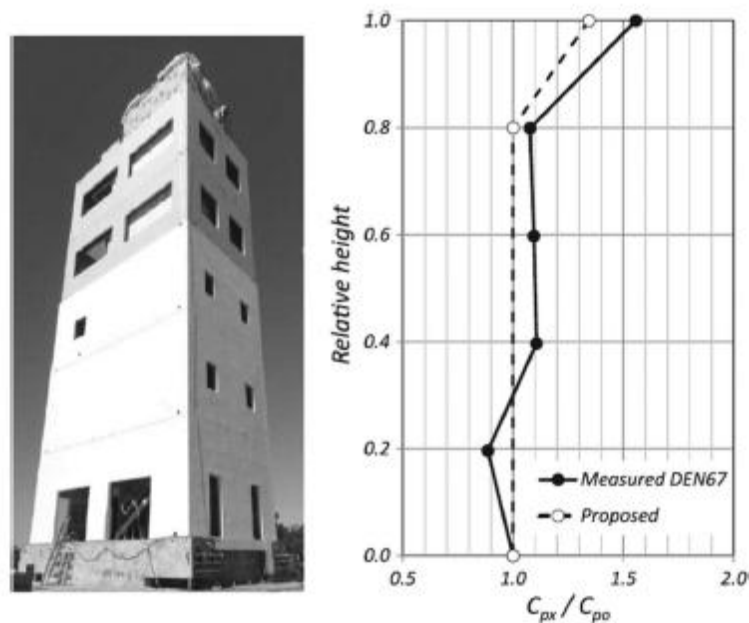


Figure C12.10-5. Comparison of measured floor accelerations and accelerations predicted by Equation (12.10-4) for a five-story special moment-resisting frame building.

Source: (left) Courtesy of Michelle Chen; (right) adapted from Chen et al. (2015).

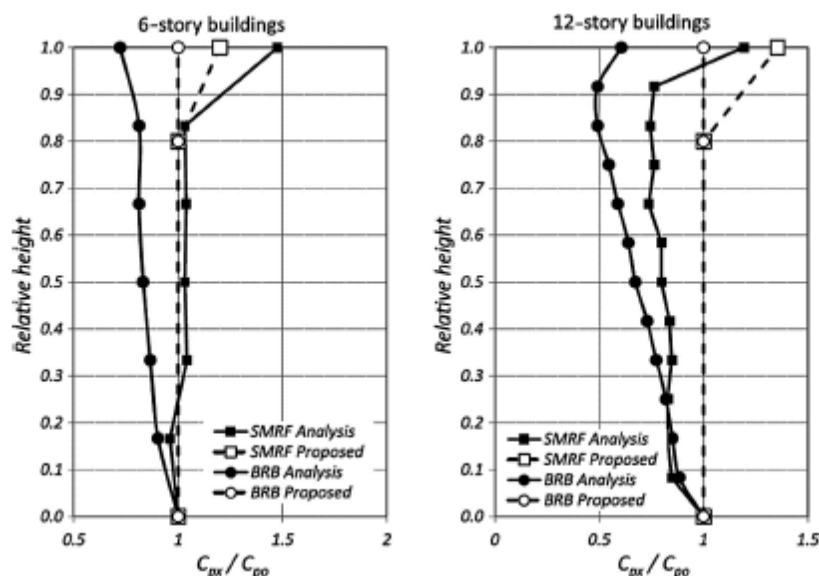


Figure C12.10-6. Comparison of measured floor accelerations with Equations (12.10-4) and (12.10-5) for steel buckling-restrained braced frame and special moment-resisting frame buildings.

Source: Adapted from Choi et al. (2008).

The significant difference between a low- z_s system such as the BRBF and a high- z_s system such as a bearing-wall system is that inelastic deformations are distributed throughout the height of the structure in a low- z_s system, whereas they are concentrated at the base of the structure in a high- z_s system. If rational analysis can be performed to demonstrate that inelastic deformations are in fact distributed along the height of the structure, as is often the case with eccentrically braced frame or coupled shear wall systems, then the use of a low z_s value, as has been assigned to the BRBF, for such a system would be justified.

During the calibration of the design procedure leading to Equation (12.10-4), it was found that at intermediate levels in lateral systems designed using large response modification coefficients, diaphragm design forces given by this Equation could be rather low. There was consensus within the Building Seismic Safety Council Provisions Update Committee (BSSC PUC) Issue Team that developed Section 12.10.2 that diaphragm design forces should not be taken as less than the minimum force currently prescribed by ASCE 7, and hence they developed Equation (12.10-5).

The procedure presented in Section 12.10.2 is based on consideration of buildings and structures whose mass distribution is reasonably uniform along the building height. Buildings or structures with tapered mass distribution along their height or with setbacks in their upper levels may experience diaphragm forces in the upper levels that are greater than those derived from Equation (12.10-4). In

such buildings and structures, it is preferable to define an effective building height, h_{ne} , and a corresponding level, n_e , the level to which the structural effective height is measured. The effective number of levels in a building, n_e , is defined as x , where the ratio $\sum_{i=1}^x w_i / \sum_{i=1}^n w_i$ first exceeds 0.95. Level 1 is defined as the first level above the base. The effective structural height, h_{ne} , is the height of the building measured from the base to level n_e . In buildings with tapered mass distribution or setbacks, the diaphragm design acceleration coefficient, C_{pn} , is calculated by interpolation and extrapolation, as shown in Figure C12.10-7, with n replaced by n_e in Equations (12.10-10) through (12.10-14).

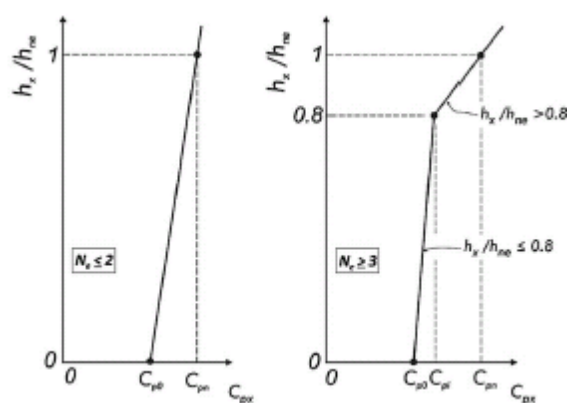


Figure C12.10-7. Diaphragm design acceleration coefficient, C_{px} , for buildings with nonuniform mass distribution.

A simplified formulation of C_{pn} is provided in Equation 12.10-8 for buildings five stories or less in height. This formulation was derived from Equation 12.10-7 by using the value of C_{s2} as given by Equation 12.10-11, which governs for buildings five stories or less in height, conservatively assuming the mode shape factor, z_s , to be equal to unity in determining the modal contribution factors, and by making further algebraic simplifications. The derivation assumes that the building's spectral response acceleration is on the peak of the response spectrum, and therefore, this simplified formula is only applicable where $T \leq T_s$. This results in a single formula to compute C_{pn} . This simplified formulation results in values of the design acceleration coefficient at the roof, C_{pn} , that nearly equal or exceed those using the more complex Equation 12.10-7.

The provisions permit the calculation of C_{px} in accordance with Figure 12.10-2(a) with a linear interpolation between C_{p0} and C_{pn} , for buildings up to five stories tall, and with Figure 12.10-2(b) with a bi-linear interpolation between C_{p0} , C_{pi} , and C_{pn} , for buildings three stories or taller in height. For buildings that are three to five stories tall, the user may compute C_{px} in accordance with either figure, recognizing that the interpolation using Figure 12.10-2(a) will often be conservative at intermediate levels. Furthermore, where C_{pn} is permitted to be calculated with the simplified formulation in

Equation 12.10-8, for buildings that are three to five stories tall, the user is permitted to calculate C_{px} in accordance with either figure.

For a five-story special reinforced concrete shear wall building, Figure C12.10-8 illustrates a comparison of the value of C_{pn} at the top story using Equation 12.10-7 and the simplified formulation with Equation 12.10-8, and the results are nearly identical. The figure also illustrates a comparison of the values of C_{px} at the intermediate levels computed using Figure 12.10-2(a) with a linear interpolation to that using Figure 12.10-2(b) with a bi-linear interpolation, and demonstrates the conservative values at intermediate levels by using Figure 12.10-2(a). The same comparison described above and illustrated in C12.10-8 was computed for all seismic force-resisting systems in Table 12.2-1 for a five-story building and is illustrated in Figure C12.10-9. The results show generally conservative values, particularly for systems with a mode shape factor, z_s , less than 1, such as buckling restrained braced frames and moment-resisting frames.

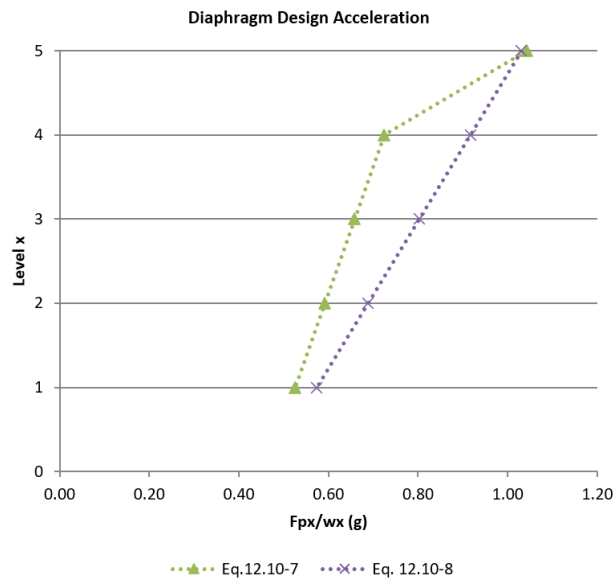


Figure C12.10-8. Comparison of design acceleration coefficient, C_{px} , from Equation 12.10-7 with Figure 12.10-2(b) distribution to Equation 12.10-8 with Figure 12.10-2(a) distribution for a five-story special reinforced concrete shear wall building

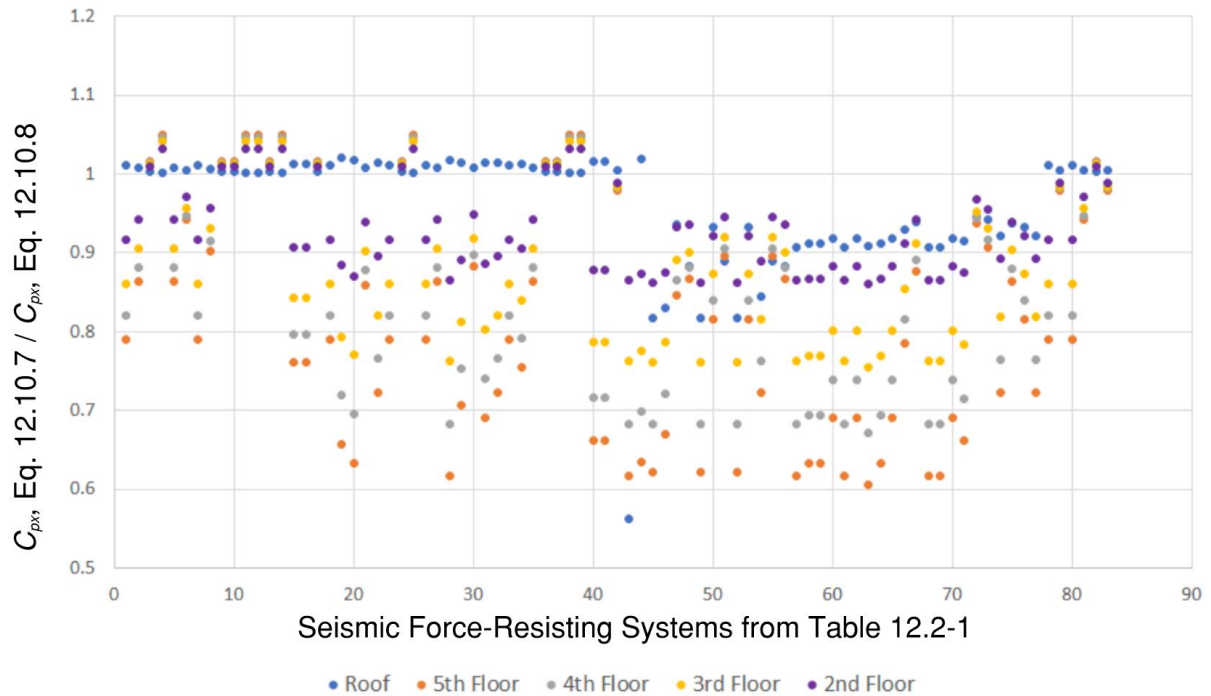


Figure C12.10-9. Comparison of design acceleration coefficient, C_{px} , from Equation 12.10-7 with Figure 12.10-2(b) distribution to Equation 12.10-8 with Figure 12.10-2(a) distribution for a five-story building using all seismic force-resisting systems in Table 12.2-1 shown on the horizontal axis in numerical order.

C12.10.2.3 Diaphragm Transfer Forces

All diaphragms are subject to inertial forces caused by the weight tributary to the diaphragm. Where the relative lateral stiffnesses of vertical seismic force-resisting elements vary from story to story, or the vertical seismic force-resisting elements have out-of-plane offsets, lateral forces in the vertical elements need to be transferred through the diaphragms as part of the load path between vertical elements above and below the diaphragm. These transfer forces are in addition to the inertial forces and can at times be quite large.

For structures that have a horizontal structural irregularity of Type 4 in Table 12.3-1, the magnitude of the transfer forces is largely dependent on the overstrength in the offset vertical elements of the seismic force-resisting system. Therefore, the transfer force caused by the offset is required to be amplified by the overstrength factor, Ω_0 , of the seismic force-resisting system. The amplified transfer force is to be added to the inertial force for the design of this portion of the diaphragm.

Transfer forces can develop in many other diaphragms, even within regular buildings; the design of diaphragms with such transfer forces can be for the sum of the transfer forces, unamplified, and the inertial forces.

C12.10.2.4 Collectors - Seismic Design Categories C through F

In this section the seismic design force for collectors and their connection to vertical elements of the seismic force-resisting system is differentiated by using an amplification of either R_s or 1.5 on the diaphragm design forces. This recognizes that the assigned R_s factor and resulting diaphragm seismic performance may be based on diaphragm system ductility only, diaphragm system overstrength only, or a mix of ductility and overstrength. In order to ensure that the collectors will not be the weak link, it is necessary to amplify the diaphragm design forces by multiplying by R_s . This reverses the reduction in diaphragm forces that dividing by R_s per equation 12.10-4 provides and reverts back to near elastic diaphragm force levels for purposes of collector design. Where applicable, the minimum amplifier of 1.5 differentiates an already near elastic collector force from the diaphragm forces to provide a margin of safety against collector failure. Where this procedure is followed, the collector should not become a weak link that would limit the seismic capacity of the diaphragm to less than would be estimated for a design earthquake. This amplified force level is also used for splices within the collectors. The intent of this requirement, originally introduced following the 1994 Northridge Earthquake, is to increase collector strength to help ensure that collectors will not be weak links in the seismic force-resisting system to the extent that premature failure results. This is a smaller multiplier than the amplification by Ω_0 that has been used in the past, but it is justified because the diaphragm forces are more accurately determined by Equation (12.10-4).

For collector elements of diaphragms that carry transfer forces caused by out-of-plane offsets of the vertical elements of the seismic force-resisting system, transfer forces, that are already amplified by Ω_0 , are at a near-elastic level and need not further amplified.

Some seismic force-resisting systems, such as braced frames and moment frames, have a fairly well-defined lateral strength corresponding to a well-defined yield mechanism. When collectors deliver seismic forces to such systems, it is not sensible to have to design the collectors for forces higher than those corresponding to the lateral strength of the supporting elements in the story below. This is why the cap on collector design forces is included. The lateral strength of a braced frame or moment frame may be calculated using the same methods as are used for determining whether a weak-story irregularity is present (Table 12.3-2). It should be noted that only the moment frames or braced frames below the collector are to be considered in calculating the upper-bound collector design force. The shear strength of the gravity columns and the lateral strength of the frames above are not included.

The design forces in diaphragms that deliver forces to collectors can also be limited by the maximum forces that can be generated in those collectors by the moment frames or braced frames below.

C12.10.2.5 Diaphragm Design Force Reduction Factor

Although analytical and shake table studies indicate higher diaphragm accelerations than have traditionally been used in diaphragm design, many commonly used diaphragm systems, including diaphragms designed under a number of US building codes and editions, have a history of excellent earthquake performance. With limited exceptions, diaphragms have not been reported to have performed below the life-safety intent of building code seismic design provisions in earthquakes.

Based on this history, it was felt that, for many diaphragm systems, no broad revision was required to the balance between demand and capacity used for design of diaphragms under recent ASCE 7 provisions. In view of this observation, it was recognized that the analytical studies and diaphragm testing from which the higher accelerations and design forces were being estimated used diaphragms that were elastic or near-elastic in their response. Commonly used diaphragm systems are recognized to have a wide range of overstrength and inelastic displacement capacity (ductility). It was recognized that the effect of the varying diaphragm systems on seismic demand required evaluation and incorporation into the diaphragm design forces. Equation (12.10-4) incorporates the diaphragm overstrength and inelastic displacement capacity through the use of the diaphragm force reduction factor, R_s . This factor is most directly based on the global ductility capacity of the diaphragm system; however, the derivation of the global ductility capacity inherently also captures the effect of diaphragm overstrength.

For diaphragm systems with inelastic deformation capacity sufficient to permit inelastic response under the design earthquake, the diaphragm design force reduction factor, R_s , is typically greater than 1.0, so that the design force demand, F_{px} , is reduced relative to the force demand for a diaphragm that remains linear-elastic under the design earthquake. For diaphragm systems that do not have sufficient inelastic deformation capacity, R_s should be less than 1.0, or even 0.7, so that linear-elastic force-deformation response can be expected under the risk-targeted maximum considered earthquake (MCE_R).

Diaphragms with R_s greater than 1.0 shall have the following characteristics: (1) a well-defined, specified yield mechanism; (2) global ductility capacity for the specified yield mechanism, which exceeds anticipated ductility demand for the risk-targeted Maximum Considered Earthquake (MCE_R); and (3) sufficient local ductility capacity to provide the intended global ductility capacity, considering that the specified yield mechanism may require concentrated local inelastic deformations to occur. The following discussion addresses these characteristics and the development of R_s -factors in detail.

A diaphragm system with R_s greater than 1.0 should have a specified, well-defined yield mechanism, for which both the global strength and the global deformation capacity can be estimated. For many diaphragm systems, a shear-yield mechanism may be appropriate, whereas for other diaphragm systems, a flexural-yield mechanism may be appropriate. This is recognized in the Table 12.10-1 assignment of a separate R_s value for cast-in-place concrete diaphragms with flexural mechanisms.

Figure C12.10-8(a) shows schematically the force-deformation ($F_{dia} - \Delta_{dia}$) response of a diaphragm with significant inelastic deformation capacity. The figure illustrates the response of a diaphragm system, such as a wood diaphragm or a steel deck diaphragm, which is not expected to exhibit a distinct yield point, so that an effective yield point (F_{Y-eff} and Δ_{Y-eff}) needs to be defined. For wood

diaphragms and steel deck diaphragms, the figure illustrates one way to define the effective yield point. The stiffness of a test specimen is defined by the secant stiffness through a point corresponding to 40% of the peak strength, F_{peak} . The effective yield point (F_{Y-eff} and Δ_{Y-eff}) for a diaphragm is defined by the secant stiffness through $0.4F_{peak}$ and the nominal diaphragm strength reduced by a strength reduction factor, ϕF_n , as shown in the figure. The $F_{dia} - \Delta_{dia}$ response is then idealized with a bilinear model, using the effective yield point (F_{Y-eff} and Δ_{Y-eff}) and F_{peak} and the corresponding deformation, Δ_{peak} , as shown in the figure.

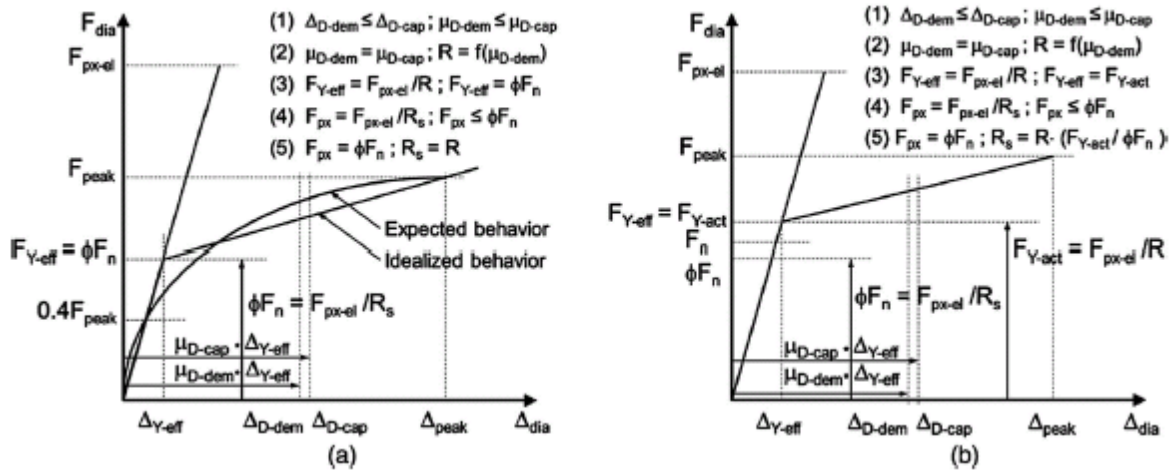


Figure C12.10-8. Diaphragm Inelastic response models for (a) a diaphragm system that is not expected to exhibit a distinct yield point, and (b) a diaphragm system that does exhibit a distinct yield point.

Figure C12.10-8(b) shows schematically the force–deformation ($F_{dia} - \Delta_{dia}$) response of a diaphragm with significant inelastic deformation capacity, which is expected to have nearly linear $F_{dia} - \Delta_{dia}$ response up to a distinct yield point, such as a cast-in-place reinforced concrete diaphragm. For this type of diaphragm system, the effective yield point can be taken as the actual yield point ($F_{Y-actual}$ and $\Delta_{Y-actual}$) of the diaphragm (accounting for diaphragm material overstrength and not including a strength reduction factor, ϕ).

The global (or system) deformation capacity of a diaphragm system, Δ_{cap} , should be estimated from analyses of test data. The force–deformation ($F_{dia} - \Delta_{dia}$) response shown schematically in Figure C12.10-8(a, b) is the global force–deformation behavior.

In some cases, tests directly provide the global deformation capacity, but more often, tests provide only the local response, including the strength and deformation capacity, of diaphragm components

and connections. When tests provide only the local deformation capacity, analyses of typical diaphragms should be made to estimate the global deformation capacity of these diaphragms. These analyses should consider (1) the specified yield mechanism, (2) the local force–deformation response data from tests, (3) the typical distributions of design strength and internal force demands across the diaphragm, and (4) any other factors that may require concentrated local inelastic deformation to occur when the intended yield mechanism forms.

After the global force–deformation ($F_{\text{dia}} - \Delta_{\text{dia}}$) response of a diaphragm has been estimated, the global deformation capacity, Δ_{cap} , can be determined. In Figure C12.10-8(a), for example, Δ_{cap} can be taken as Δ_{peak} , which is the deformation corresponding to the strength, F_{peak} . For some diaphragm systems, it may be acceptable to take the deformation corresponding to 80% of F_{peak} (i.e., post-peak) as Δ_{cap} .

Only a selected portion of the deformation capacity of a diaphragm, Δ_{cap} , should be used under the design earthquake, in recognition of two major concerns: (1) the diaphragm must perform adequately under the MCE_R, which has a design spectrum 50% more intense than the design earthquake design spectrum; and (2) significant inelastic deformation under the design earthquake may result in undesirable damage to the diaphragm. As a rough estimate, the diaphragm deformation capacity under the design earthquake, $\Delta_{D\text{-cap}}$, should be limited to approximately one-half to two-thirds of the deformation capacity, Δ_{cap} .

To develop the diaphragm force reduction factor, R_s , the diaphragm global deformation capacity should be expressed as a global ductility capacity, μ_{cap} , which equals the deformation capacity, Δ_{cap} , divided by the effective yield deformation, $\Delta_{Y\text{-eff}}$. The corresponding diaphragm design ductility capacity, $\mu_{D\text{-cap}}$, equals $\Delta_{D\text{-cap}} / \Delta_{Y\text{-eff}}$.

From the diaphragm global deformation capacity and corresponding ductility capacity, an appropriate R_s factor can be estimated. Use of the estimated R_s factor in design should result in diaphragm ductility demands that do not exceed the ductility capacity that was used to estimate R_s . The force reduction factor is ideally derived from system-specific studies. Where such studies are unavailable, however, some guidance on the conversion from global ductility to force reduction is available from past studies.

Expressions that provide the force reduction factor, R , for the seismic force-resisting system of a building corresponding to an expected ductility demand, μ_{dem} , have been proposed by numerous research teams. Numerous factors, including vibration period, inherent damping, deformation hardening (stiffness after the effective yield point), and hysteretic energy dissipation under cyclic

loading, have been considered in developing these expressions. Two such expressions, which are based on elastoplastic force–deformation response under cyclic loading (Newmark and Hall 1982), are $R = (2\mu_{\text{dem}} - 1)^{0.5}$, applicable to short-period systems, and $R = \mu_{\text{dem}}$, applicable to systems with longer periods. The first function, known as the equal energy rule, gives a smaller value of R for a given value of μ_{dem} ; the second function, known as the equal displacement rule, is also widely used.

Figure C12.10-8(a, b) summarize an approach to estimating R_s as follows:

1. For the selected value of R_s , the diaphragm deformation demand under the design earthquake, $\Delta_{D\text{-dem}}$, should not exceed the diaphragm design deformation capacity, $\Delta_{D\text{-cap}}$. This design constraint, expressed in terms of diaphragm ductility, requires that the diaphragm ductility demand under the design earthquake, $\mu_{D\text{-dem}}$, not exceed the diaphragm design ductility capacity, $\mu_{D\text{-cap}}$.
2. The largest value of R that can be justified for a given diaphragm design deformation capacity is obtained by setting the ductility demand, $\mu_{D\text{-dem}}$, equal to the design ductility capacity, $\mu_{D\text{-cap}}$, and determining R from a function that provides R for a given μ_{dem} . For example, if $\mu_{D\text{-cap}} = 2.5$, then $\mu_{D\text{-dem}}$ is set equal to 2.5, and the corresponding $R = 2$ from the equal energy rule or $R = 2.5$ from the equal displacement rule.
3. R from step (2) is the ratio of the force demand for a linear-elastic diaphragm, $F_{px\text{-el}}$, to the effective yield strength of the diaphragm, $F_{Y\text{-eff}}$. For a diaphragm system that is not expected to exhibit a distinct yield point (Figure C12.10-8a), $F_{Y\text{-eff}}$ equals the factored nominal diaphragm strength, ϕF_n . For a diaphragm system that is expected to exhibit a distinct yield point (Figure C12.10-8b), $F_{Y\text{-eff}}$ equals the actual yield strength, $F_{Y\text{-actual}}$, accounting for diaphragm material overstrength and not including the strength reduction factor, ϕ .
4. R_s , however, is defined as the ratio of the force demand for a linear-elastic diaphragm, $F_{px\text{-el}}$, to the design force demand, F_{px} . The diaphragm must be designed such that F_{px} is less than or equal to the factored nominal diaphragm strength, ϕF_n .
5. For a diaphragm system without a distinct yield point (Figure C12.10-8(a)) that has the minimum strength $F_{px} = \phi F_n$, R_s equals R from step (2). For a diaphragm system with a

distinct yield point (Figure C12.10-8(b)), which has the minimum strength $F_{px} = \phi F_n$, R_s equals R from step 2 multiplied by the ratio $F_{y-eff} / \phi F_n$.

Diaphragm force reduction factors, R_s , have been developed for some commonly used diaphragm systems. The derivation of factors for each of these systems is explained in detail in the following commentary sections. For each, the specific design standard considered in the development of the R_s factor is specified. The resulting R_s factors are specifically tied to the design and detailing requirements of the noted standard, because these play a significant role in setting the ductility and overstrength of the diaphragm system. For this reason, the applicability of the R_s factor to diaphragms designed using other standards must be specifically considered and justified.

Cast-in-Place Concrete Diaphragms. The R_s values in Table 12.10-1 address cast-in-place concrete diaphragms designed in accordance with ACI 318.

Intended Mechanism. Flexural yielding is the intended yield mechanism for a reinforced concrete diaphragm. Where this can be achieved, designation as a flexure-controlled diaphragm and use of the corresponding R_s factor in Table 12.10-1 is appropriate. There are many circumstances, however, where the development of a well-defined yielding mechanism is not possible because of diaphragm geometry (aspect ratio or complex diaphragm configuration), in which case, designation as a shear-controlled diaphragm and use of the lower R_s factor is required.

Derivation of Diaphragm Force Reduction Factor. Test results for reinforced concrete diaphragms are not available in the literature. Test results for shear walls under cyclic lateral loading were considered. The critical regions of shear wall test specimens usually have high levels of shear force, moment, and flexural deformation demands; high levels of shear force are known to degrade the flexural ductility capacity. The flexural ductility capacity of shear wall test specimens under cyclic lateral loading was used to estimate the flexural ductility capacity of reinforced concrete diaphragms, using the previously described method based on Newmark and Hall (1982).

Based on shear wall test results, the estimated global flexural ductility capacity of a reinforced concrete diaphragm is 3, based on the actual yield displacement, $\Delta_{y-actual}$, of the test specimens. The design ductility capacity, μ_{D-cap} , is taken as 2/3 of the ductility capacity; the design ductility capacity (μ_{D-cap}) is 2.

Setting the ductility demand, ϕ_{dem} , equal to the design ductility capacity, μ_{D-cap} , and using the equal energy rule, the force reduction factor $R = (2\mu_{dem} - 1)^{0.5} = 1.73$.

R_s equals R multiplied by the ratio $F_{Y\text{-eff}} / \phi F_n$. $F_{Y\text{-eff}}$ is taken equal to $F_{Y\text{-actual}}$, which is assumed to be $1.1F_n$, and ϕ equals 0.9. Therefore, $R_s = 2.11$, which is rounded to 2.

Because of the geometric characteristics of a building or other factors, such as minimum reinforcement requirements, it is not possible to design some reinforced concrete diaphragms to yield in flexure. Such diaphragms are termed “shear controlled” to indicate that they are expected to yield in shear. Shear-controlled reinforced concrete diaphragms should be designed to remain essentially elastic under the design earthquake, with their available global ductility held in reserve for safety under the MCE_R .

Based on the following considerations, R_s is specified as 1.5 for shear-controlled reinforced concrete diaphragms. Reinforced concrete diaphragms have performed well in past earthquakes. ACI-318 specifies ϕ of 0.75 or 0.6 for diaphragm shear strength and limits the concrete contribution to the shear strength to only $2(f'_c)^{0.5}$. In addition, reinforced concrete floor slabs often have gravity load reinforcement that is not considered in determining the diaphragm shear strength. Therefore, shear-controlled reinforced concrete diaphragms are expected to have significant overstrength. The ratio $F_{Y\text{-eff}} / \phi F_n$ for a reinforced concrete diaphragm, where $F_{Y\text{-eff}}$ is taken equal to $F_{Y\text{-actual}}$, is expected to exceed 1.5, which is the rationale for $R_s = 1.5$, even though μ_{dem} is assumed to be 1 for the design earthquake.

Precast Concrete Diaphragms. The R_s values in Table 12.10-1 address precast concrete diaphragms designed in accordance with ACI 318.

Derivation of Diaphragm Force Reduction Factors. The diaphragm force reduction factors, R_s , in Table 12.10-1 for precast concrete diaphragms were established based on the results of analytical earthquake simulation studies conducted within a multi-university project: Diaphragm Seismic Design Methodology (DSDM) for Precast Concrete Diaphragms (Fleischman et al. 2013). In this research effort, diaphragm design force levels have been aligned with the diaphragm deformation capacities specifically for precast concrete diaphragms. Three different design options were proposed according to different design performance targets (Table C12.10-1). The relationships between diaphragm design force levels and diaphragm local/global ductility demands have been established in the DSDM research project. These relationships have been used to derive the R_s for precast concrete diaphragms in Table 12.10-1.

Table C12.10-1. Diaphragm Design Performance Targets.

Options	Flexure		Shear
	DE	MCER	DE and MCER

Elastic design objective	Elastic	Elastic	Elastic
Basic design objective	Elastic	Inelastic	Elastic
Reduced design objective	Inelastic	Inelastic	Elastic

Notes: DE = design earthquake; MCE_R = risk-targeted maximum considered earthquake.

Diaphragm $R_{dia}-\mu_{global}-\mu_{local}$ Relationships. Extensive analytical studies have been performed (Fleischman et al. 2013) to develop the relationship of $R_{dia}-\mu_{global}-\mu_{local}$. R_{dia} is the diaphragm force reduction factor (similar to the R_s in Table 12.10-1) measured from the required elastic diaphragm design force at MCE_R level. μ_{global} is the diaphragm global ductility demand, and μ_{local} is the diaphragm local connector ductility demand measured at Maximum Considered Earthquake (MCE) level. Figure C12.10-9 shows the $\mu_{global}-\mu_{local}$ and $R_{dia}-\mu_{global}$ analytical results for different diaphragm aspect ratios (AR) and proposed linear Equations derived from the data.

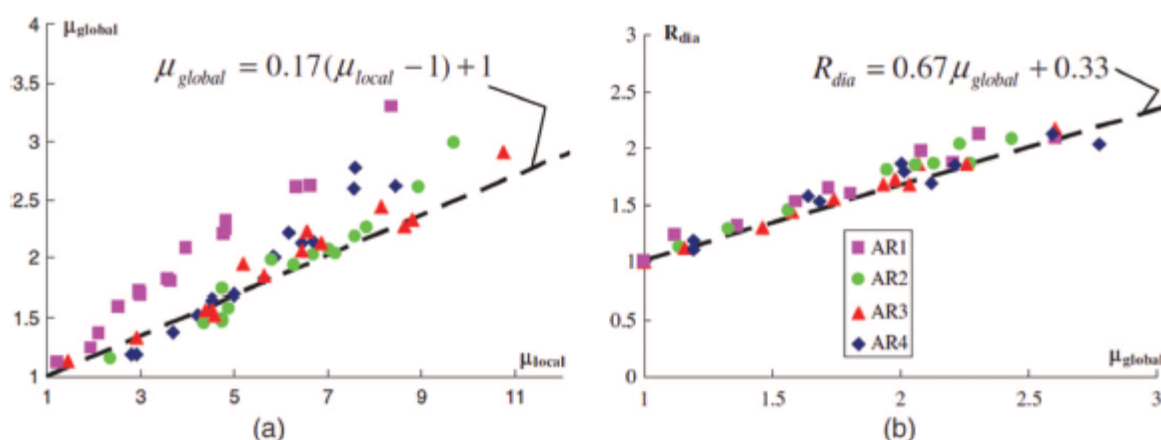


Figure C12.10-9. Relationships: (a) $\mu_{global}-\mu_{local}$; (b) $R_{dia}-\mu_{global}$.

Note: AR = aspect ratio.

Diaphragm Force Reduction Factor, R_s . Using the Equations in Figure C12.10-9, R_s can be calculated for different diaphragm design options, provided that the diaphragm local reinforcement ductility capacity is known. In the DSDM research, precast diaphragm connectors have been extensively tested (Fleischman et al. 2013) and have been qualified into three categories: high-deformability elements, moderate-deformability elements, and low-deformability elements, which are required as a minimum for designs using the reduced design objective, the basic design objective, and the elastic design objective, respectively. The local deformation and ductility capacities for diaphragm connector categories are shown in Table C12.10-2. Considering that the proposed

diaphragm design force level targets elastic diaphragm response at the design earthquake, which is equivalent to design using the basic design objective, where $\mu_{\text{local}} = 3.5$ at MCE_R (see Table C12.10-2), the available diaphragm global ductility capacity has to be reduced from Figure C12.10-9(a), acknowledging more severe demands in the MCE_R :

$$\mu_{\text{global,red}} = 0.17(\mu_{\text{local}} - 3.5) + 1 \quad (\text{C12.10-1})$$

Table C12.10-2. Diaphragm Force Reduction Factors, R_s .

Options	Diaphragm Connector Category	δ_{local} (in.)	μ_{local}	μ_{global}	$\mu_{\text{global,red}}$	R_s
Elastic design objective	Low-deformability elements	0.06	1.0	1.0	0.58	0.7
Basic design objective	Moderate-deformability elements	0.2	3.5	1.4	1.0	1.0
Reduced design objective	High-deformability elements	0.4	7.0	2.0	1.6	1.4

Accordingly, the R_s -factor can be modified from Figure C12.10-9(b) (see Table C12.10-2):

$$R_s = 0.67\mu_{\text{global,red}} + 0.33 \quad (\text{C12.10-2})$$

Wood-Sheathed Diaphragms. The R_s values given in Table 12.10-1 are for wood-sheathed diaphragms designed in accordance with AWC (2014) *Special Design Provisions for Wind and Seismic*.

Intended Mechanism. Wood-sheathed diaphragms are shear-controlled, with design strength determined in accordance with AWC (2014) and the shear behavior based on the sheathing-to-framing connections. Wood diaphragm chord members are unlikely to form flexural mechanisms (ductile or otherwise) because of the overstrength inherent in axially loaded members designed in accordance with applicable standards.

Derivation of Diaphragm Design Force Reduction Factor. An R_s factor of 3 is assigned in Table 12.10-1, based on diaphragm test data (APA 1966, 2000; DFPA 1954, 1963) and analytical studies. The available testing includes diaphragm spans (loaded as simple-span beams) ranging from 24 to 48 ft (7.3 to 14.6 m), aspect ratios ranging from 1 to 3.3, and diaphragm construction covering a range of construction types, including blocked and unblocked construction and regular and high-load diaphragms. The loading was applied with a series of point loads at varying spacing; however, the loading was reasonably close to uniform. Whereas available diaphragm testing was monotonic, based on shear wall loading protocol studies (Gatto and Uang 2002), it is believed that the

monotonic load–deflection behavior is reasonably representative of the cyclic load–deflection envelope, suggesting that it is appropriate to use monotonic load–deflection behavior in the estimation of overstrength, ductility, and displacement capacity.

Analytical studies using nonlinear response history analysis evaluated the relationship between global ductility and diaphragm force reduction factor for a model wood building. The analysis identified the resulting diaphragm force reduction factor as ranging from just below 3 to significantly in excess of 5. A force reduction factor of 3 was selected so that diaphragm design force levels would generally not be less than those determined in accordance with provisions of Section 12.10.1.

The calibration approach for selection of R_s of 3 was considered appropriate to limit conditions where diaphragm force levels would drop below those determined in accordance with Section 12.10.1. This was due in part to historical experience of good diaphragm performance across a range of wood diaphragm types, even though test data showed varying levels of ductility and deformation capacity. Tests of nailed wood diaphragms showed significant but varying levels of overstrength. It is recognized that even further variation of overstrength will result from:

- presence of floor coverings or toppings and their attachment or bond to diaphragm sheathing,
- presence of wall to floor framing nailing through diaphragm sheathing, and
- presence of adhesives in combination with required sheathing nailing (commonly used to mitigate floor vibration, increase floor stiffness for gravity loading, and reduce the potential for squeaking).

These sources of overstrength are not considered to be detrimental to overall diaphragm performance.

Cross-laminated Timber Diaphragms. The R_s value given in Table 12.10-1 is for cross-laminated timber diaphragms designed in accordance with *AWC Special Design Provisions for Wind and Seismic* (SDPWS) (AWC, 2021).

Intended Mechanism. Cross-laminated timber diaphragms are shear-controlled, with design strength determined in accordance with SDPWS and the shear behavior based on the panel-to-panel and panel-to-diaphragm boundary element connections. CLT diaphragm chord members are unlikely to form flexural mechanisms (ductile or otherwise) because the overstrength required in the SDPWS provisions promotes a yielding mechanism in the panel-to-panel and panel-to-boundary element connections.

Derivation of Diaphragm Design Force Reduction Factor. An R_s factor of 2.5 is assigned in Table 12.10-1, based on the analyses of full-scale diaphragm test data (Kode et al., 2021; Line et al., 2022; Bearisto et al. 2022; Bhardwaj et al., 2024a, 2024b). The diaphragm dimensions ranged from 15-24 ft. in each direction and the testing configurations included simply supported and cantilever

configurations. CLT panels were either simply supported or two-span continuous and aspect ratios of the panels included 2:1, 3:1, and 4:1. Panel-to-panel connections included steel, plywood, and laminated veneer lumber (LVL) splines with fasteners conforming to SDPWS requirements for the lateral design value to be controlled by Mode III_s or Mode IV fastener yielding. Full-scale tests demonstrated significant overstrength, R_d , ranging from 1.67 to 6.75. This variation of overstrength is to be expected and further variation can result from the presence of floor toppings, alternative load paths, and simplification in the design and/or construction process (e.g. reduced number of fastening schedules.)

Full-scale test data analyzed using the Component Ductility Method, referenced in Resource Paper 7 of NEHRP 2020 provisions (FEMA, 2020) and based on ATC 19 (ATC, 1995), resulted in the diaphragm force reduction factor ranging from 2.48 to 7.86 with an average value of 5.11. The proposed force reduction factor of 2.5 closely aligns with lower bound values from cyclic full-scale diaphragm test data.

Bare Steel Deck Diaphragms. The R_s values in Table 12.10-1 address bare (untopped) steel deck diaphragms designed in accordance with AISI S400 (2020b). These include diaphragms meeting requirements for special seismic detailing, and all other bare steel deck diaphragms.

Intended Mechanism. The ductility of bare steel deck diaphragms is largely driven by the performance of the deck profile and its interaction with sidelap and structural connections. It has been found for a specific class of wide-rib roof deck that, if the sidelap and structural connections have adequate ductility and deformation capacity, the full bare steel deck diaphragm can similarly develop productive levels of ductility with sufficient system-level deformation capacity (O'Brien et al. 2017, Schafer 2019). These findings formed the basis for prescriptive special seismic detailing requirements that are found in AISI S400 (2020b) as referenced from Section 14.1.5. AISI S400 (2020b) also provides performance-based criteria to establish that selected detailing associated with a particular bare steel deck diaphragm (e.g., new profile or new connector) meets the same performance objectives as the prescriptive system and thus is deemed to provide an intended ductile mechanism. Other bare steel deck diaphragms have fasteners and system behavior that are less ductile. As a result, the R_s factor is smaller, resulting in design for near-elastic level forces.

Derivation of Diaphragm Force Reduction Factor. The derivation of the bare steel deck diaphragm force reduction factor, R_s , is summarized in Appendix 1 of Schafer (2019). The ductility and deformation capacity of sidelap and structural connections employed in bare steel deck diaphragms are established by evaluation of cyclic shear testing (NBM 2017, 2018; Schafer 2019). The ductility of bare steel deck diaphragms is preliminarily established by assembly and evaluation of cyclic cantilever diaphragm tests (O'Brien et al. 2017). The impact of the connector and cantilever diaphragm tests on full building performance is assessed in a 3D building model, as detailed in Schafer (2019). The model shows that only bare steel deck diaphragms with connections that have sufficient ductility and deformation capacity provide adequate inelastic diaphragm performance, leading to special seismic detailing requirements. For the subset of cyclically tested diaphragms that meet the special seismic detailing requirements, the tested subsystem ductility and system overstrength are established (Schafer 2019). To establish the diaphragm system ductility, an

additional correction is provided for the reduction in ductility of a roof that experiences varying shear across its width, compared with a cantilever diaphragm test which is under constant shear (O'Brien et al. 2017, Schafer 2019). From the system ductility and overstrength, the diaphragm force reduction factor R_s was developed based on μ - R relations using the method documented in ATC-19 (1995).

Concrete-Filled Steel Deck Diaphragms. The R_s values in Table 12.10-1 address concrete-filled steel deck diaphragms designed in accordance with AISC 341 (2022).

Intended Mechanism. Concrete-filled steel deck diaphragms exhibit two types of failure limit states: diagonal tension cracking in the field of the diaphragm, and failure of the shear transfer at the fasteners to the collectors. Both failure modes are considered shear-controlled limit states, and thus concrete-filled steel deck diaphragms only have a value listed for the shear-controlled category in Table 12.10-1. Previous testing by Porter and Easterling (1994) and Avellaneda et al. (2019) has demonstrated ductility and overstrength for concrete-filled steel deck diaphragms experiencing both limit states. These tests also covered a range of variables, including concrete specific weight (normal weight and lightweight), reinforcing steel (unreinforced and with reinforcing steel), concrete strength [2,500 to 6,000 psi (17,000 to 41,000 kPa)], deck height [1.5 to 3.0 in. (38 to 76 mm)], total thickness [4 to 7.5 in. (100 to 190 mm)], and perimeter fastener type (headed shear studs and arc spot welds). The most common limit state in testing was diagonal tension cracking of the concrete, but specimens experiencing other limit states, such as failure of perimeter fasteners or debonding between the deck and concrete, had similar or better ductility. Because this set of tests considered a wide range of variables, the resulting R_s factor is associated with concrete-filled steel deck diaphragms designed according to AISC 341 (2022) without any special detailing. It is anticipated that ongoing and future work may lead to a new category of concrete-filled steel deck diaphragm with special detailing and larger R_s value.

Derivation of Diaphragm Force Reduction Factor. The derivation of R_s for concrete-filled steel deck diaphragms is described in Eatherton et al. (2020), O'Brien et al. (2017), and Wei et al. (2020). A two-stage procedure was used to define the R_s factor: (1) using measured ductility and overstrength obtained from cantilever diaphragm tests to calculate a value of R_s , and (2) validating the R_s factor using a computational study.

The beginning of this section's commentary describes a method for calculating the diaphragm design force reduction factor, R_s , based on measured ductility and overstrength obtained from test data. That method is essentially the same as that described in ATC-19 (1995) and the approach used to derive R_s for cast-in-place concrete diaphragms. In this method, the R_s factor is the sum of two components; the first is associated with the expected overstrength of the diaphragm, and the second is a function of the ductility. Eatherton et al. (2020) and O'Brien et al. (2017) evaluate the ductility and overstrength of 13 cantilever tests on concrete-filled steel deck diaphragm specimens conducted by Easterling and Porter (1994). The average ductility of these specimens was 3.63, using a definition of ductility as the shear angle associated with 20% loss of strength compared to the peak, divided by the shear angle associated with yield. Eatherton et al. (2020) also propose a method for converting ductility from a cantilever diaphragm test into expected system-level ductility

for a full-diaphragm span. Using this conversion, the average system-level ductility was found to be 2.05. Eatherton et al. (2020) calculated values of the diaphragm force reduction factor, R_s , of between 2.00 and 3.35, depending on the building period and whether the resistance factor was included in the calculation of overstrength. Based on this study, a value of $R_s = 2.0$ was proposed. It is noted that more recent cantilever diaphragm tests by Avellaneda et al. (2019) on concrete-filled steel deck diaphragm specimens that are more representative of contemporary construction resulted in ductility and overstrength consistent with the tests by Easterling and Porter (1994) as analyzed in FEMA (2020).

To validate this proposed value of $R_s = 2.0$ for concrete-filled steel deck diaphragms, a computational study was conducted to evaluate the associated effect on collapse probability (Wei et al. 2020). Eight buildings with buckling restrained braced frames as the seismic force-resisting system were analyzed. The height of the building was varied with 1-, 4-, 8-, and 12-story buildings, and two variations were designed for each height: floor diaphragms designed using $R_s = 1.0$ and $R_s = 2.0$. A FEMA P-695 (2009b) type of study was conducted. Using the criteria in FEMA P-695, the collapse probability was found to be acceptable for the 1-, 4-, and 8-story buildings designed using $R_s = 2.0$ for the concrete-filled steel deck floor diaphragms. The 12-story building exceeded the limit for collapse probability by 16%, but changing the diaphragm design from $R_s = 2.0$ to $R_s = 1.0$ only reduced the collapse probability by 4%, and it had a negligible effect on median peak story drifts for all buildings. This study concluded that diaphragm design using $R_s = 2.0$ resulted in similar collapse performance and peak drift performance as $R_s = 1.0$ buildings and thus was acceptable. The study further noted that the vast majority of past FEMA P-695 studies have been conducted on 2D frames and that more research is required to define appropriate collapse performance criteria for 3D building models that consider diaphragm inelasticity.

Rod-Braced Diaphragms

The R_s values in Table 12.10-1 address rod-braced diaphragms designed in accordance with AISC 360 and AISI S100 as applicable. These include rod-braced diaphragms meeting requirements for special seismic detailing, and all other rod-braced diaphragms.

Intended Mechanism.

The expected behavior of rod-braced diaphragms with special seismic detailing is for the rod-braces to yield along the length of the unthreaded body of the rod prior to fracture of the rod in the connection region as enforced by the inequality in Section 14.1.11.4(b). Connection tests showed that the tensile yield strength of the rod was developed when the limit states of bearing in the web of the framing member (rafter beam), local buckling of the flange of the framing member (rafter beam), and shear of the hillside washer bearing tab, were prevented (Foroughi et al. 2020; Sinno 1993; Meek et al. 2023). Large-scale diaphragm subassembly tests showed that tensile yielding of the unthreaded portion of the rod provided a well-defined yield mechanism when the stability of the framing members (roof purlins) was maintained, and the hillside washers remained engaged in the hole of the framing member (Meek et al. 2023).

Derivation of Diaphragm Force Reduction Factor.

The derivation of the diaphragm force reduction factor for rod-braced diaphragms is described in Meek et al. (2023). The behavior of rod-braced diaphragms with special seismic detailing was confirmed through connection tests and subassembly tests. Computational models were then calibrated against the test data and used to represent full rod-braced diaphragm spans. Six models were created with varied rod sizes including designs where all the rods are the same diameter, and others where the rods are proportioned according to calculated diaphragm shear. The model results were used to determine the ductility and overstrength of the diaphragms. The method described at the beginning of this section's commentary was used to calculate the diaphragm design force reduction factor, R_s as the product of one component associated with expected overstrength, and a second component associated with ductility. This is the same process used to define a diaphragm design force reduction factor for bare steel deck with special seismic detailing.

Rod-braced diaphragm systems without special detailing typically fail by framing member instability or disengagement of the hillside washer. This type of behavior is expected to produce minimal ductility and overstrength consistent with a value of $R_s=1.0$.

Similar to the requirement for the horizontal combination of seismic force-resisting systems, this requirement reflects the expectation that the diaphragm will undergo the same deformation with its behavior controlled by the least ductile portion of the diaphragm. Inelastic performance of diaphragm is independent from level to level, regardless of the diaphragm system at each level. Thus, there is no restriction on varying diaphragm systems or the R_s factor between levels.

At a given level, it may be possible for multiple diaphragm systems to perform independently. If it can be demonstrated that they can deflect independently in one or both orthogonal directions, they are permitted to be designed using the R_s assigned to the diaphragm system in that direction. This could result in diaphragm design for different R_s factors in each orthogonal direction. See Figure C12.10-10.

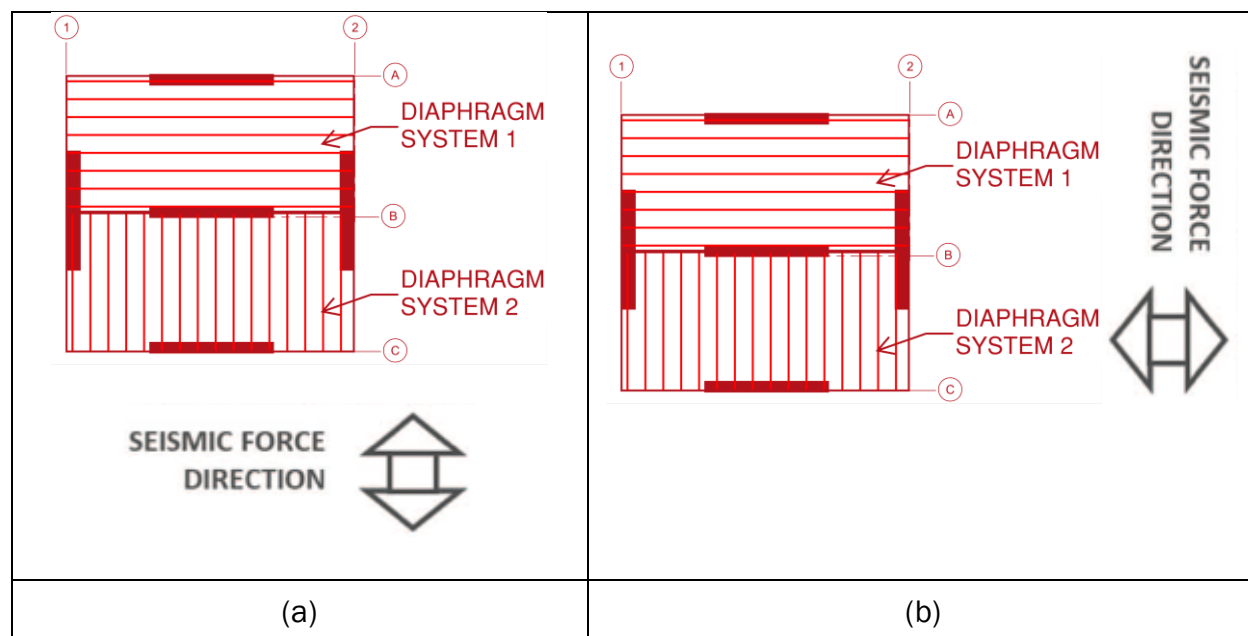


Figure C12.10-10

Assuming that Diaphragm Systems 1 and 2 are flexible diaphragms, Diaphragm System 1 is not independent of Diaphragm System 2 in Figure C12.10-10(a), because they cannot both deflect independently, when loaded in the specified direction. In Figure C12.10-10(b), the two systems are independent, since they can deflect independently, when loaded in the specified direction. There are various other configurations that result in either independent deflection or coupled behavior. For example, if Diaphragm Systems 1 and 2 are rigid, semirigid, or a combination of rigid and semirigid, they cannot deflect independently.

Engineering judgment should be used to interpret what constitutes independent diaphragm deflection. One example is a bare steel deck diaphragm with special seismic detailing that has a few concrete-topped housekeeping pads for mechanical units. A small percentage of roof area consisting of concrete-topped steel deck should not govern the overall performance of the diaphragm. Another example is a concrete topped steel deck diaphragm with a bare steel deck diaphragm balcony or canopy. The balcony or canopy is an appendage of the level's diaphragm and should not govern diaphragm behavior and thus independent R_s values may be used. In both of these cases, it is necessary to consider overall diaphragm performance to properly determine magnitude and distribution of forces.

C12.10.3 One-Story Diaphragm Design Method Provisions for Structures with Flexible Diaphragms and Rigid Vertical Elements

Section 12.10.3 introduces diaphragm seismic design provisions for one-story structures combining flexible diaphragms with rigid vertical elements. This approach is based on numerical studies conducted as part of the development of the 2015 (updated 2021) guideline document *Seismic Design of Rigid Wall-Flexible Diaphragm Buildings: An Alternate Procedure* (FEMA P-1026), supplemented by additional recent steel deck diaphragm research. The numerical studies and the

resulting seismic design methodology specifically recognized the dynamic response of rigid wall–flexible diaphragm structures as being dominated by dynamic response of and inelastic behavior in the diaphragm. These studies indicate that improved seismic performance can be obtained for this group of structures through use of this design methodology. While the most common occurrences of this structure type are the concrete tilt-up and masonry “big box” structures, the “rigid vertical element” terminology of this section recognizes a wider range of vertical elements with which this methodology is permitted to be used.

C12.10.3.1 Limitations

This section imposes a series of limitations intended to restrict use of the methodology to flexible diaphragm–rigid vertical element structures consistent with the FEMA P-1026 basis.

Item 1 requires that when these alternative provisions are used they be used in both orthogonal directions. Performance of diaphragms designed using a mix of different design provisions for each orthogonal direction has not been studied, so the resulting seismic performance is not known.

Item 2 limits use to wood structural panel or bare steel deck diaphragms, consistent with the recommendations of the FEMA P-1026 document.

Item 2 limits wood structural panel diaphragms to those designed in accordance with the AWC (2014) SDPWS standard. The Item 2 reference to SDPWS tables further limits use to wood structural panel sheathing fastened to wood framing members or fastened to wood nailers (e.g., wood nailers attached to steel open-web joists) with nailing as specified in the SDPWS diaphragm tables. This is intended to limit fastening to nailed diaphragms and the nail types and nail patterns specified in the table. The FEMA P-1026 wood numerical studies were conducted using hysteretic descriptions derived from testing of diaphragms using full-length common nails based on the SDPWS tables. The performance of wood diaphragms using alternative fasteners and boundary members (chords, collectors, ledgers) has not been studied.

Dynamic testing of SDPWS diaphragms using short nails [nails shorter than specified by ASTM F1667 (2017), but still meeting the minimum penetration requirements of the SDPWS tables] is not readily available. APA (1966, 2007) testing of diaphragms with short nails showed little change in strength or displacement ductility. Thus the use of short nails in diaphragm construction is not prohibited. The provisions of Section 12.10.3 are, however, very narrowly limited to the nails specified in the SDPWS table. Use of shorter nail shank lengths can be justified by the alternate methods of construction provisions of the International Building Code, Section 104.11 (ICC 2012). The provisions of Section 12.10.3 are not intended to carry over to other nail types or other fastener types (staples, screws, etc.).

Direct attachment of wood structural panel diaphragm sheathing to perimeter structural steel ledgers using power-actuated fasteners (PAFs) is believed to be very common in concrete and masonry wall buildings. Although approval of this type of fastener falls under the “alternative methods of construction” provisions of the International Building Code, it is appropriate to note that

the use of PAFs at the perimeter is of lesser concern when using the Section 12.10.3 diaphragm design method because amplified shear forces in the boundary zone reduce the inelastic deformation demands at the diaphragm perimeter. Therefore, wood diaphragm use of PAFs in combination with steel members at the diaphragm boundary is not prohibited. This is, however, very narrowly limited to PAFs at the diaphragm perimeter in the amplified shear zone and where the PAFs can be justified by the “alternative methods of construction” provisions of the International Building Code, Section 104.11.

For bare steel deck diaphragms, Item 2 references the recently developed steel deck design and detailing provisions of AISI S400 (2020b) and provisions of AISI S310 (2020a).

Item 3 prohibits use of this methodology where materials installed over the diaphragm would add significant diaphragm stiffness. This limitation is included because where such materials are used, the diaphragm period and therefore seismic forces would not be appropriately estimated by this section, and the diaphragm may no longer qualify as flexible. Prohibited materials include concrete and similar materials (e.g., vermiculite concrete, cellular concrete, gypsum concrete, etc.) that provide significant stiffness and impede diaphragm deflection. Other toppings of potential concern include rigid insulation board (if bonded to the sheathing) and spray foam, as the bonding caused by these types of materials can impede the fastener behavior that is fundamental to deflection of diaphragms. Available testing of polyisocyanurate board attached with screws indicates an initial increase in stiffness only at very low load levels, beyond which the board does not affect the strength or stiffness of the diaphragm; thus it is considered acceptable to use this system. The wording used for Item 3 is taken from Section 12.3.1.1 because of the very similar intent of both sections. Where isolated pads of concrete occur on the bare steel deck (such as for rooftop equipment), the engineer will need to judge whether the concrete pads reduce the flexibility of the diaphragm such that the diaphragm falls outside the intent of Section 12.10.3.

Item 4 prohibits use with horizontal irregularities including torsional and diaphragm discontinuity irregularities. These configurations generate forces in diaphragms outside those studied in the FEMA P-1026 numerical studies, and the force levels and amplified shear boundary zones may not be appropriate with these irregularities.

Re-entrant corners are specifically permitted in structures designed using these seismic design forces. Re-entrant corners are believed to be prevalent in the existing building stock, and so anticipated to be common in buildings to be constructed under these provisions in the future. Because these diaphragms are dominated by shear deformation, it is judged that reentrant corners will not influence building seismic performance in a manner that would make use of these provisions inappropriate. Vertical irregularities are not addressed, because use of the section is limited to one-story buildings.

Horizontal irregularity Table 12.3-1 triggers two design requirements for structures in Seismic Design Categories D, E, and F that have re-entrant corners. The first is the Section 12.3.3.5 requirement for increasing diaphragm collector design forces determined per Section 12.10.1; new Section 12.10.3.2.6 identifies this requirement as not being applicable because design per Section 12.10.3

requires use of an overstrength factor, and therefore the design forces fall under the exception to Section 12.3.3.5. The second references Table 12.6-1 limitations on analytical procedures; this requirement will not affect the one-story structures intended to fall under Section 12.10.3 provisions, because they will always be significantly below the 160 ft (49 m) height limit of Table 12.6-1.

Item 5 requires that rectangular buildings with interior vertical elements or buildings with nonrectangular plan configurations be broken down into a series of rectangular sections for purposes of diaphragm design. Buildings to which this methodology might be applied are often not completely rectangular in plan, and often combine both long and short diaphragm spans. This provision requires that each section of diaphragm be defined as spanning between boundaries consisting of either vertical elements or collectors, with each span referred to as a “diaphragm segment.” Figure C12.10-10(a) illustrates diaphragm segments for transverse seismic forces, with each segment supported on all sides by concrete or masonry walls. Figure C12.10-10b illustrates the same concept, with two building plans in which, for transverse design, the diaphragm segments span to a combination of walls and collectors. The Item 5 limitation would also prohibit application to nonrectangular diaphragm segments such as triangular, trapezoidal, or curved configurations; this is because of the more complex response of these configurations, and the difficulty in defining the effective diaphragm span and resulting diaphragm period (Figure C12.10-11).

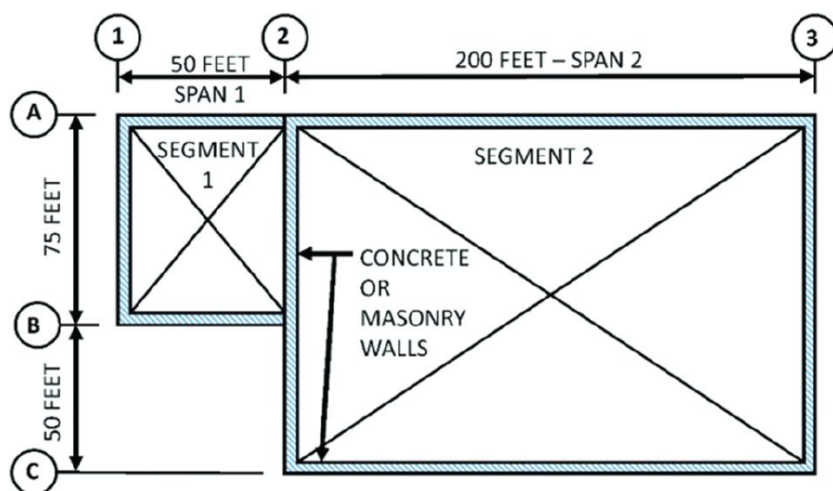


Figure C12.10-10a. Structure in plan view, divided into rectangular diaphragm segments for purposes of transverse seismic design. Note that identification of different segments will be required for longitudinal diaphragm forces.

Note: 1 ft = 305 mm.

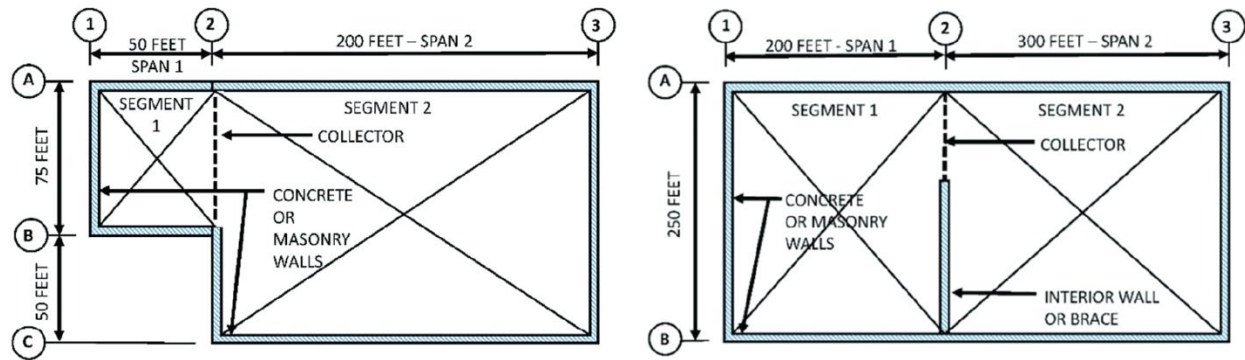


Figure C12.10-10b. Structures in plan view, divided into rectangular diaphragm segments for purposes of transverse seismic design. The Line 2 collector serves as a boundary between diaphragm segments for transverse direction loading. Note that identification of different segments will be required for longitudinal diaphragm forces in the building on the left.

Note: 1 ft = 305 mm.

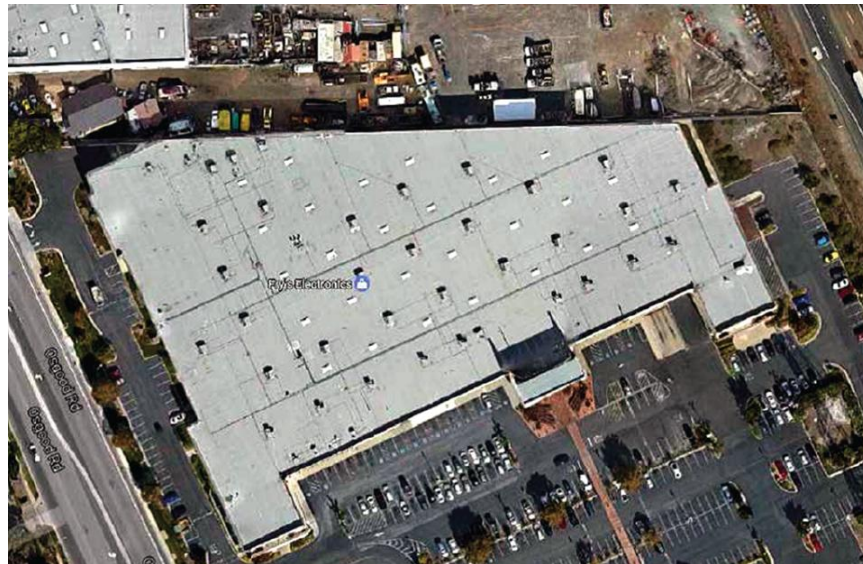


Figure C12.10-11. Example of a structure with nonrectangular diaphragm that is beyond the scope of the Section 12.10.3 provisions.

Source: Google Maps

Item 6 limits the vertical elements to systems that are inherently rigid for in-plane forces. This list functions as a definition of rigid vertical elements for purposes of Section 12.10.3. The list is in lieu of the FEMA P-1026 recommended minimum ratio of 3 between the fundamental period of the diaphragm and vertical elements. Limitation by system was identified to be the easiest and most direct way to address this criterion; see FEMA P-1026 for further discussion. The modifiers “ordinary,” “intermediate,” and “special” are not included for the vertical elements, with the intent that all types are sufficiently rigid and therefore permitted.

For Item 6, questions have arisen as to why steel and composite concentric braced frames are included in the scope and why buckling restrained braced frames (BRBFs) are not. The criteria established and studied during development of FEMA P-1026 required that the diaphragm calculated period be at least three times the period of the vertical elements. When considering wood structural panel and bare steel deck diaphragms, one-story concentric braced frames designed in accordance with the requirements of ASCE 7 and adopted material standards will very clearly categorically meet these criteria, and are therefore included in the scope of these provision. Because BRBFs designed in accordance with adopted standards tend to have longer periods (more in line with steel moment frames), it is believed that they will generally not meet the FEMA P-1026 criteria, and are therefore excluded from the scope of these provisions. The drafters of the Section 12.10.3 provisions believed it important to use item 6 to limit use to vertical elements to those where use of the provisions is perceived to be of benefit.

Item 7 serves as a reminder that the provisions of Section 12.10.3 address only design of the diaphragm, and it is not intended or permitted that these force levels also be used for the vertical elements of the seismic force-resisting system. Vertical elements are to be designed using the equivalent lateral force (ELF) procedures of Section 12.8, except when designed in accordance with the two-stage analysis procedure of Section 12.2.3.2.2.

C12.10.3.2 Design

Section 12.10.3.2.1 provides new Equations (12.10-16), (12.10-17a), and (12.10-17b), to be used for calculation of diaphragm design forces. These Equations incorporate the diaphragm approximate period, T_{diaph} . Where T_{diaph} is greater than T_s , this will permit C_s to be defined by the descending velocity-controlled portion of the response acceleration spectrum, thereby reducing diaphragm design forces. This is a distinct deviation from past design practices where seismic forces for diaphragm design were determined exclusively based on the approximate period and response modification factor of the vertical elements of the seismic force-resisting system. It is not necessary to set a lower bound on C_s , because the numerical studies for FEMA P-1026 investigated C_s values without a lower bound and found acceptable performance. Lower bounds for diaphragm design forces have been provided in the past primarily to guard against the higher diaphragm forces associated with higher-mode shapes of tall multistory buildings; however, the numerical studies for FEMA P-1026's single-story-building methodology saw limited participation of these higher modes and no reason to maintain this lower bound with these provisions.

Design Diaphragm Deflection Calculation Using SDPWS Engineered Design Methods. FEMA P-1026 provides design examples of a diaphragm with a 400 ft (12 m) span and 200 ft (60 m) width using common engineered design practice and SDPWS. FEMA P-1026, Chapter 3, provides a design example of the diaphragm using the non-system-based diaphragm design method (Section 12.10.1 diaphragm forces). FEMA P-1026, Chapters 5 and 6, provide a parallel design example using the new provisions of ASCE 7-28 Section 12.10.2. Using ASCE/SEI 7-22 Section 12.10.1 diaphragm design forces and C_d -diaph of 4 consistent with an intermediate precast shear wall, the estimated maximum diaphragm deflection is 29 in. (736 mm) [using the three-term Equation of SDPWS Section 4.2.2. Using Section 12.10.3 diaphragm design forces, the prescribed C_d -diaph of 3.0, and the

design assumptions used in the FEMA P-1026 examples, the estimated maximum diaphragm deflection is 19 in. (482 mm)]. Depending on calculation assumptions and calculation methods, it is anticipated that design engineers might calculate maximum diaphragm deflection as being anywhere between 10 and 19 in. (254 and 482 mm). This is a relative estimated displacement between the foundation and roof diaphragm at diaphragm midspan, which will be a maximum imposed drift on the vertical elements of the gravity system. The primary contributions to roof diaphragm deflection come from the shear deformation of the wood structural panel diaphragm (combined nail slip and panel shear deformation) and flexural deformation from tension and compression of the chord member.

Numerical Studies Using NLRHA. Numerical studies used as the basis for FEMA P-1026 provide data on analytical predictions of average peak diaphragm deflections. Diaphragm drift ratios published in Koliou et al. (2015a, b) are average peak ratios for the FEMA P-695 ground motion suite, scaled to $SDS = 1.0$. The published diaphragm drift ratios correspond to an average peak roof deflection of 7 in. (178 mm) for the Chapter 3 example of the 400 ft (12 m) span and 200 ft (60 m) wide diaphragm designed for non-system-based diaphragm design (Section 12.10.1) forces. The published diaphragm drift ratios correspond to an average peak roof deflection of 10 in. (254 mm) for a structure designed using a method close to but not exactly matching Section 12.10.3 (the design of this similar building model used a period that combined diaphragm and shear wall period, modestly increasing the period, lowering the design forces, and lowering diaphragm stiffness).

Revise Section C12.12 as follows:

C12.12 DRIFT AND DEFORMATION

There are multiple reasons for computing drift and limiting building deformations. These may be summarized as follows:

1. Ensuring that the Seismic-Force-Resisting System can prevent collapse (i.e., provide stability) by:
 - a. Limiting ductility demands on the elements of the seismic-force-resisting system (indirectly) such that the system maintains its lateral resistance.

Certain system requirements address inelastic-deformation demands directly, such as links in steel EBFs and buckling-restrained braces. Many systems, however, have no such safeguards and may rely on limiting building drift to limit inelastic-deformation demands. Even Steel Special Moment Frames, which have tested or prequalified connections with demonstrated rotation capacity, rely on designs limiting the rotation demand by meeting the drift limits of Table 12.12-1.

- b. Preventing a condition in which the inelastic system strength is inadequate to prevent the destabilizing P-Delta force at the inelastic lateral drift from causing a sway mechanism. P-Delta effects are addressed by Section 12.8.7.

2. Ensuring other systems are not compromised in a way that poses a risk to life safety considering the deformations and displacements they will experience as the Seismic-Force-Resisting System reaches its inelastic displacement. Specifically:
 - a. Limiting deformation demands on the gravity system. Excessive deformation of non-ductile gravity-load-resisting elements can lead to instability through local collapse. This is addressed in Seismic Design Category D and higher by Section 12.12.4.
 - b. Limiting displacement demands on non-structural elements. Excessive displacement could result in loss of support or function, resulting in a life-safety hazard. This is addressed by Chapter 13.
 - c. Preventing movements of adjacent structures or portions of structures from causing damaging contact (pounding). This is addressed by Section 12.2.2.
 - d. Preventing collapse of members (bridges) spanning between adjacent structures. This is addressed by Section 12.2.3.

These purposes were identified as far back as ATC-3, published in 1978. However, subsequent codes have begun to address these purposes separately from the drift limit table, which can be seen in the referenced sections and chapter above. This allows for the simplification of the table as is discussed in more detail below. There is, however, still work to be done, especially in establishing deformation limits for the full range of lateral and gravity systems. As used in the standard, *deflection* is the absolute lateral displacement of any point in a structure relative to its base, and *design story drift*, Δ , is the difference in deflection along the height of a story (i.e., the deflection of a floor relative to that of the floor below). The drift, Δ , is calculated according to the procedures of Section 12.8.6. (Sections 12.9.2 and 16.1 give procedures for calculating displacements for modal response spectrum and linear response history analysis procedures, respectively.)

Calculated story drifts generally include torsional contributions to deflection (i.e., additional deflection at locations of the center of rigidity at other than the center of mass caused by diaphragm rotation in the horizontal plane).

The deflections and design story drifts are calculated using the design earthquake ground motion (Sections 12.8.6.3 and 12.8.6.5). The resulting drifts are therefore underestimated for MCE_R ground motions.

The design base shear, V , used to calculate Δ is reduced by the response modification coefficient, R . Multiplying displacements by the deflection amplification factor, C_d , is intended to correct for this reduction and approximate inelastic drifts corresponding to the design response spectrum unreduced by R . However, it is recognized that use of values of C_d less than R underestimates deflections (Uang and Maarouf 1994) for some seismic force-resisting systems. Sections C12.8.6.2 and C12.9.1.4 deal with the appropriate base shear for computing

displacements. While the resulting displacements do not reflect the expected response to MCE_R ground motions, they are appropriate for use in evaluating the structure's compliance with the story drift limits given in Table 12.12-1.

The allowable story drifts, Δ_a , of Table 12.12-1 were adopted based on committee judgment in the 1994 NEHRP provisions and have subsequently been maintained on the same basis as subsequent Provisions Update Committees did not find specific reason to alter them.

The drift limits in Table 12.12-1 have been established without regard to economic considerations such as a comparison of present worth of future repairs with additional structural costs to limit drift. These are matters for building owners and designers to address.

Because the displacements induced in a structure include inelastic effects, structural damage as the result of a design-level earthquake is likely. For structures assigned to Risk Category I or II, the value of Δ_a is many times the typical drift associated with wind loads.

To provide better performance for structures assigned to Risk Category III or IV, the allowable story drift is reduced from that assigned to Risk Category I or II. However, the reduced limits are still greater than the damage thresholds for most nonstructural components. Therefore, though the performance of nonstructural components in structures assigned to Risk Category III or IV should be improved, there may be considerable damage from the design-level earthquake.

The allowable story drifts, Δ_a , for structures a maximum of four stories above the base are relaxed somewhat, provided that the interior walls, partitions, and ceilings have been designed to accommodate story drifts. These more relaxed drift limits require that interior walls, partitions, and ceilings be designed to accommodate story drifts to limit damage for these components, which are not otherwise required to accommodate the story drift per Chapter 13. Prior provisions that required exterior wall systems to accommodate the more relaxed story drift limits have been removed, since this is required by Section 13.5.3 regardless of the applicable drift limit.

The values of Δ_a in Table 12.12-1 apply to each story. For some structures, satisfying strength requirements may produce a system with adequate drift control. However, the design of moment-resisting frames and of tall, narrow shear walls or braced frames often is governed by drift considerations.

Previous versions of Table 12.12-1 included unique limits for masonry wall structures, and no drift limits at all for certain one-story structures. The unique limits on masonry walls were removed based on an examination of the deformation capacity using ASCE 41-23 of deformation-limited masonry walls designed in accordance with TMS 402-22. The committee also noted that several changes have been made to improve the ductility of masonry walls since 1998 when the unique limits on masonry walls were introduced to this code. The removal of the no-drift limit for certain one-story structures was based on research on one-story moment frames (Meimand, Moen and Schafer, 2018) suggesting a potential fracture-based collapse drift limit between 4% and 5%. This indicated a

need to cap drifts of these structures. This research also demonstrated that a 2.5% drift limit would provide code compliant performance of these structures even with heavy cladding and mezzanines. While the technical basis for the one remaining unique limit that changes the drift limit between four-story and five-story buildings is not well documented, the limit of 2.5% has been used for buildings under five stories for decades, as has the limit of 2% for buildings five or more stories. Absent compelling evidence of the inadequacy of these limits, they are maintained for continuity of practice.

Revise Section C12.13.9 as follows:

C12.13.9 Requirements for Foundations Subject to Seismically Induced Soil Displacement or Strength Loss

This section has been revised to include all forms of seismically induced permanent ground displacement and reduction in soil strength when subjected to the MCE_L and MCE_G peak ground accelerations, as defined in Section 11.8.3. This section complements the requirements of Section 11.8, which provides requirements for geotechnical investigations in areas with significant seismic ground motion hazard, with specific requirements for additional geotechnical information and recommendations for sites that have the potential to liquefy when subjected to the MCE_L ground motion.

The requirements of this section, along with the seismic requirements of this standard, are intended to result in structure foundation systems that satisfy the performance goals stated in Section 1.1 of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA 2009a) for structure sites that have been determined to have geologic and seismic hazards per Section 11.8. They require mitigation of significant slope instability, liquefaction, settlement, fault, and lateral spreading risks, through either ground improvement or structural measures, aimed at preventing building collapse and permitting the structure and its nonstructural system to satisfy the Section 1.1 performance goals. With the exception of Risk Category IV Essential Facilities, the provisions do not seek to control non-life-threatening damage to buildings that may occur as a result of liquefaction-induced settlement. For Risk Category IV Essential Facilities, the provisions seek to limit damage attributable to geologic and seismic hazards to levels that would permit post-earthquake use. For example, settlement is controlled to levels that would be expected to allow for continued operation of doors. Where the geotechnical investigation report indicates flow failure (i.e., liquefaction-induced strength loss that causes a slope to become statically unstable and therefore will have very large movement) or seismically induced long-runout landslides (e.g., landslides, debris flows, mudflows, and similar phenomena where strength loss causes very large slope movements) could affect a structure, these conditions are typically too severe to safely address only by foundation design, and mitigation of the condition should occur.

The ASCE 7-16 requirements for foundation design were limited in application to sites where liquefaction would cause a loss in soil strength. Starting with ASCE 7-22, the foundation design requirements were generalized to apply to any type of seismically induced soil displacement or strength loss. The provisions now capture the effects of differential settlement caused by liquefaction even if no associated strength loss occurs, seismic settlement caused by unsaturated

soils, seismically induced slope movement, and other conditions. Differential settlement and horizontal movement are the typical methods to evaluate structural distress caused by a variety of ground movements, including seismic slope movement, fault movement, seismic settlement, and other types of movement. Key literature on this issue includes Boscardin and Cording (1989) that contains a comprehensive assessment of building damage from ground movement. In general, differential settlement and horizontal displacement closely relate to structural damage, regardless of why the movement occurs. Geotechnical mitigation measures that prevent the movement from occurring in the first place can vary widely depending on the underlying cause of the movement (Oettle and Bray 2013), but once the movement occurs, structures respond similarly to ground movement regardless of the cause.

There is nothing in these provisions that is intended to preclude the Authority Having Jurisdiction from enacting more stringent planning regulations for building on sites susceptible to potential geologic hazards, in recognition of losses that may occur in the event of an earthquake that triggers liquefaction. Also, there is nothing in these provisions that is intended to indicate that shallow foundations, deep foundations, or mitigation is preferred for a specific structure; rather, the intent is to indicate minimum loading requirements for each foundation type. Stronger shallow foundations, deep foundations, or mitigation may be preferable, depending on site-specific conditions, which are to be discussed in the geotechnical investigation report per Section 11.8.3.

In the first paragraph of Section 12.13.9, it is stated that the foundation must also be designed to resist the effects of design earthquake seismic load effects assuming that liquefaction does not occur. This additional requirement is imposed since maximum seismic loads on a foundation during an earthquake can occur before liquefaction. This additional requirement provides assurance that the foundation will be adequate regardless of when liquefaction occurs during the seismic event.

Add Section C12.13.10 as follows:

C12.13.10 Requirements for Foundations and Structures Within Known Active Surface Fault Zones

The provisions in Section 12.13.9 to address soil liquefaction, ground movement, and soil strength loss are typically not applicable to ground movement from earthquake fault rupture that reaches the ground surface. Surface fault rupture can have compressive and shear type movement (reverse and strike slip faults) in addition to extension movement (normal faults), and the source of the ground movement extends deep into the earth, changing the potential mitigation measures for fault offset relative to other types of ground deformation addressed in 12.13.9. Therefore, different provisions are provided in this section compared to those for settlement, liquefaction, or lateral spreading. Designing a building to withstand the effects of a fault offset requires an extraordinary effort and very careful peer review. Normally designers will relocate the building to avoid the fault rather than designing for effects of fault offset. However, there will be times when design is required.

C12.13.10.1 MCFD and Threshold Values of Fault Displacement

Section 11.8.1 addresses how to determine the MCFD (i.e., the maximum considered value of relative surface displacement across an earthquake fault, also called fault offset), locations where the MCFD is expected to occur, and the orientation in three dimensions of the surface fault relative displacement.

If either the vertical or horizontal relative displacement exceeds the threshold value in Table 12.13.10-1, the MCFD shall be accommodated by the structural, geotechnical, or foundation design according to the requirement of Section 12.13.10. The horizontal displacement value considered shall be in the horizontal direction of maximum offset (i.e., the vector sum of two horizontal components of offset; this is shown as the Net Horizontal Slip (NHS) in Figure 11.8-X).

Factors affecting the damageability of structures from relative ground movement

The threshold values and criteria of Table 12.13.10-1 are based on observations of cases when surface fault displacement has and has not damaged structures; See Oettle and Bray (2013), Kropp and Comartin (2016), Gazetas et al. (2008). The characteristics listed in Section 12.13.10.1 that trigger Foundation Type B represent conditions under which the soil can impose significant horizontal forces that can pull apart the structure and foundation causing damage and possibly compromising safety. These are illustrated in Figures C12.10-1 and C12.10-2. Key points include the following:

- When an earthquake fault with horizontal displacement passes just under the edge of a building footprint, the foundation is more likely to remain integral as one piece because more limited forces can be transferred to the foundations from the more limited overlap of building over the fault. Other factors being equal, larger buildings are more likely to have larger overlap of a fault and therefore the forces transmitted from the ground to the foundation that tend to pull the structure apart can be greater.
- Given a situation in which horizontal forces from relative ground movement act to pull apart a foundation, foundations with elements well tied together in the horizontal direction (for example meeting the requirements of Section 12.13.9.2.1.1 of ASCE 7-22) will perform better.
- Irregular foundation footprints with reentrant corners are more likely to have weak sections that could be pulled apart by relative ground movement. The geometric definitions for reentrant corners in this section are selected to match those of Table 12.3-1 Item 2 regarding horizontal structural irregularities.
- Compared to a structure embedded several feet into the ground, a structure with a shallow mat slab foundation with little embedment into the ground is less likely to be damaged by relative ground movement because ground is likely to slide beneath the foundation without creating enough force to pull the foundation apart.

- Conversely, when a building is embedded several feet into the ground, relative horizontal ground displacement acts to move the building and forces up to the full passive soil pressure can develop on the embedded vertical faces of footings and integral retaining walls. Such pressures are often assumed to be proportional to the square of the embedment depth, making embedded structures more vulnerable to damage.

The issues are discussed in more detail in Maffei et al. (2009), which also discusses building resistance to vertical fault offset and strategies for the design of structures subject to fault displacement.

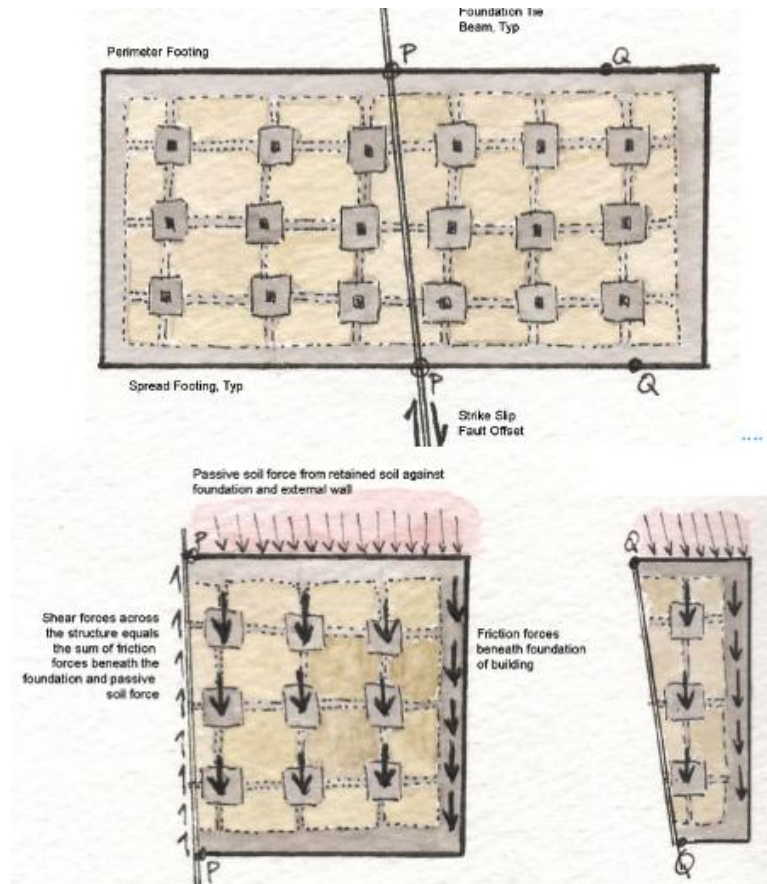
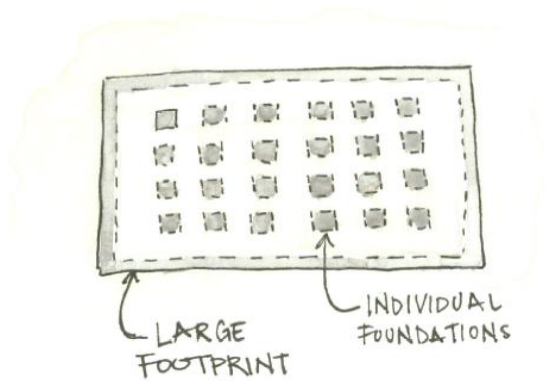
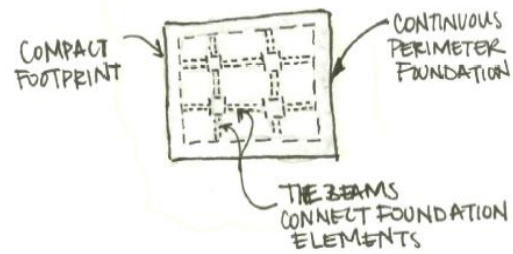


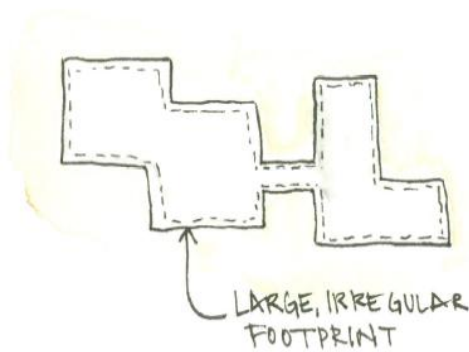
FIGURE C12.10-1 Foundation plan and free-body diagrams for an example building showing that maximum forces on the foundation from horizontal offset will be greater if the earthquake fault passes through the center of the building, line P-P, rather than toward one side of the building, line Q-Q. (Maffei et al., 2009)



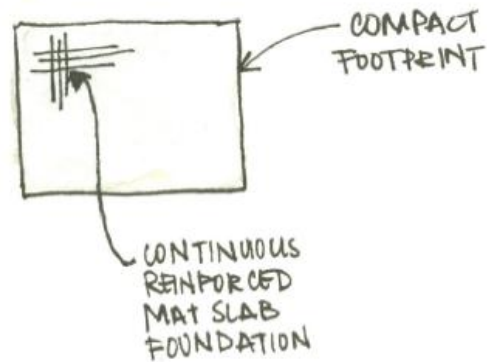
Larger footprint with individual foundations without foundation ties.



Small footprint with foundation elements tied together.



Large footprint with plan irregularities. Creates areas more vulnerable to stress concentration under ground deformation.

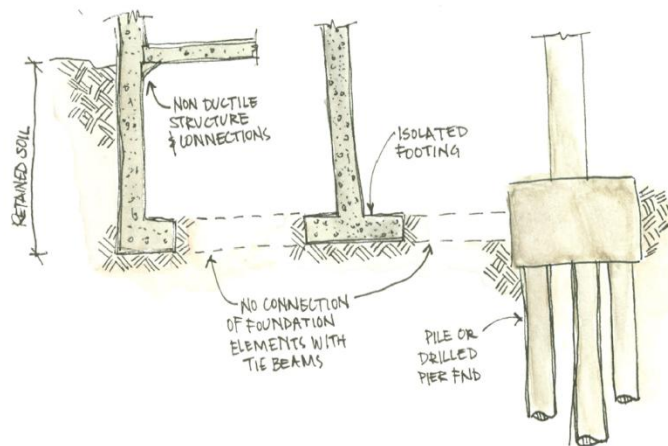


No irregularities in plan with ductile design and construction.

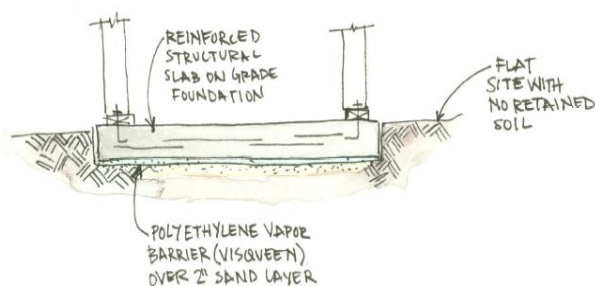
- Disadvantageous configurations for foundation plans

- Advantageous configurations for foundation plans

FIGURE C12.10-2 Advantageous and disadvantageous plan configurations and inter-connection, for foundations subjected to horizontal ground displacement from surface fault rupture (Maffei et al., 2009)



(a) Disadvantageous foundation conditions, which lack reliable horizontal interconnection between footings or pile caps, or which attract increased forces from embedment in the soil.



(b) Advantageous foundation condition for a wood-framed structure.

FIGURE C12.10-3 Advantageous and disadvantageous conditions for foundations subjected to horizontal ground displacement from surface fault rupture (Maffei et al., 2009)

C12.13.10.2 Structural Design to Accommodate Fault Offset Fault Displacement

The intention of the requirements of Section 12.13.10 is that, where the potential for surface fault displacement exists, the combined safety and damage risk to the structure from earthquake shaking and surface fault displacement remains within the building code's performance objective for the structure's Risk Category. Because of the complexities of comparing a range of expected strengths in both soil and structural elements, there are typically not standard or prescriptive solutions to designing for fault offset. For this reason, peer review is required.

In locations where there is a potential for earthquake surface fault displacement, there are scenarios in which earthquake shaking can occur without concurrent fault offset. For example, this could occur from an earthquake that does not rupture the ground surface, an earthquake that ruptures the ground surface but in a part of the fault trace not at the building site, or from an earthquake on a different fault. Thus Section 12.13.10.2 requires evaluating the structure for earthquake shaking with and without concurrent fault offset.

In Section 12.13.10.2, the reduction factor of 1.5 for reducing maximum considered fault offset displacements (MCFD) in load combinations with Design Earthquake ground shaking effects is consistent with reducing MCE ground shaking to Design Earthquake ground shaking.

In designing for fault offset, strategies include:

- Designing the structure with a strong, stiff mat foundation or similar interconnecting structure that is stronger than the forces that can be imposed such that the structure responds to the displacement with rigid body.
- Reducing passive pressure from retained soil on vertical surfaces of the structure.
- Reducing friction underneath the structure, for example with an intentional low-friction membrane.
- Placing the foundation on thick, ductile fill to reduce the foundation distortion, where reductions are supported by numerical modeling of the soil–structure–fault interaction;
- Strengthening the structure to resist vertical offset.

Observed Ground-Movement-Related Structural Damage in Past Earthquakes.

The 15-story Banco Central Building was not damaged due to its strong foundations when subjected to a 7 in. (0.18 m) surface fault rupture in the 1972 Nicaragua earthquake. However, in the 1999 Chi-Chi earthquake, 3 to 12 ft (1 to 4 m) of reverse fault rupture resulted in significant damage to a number of structures, including the collapse of a building with a weak foundation. Structures with stronger foundations tied together suffered less damage and did not collapse. However, the Attaturk Basketball Court in Turkey suffered significant damage from 5 ft (1.5 m) of normal fault movement in the 1999 Kocaeli earthquake because its pile foundation could not resist the forces generated by the piles tying the structure to both sides of the fault.

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COMMENTARY TO CHAPTER 13, SEISMIC DESIGN REQUIREMENTS FOR NONSTRUCTURAL COMPONENTS

Revise Section C13.3.1 as follows:

C13.3.1 Horizontal Seismic Design Forces

The seismic design force for a component depends on the weight of the component, the component Importance Factor, ground shaking intensity, seismic force-resisting system (SFRS) and dynamic properties of the supporting structure, vertical location of the component.....

Key Features

The parameters that were investigated include ground shaking intensity, seismic force-resisting system (SFRS) of the building, building modal period, building ductility, inherent building damping, building configuration, floor diaphragm rigidity, vertical location of the component within the building, component period, component and/or anchorage ductility, inherent component damping, and component overstrength. Parameters selected for inclusion in the final set of equations include all of the above except inherent building damping, building configuration, and floor diaphragm rigidity.

Using the above selected parameters, the proposed equations in NIST (2018) and in Section 13.3.1 include a set of key features. These include:

- *Ratio of Peak Floor Acceleration (PFA) to Peak Ground Acceleration (PGA).* Based on a detailed review of instrumented building strong motion records, a more refined equation was developed to relate PFA to PGA at different heights in the building. The equation incorporates the building period. This is accounted for in the variable H_f of Equation (13.3-1). The term PGA, as used in the nonstructural equations, refers to the zero-period acceleration at the base of the structure, which can be different (i.e., lower) than free-field acceleration (i.e., Multi-Period Response Spectrum zero period acceleration). See commentary Section C11.4.5 for more information regarding the reasons why base-of-structure and free-field zero-period accelerations can be quite different in the very-short-period zone. The value of PGA used in Equation 13.3-1 is taken as $0.4 S_{DS}$. This value represents the actual building response at the base of the structure that has historically been used in building codes and ASCE 7, where the ground motion response is represented by a two-period spectrum. The two-period spectrum matched reasonably well with earthquake strong motions measured in the WUS and it was the shape of the spectrum of ground motions that was used in the ATC-120

analytical studies used to develop Equation 13.3-1. However, very high frequency acceleration spikes in the period range of from 0.075 s to 0.15 s are observed in the MPRS (first introduced in ASCE 7-22) for sites in the CEUS that are not observed in the WUS. Variations of the shape of the response spectrum and especially the shapes found in the CEUS can significantly alter the actual ratio of the short-period response accelerations to the PGA. Because no damage has been observed associated with these high frequency acceleration spikes, for the interim, it has been decided to retain the definition of the base-of-structure PGA as $0.4 S_{DS}$. Further study is needed to understand how variations in the shape of the response spectrum of the type found in the CEUS MPRS affect the resulting floor accelerations.

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COMMENTARY TO CHAPTER 14, MATERIAL-SPECIFIC SEISMIC DESIGN AND DETAILING REQUIREMENTS

Revise Section C14.1.5 as follows:

C14.1.5 Cold-Formed Steel Deck Diaphragms

This section adopts the applicable standards for the general design of cold-formed steel deck diaphragms and steel roof, non-composite floor, and composite floor deck. The SDI standards also reference AISI S100 (2020c) for materials and determination of cold-formed steel cross-section strength and specify additional requirements specific to steel deck design and installation.

In addition, design of cold-formed steel deck diaphragms is to be based on AISI S310 (2020d). All fastener design values (welds, screws, power-actuated fasteners, and button punches) for attaching deck sheet to deck sheet or for attaching the deck to the building framing members must be per AISI S310 with specific testing procedures as prescribed in AISI S310. All cold-formed steel deck diaphragm and fastener design properties not specifically included in AISI S310 must be approved for use by the Authority Having Jurisdiction in whose jurisdiction the construction project occurs. Deck diaphragm in-plane design forces (seismic, wind, or gravity) must be determined per ASCE 7 with seismic forces determined in accordance with Section 12.10. Seismic diaphragm design forces for welded bare steel deck diaphragms are required to be determined in accordance with Section 12.10.2 or 12.10.3. For other bare and concrete-filled steel deck diaphragms, any of the diaphragm design method of Section 12.10.1 is permitted. Cold-formed steel deck manufacturer test reports prepared in accordance with this provision can be used where adopted and approved by the Authority Having Jurisdiction for the building project. The Diaphragm Design Manual produced by the Steel Deck Institute (2015) is also a reference for design values.

Equivalent lateral force (ELF)--based seismic design of bare steel deck diaphragms per Section 12.10.1 allows diaphragm forces to be reduced based on the response modification factor, R , for the particular vertical seismic force-resisting system, subject to minimum diaphragm force levels as defined in this standard. The reduction in the diaphragm force levels is independent of the ductility or deformation capacity of the diaphragm. Analysis of a large-scale RWFD archetype building under high demand with precast tilt-up walls and bare steel deck diaphragm roofs that either meet or violate the special seismic detailing requirements was completed by Schafer (2019). He found that a mechanically fastened roof that met the special seismic detailing requirements of AISI S400, Section F3.5, had approximately one-half the roof shear angle demands and one-half the anchorage demands of an equivalent welded bare steel deck diaphragm roof that did not meet the special

seismic detailing requirements. If the designer desires (for force reduction) or expects (due to the nature of the structure) inelastic demands in a bare steel deck diaphragm, the special seismic detailing requirements provide a means to ensure ductility and deformation capacity in the diaphragm. Both Section 12.10.2 and 12.10.3 require higher diaphragm design forces if the special seismic detailing is not met and allow reduced forces when the special seismic detailing is met.

Add Section C14.1.10 as follows:

C14.1.11 ROD-BRACED DIAPHRAGMS

The expected behavior of rod-braced diaphragms with special seismic detailing is for the rod-braces to yield along the length of the unthreaded body of the rod, while preventing net section fracture of the rods and failure of the other diaphragm members and their connections. There needs to be an explicit load path to deliver the lateral loads to the rods. The rods should be designed to resist all of the diaphragm shear, without consideration of floor grating, steel deck, standing seam roof, or other sheathing.

Experimental tests and computational studies were used to evaluate the strength and ductility of rod-braced diaphragm systems (Meek et al. 2023). The study considered rod diameters between 3/8 in. up to 1-1/2 in. although only a subset of the diameters were tested. These tests used a hillside washer at the connection of the rod to the beam that can accommodate a wide range of angles between rod and beam as shown in Figure X-1. Rod connection specimens with hillside washers were subjected to monotonic and static partially-reversed cyclic loads. The test results showed that the tensile yield strength of the rods was developed when the limit states of bearing in the web of the framing member (rafter beam), local buckling of the flange of the framing member (rafter beam), and shear of the hillside washer bearing tab, were prevented. The findings confirm results from previous studies (Foroughi et al. 2020; Sinno 1993) but indicated that the hillside washer needed to be mechanically attached to the web to remain seated during cyclic loading. The method used in the tests to attach the hillside washer to the web of the framing member was a #12 self-tapping screw with washer placed through an oversized hole (1/2 in. diameter) or slotted hole (3/16 in.x1/2 in.) in the hillside washer as shown in Figure X-1. In this way, the hillside washer was allowed to slide until the bearing tab engaged the edge of the hole in the beam web, but was prevented from becoming unseated during cyclic loading. Diaphragm subassembly specimens using rod connections with hillside washers were tested under static cyclic loads. The test results showed that tensile yielding of the unthreaded portion of the rod provided a well-defined yield mechanism when the stability of the framing members (roof purlins) was maintained, and the hillside washers remained engaged in the hole of the framing member (rafter beam). Figure X-2 demonstrates the requirements for lateral bracing and extents of the compact flange requirement. Two hillside washers are shown suggesting a case where two adjacent diagonal rods meet.

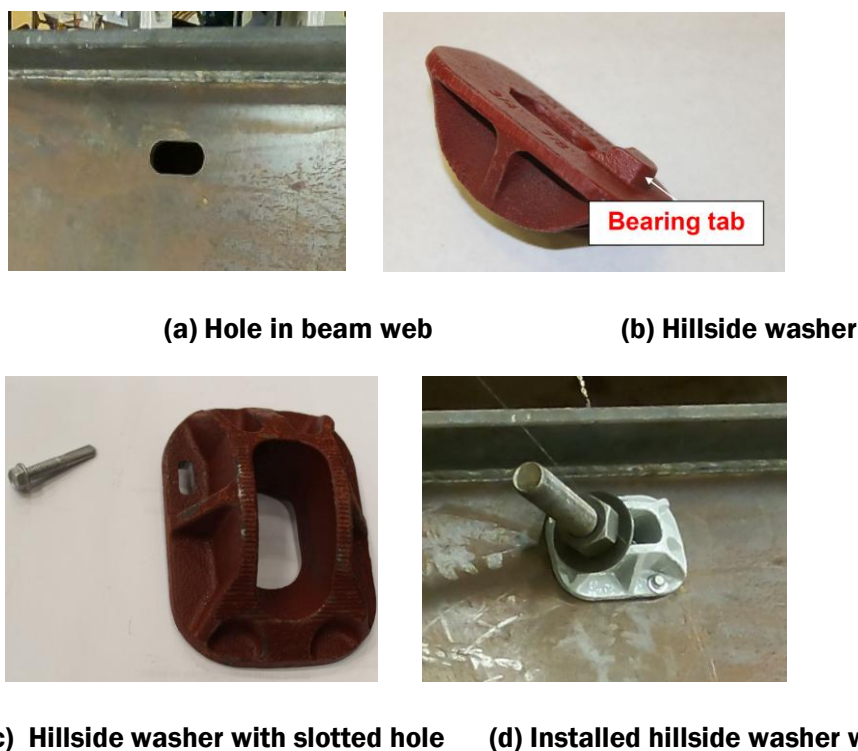


Figure X-1 – Example of hillside washer with screw to beam web to maintain position.

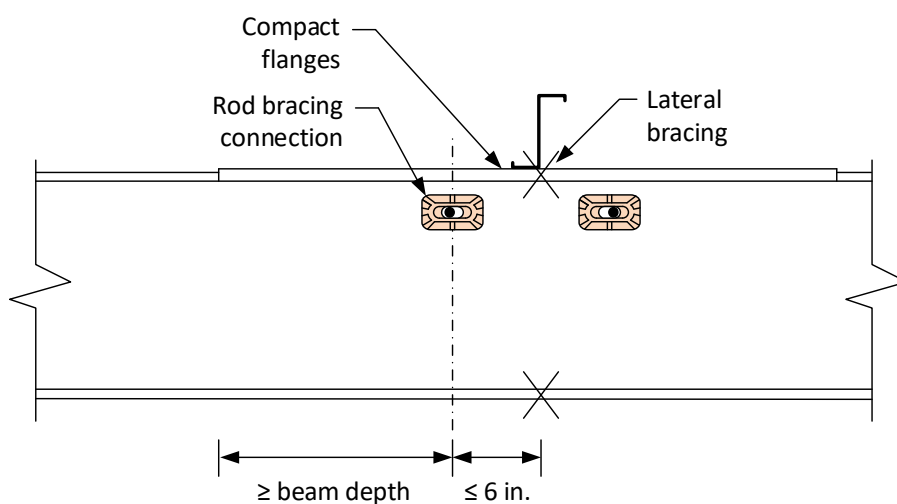


Figure X-2 – Schematic diagram demonstrating location of lateral bracing and extents of compact flanges in vicinity of a hillside washer.

Surrounding members and connections are capacity designed for the expected rod brace strength. Hillside washers, clevises and turnbuckles need to be shown capable of developing the tensile strength of the rod. The exception listed in 14.1.11.4(e) provides an upper limit on rod forces for cases where rods are oversized. The value of 2.5 is equal to the Diaphragm Design Force Reduction Factor, R_s , and the resulting forces are associated with elastic rod behavior during the design earthquake.

Section 14.1.11.4(f) amplifies the diaphragm design forces in cases where there is a single bay of rod bracing. In these cases, the rod-brace diaphragm has a lack of redundancy. A factor of 1.3 was selected because it has been used to address similar cases of low redundancy in ASCE 7.

Add Section C14.1.12 as follows:

C14.1.12.1 General

The Steel Damped Special Moment Frame (DSMF) system provides a simplified way to design new steel moment frames with supplemental viscous dampers, following Chapter 12 with slight modifications rather than requiring full compliance with Chapter 18. This design procedure is informed by elements of Chapter 18 although the procedure is separate from Chapter 18 and has been independently validated by FEMA P695 (FEMA, 2009).

The scope of buildings allowed to utilize the DSMF seismic force-resisting system is narrower than those permitted in Chapter 18. This was done so that the procedure can be implemented prescriptively. A key constraint of this design procedure is that 25% damping in the fundamental mode at the design earthquake (DE) level shaking and a damping velocity exponent of $\alpha = 0.4$ are prescribed. Limiting these parameters allows for a predictable damping response, such that analyses of the Steel Moment Frame (SMF) system and Damper Frame (DF) system can be decoupled.

The following is a brief summary of the design procedure:

Steel Moment frames (SMF): The SMF is designed in alignment with Ch. 12 as a Steel Special Moment Frame using modal response spectrum analysis. The analytical model of the SMF structure is not required to include any elements of the DF that are not shared elements (e.g. the dampers, extender braces or gusset plates).

Modifications to the parameters for a Steel Special Moment Frame (without dampers) are:

- $C_d = 4.5$ instead of 5.5 for undamped Steel Special Moment Frames.
- Base shear for strength design is reduced by 25%.
- Minimum base shear for checking drifts uses a new equation that is based on S_{D1} , rather than Equation 12.8-7, which is based on S_1 – although Equation 12.8-7 is still held as a bound.

Damper Frame (DF): The DF (i.e. the viscous dampers and the framing that constitutes the damper force load path) has the following features:

- Provides 25% damping at DE level shaking by computing the target amount of stiffness-proportional damping required at each damper location, then selecting available dampers within a prescribed range around that target. These are the specified dampers.

- DF elements are designed for overstrength damping forces that could be generated by the specified dampers. The analysis of the DF is based on the simultaneous activation of maximum damper forces similar to the procedure for a buckling restrained brace frame.
- Damper stroke requirements make use of an amplification factor that is calibrated to ensure that buildings designed with the procedure will have generally less than 10% probability of exhausting the damper stroke in the MCE_R event.

Shared SMF and DF elements: Shared elements and transfer elements between the moment frame and damper frame are required to use a load combination that accounts for the expected overlap between displacement-based demands (strain) and velocity-based demands from a nonlinear damper with an exponent of 0.4.

C14.1.12.1.1 Scope

This design procedure applies to DSMFs used as the primary lateral force resisting system in a principal direction. The DSMF system may be combined with other systems according to the rules of Section 12.2.2 and 12.2.3. To avoid duplicate requirements, references to specific sections of Chapter 18 are used. However, it is intended to invoke only those specific sections and not the entirety of Chapter 18.

C14.1.12.1.2 System Description

The Steel Damped Special Moment Frame System (DSMF) consists of Steel Special Moment Frames (SMFs) with supplemental damper frames (DFs) utilizing fluid viscous dampers (FVDs) – other forms of supplemental damping are not allowed given the validation process used to support the development of this procedure. The DF is defined separately from the SMF, although the two systems may have common elements. SMFs act as the primary lateral load resisting system for both wind and seismic excitation; however, the DSMF design procedure only addresses seismic excitation and the procedural modification to the SMF design as a result of the supplemental damping. Accounting for the supplemental damping for wind design would require alternative analysis beyond the scope of this procedure.

The relationship between the SMF and DF can be characterized by three types of configurations, illustrated in Figure C14.1-1. In a Type I system, there are no shared elements between the SMF and DF. In a Type II system, the DF is in the same bay as the SMF. In a Type III system, the SMF and DF share some common elements. Elements common to the DF and SMF are designed for a combination of the demands of the two systems. The DF may also be offset from the SMF in plan as shown in Figure C14.1-2, meaning that the DF can be located in gravity frames as well as within the SMF. In this procedure, buildings are permitted to have any combination of Type I or Type III configurations. Additionally, the vertical distribution of dampers can vary from floor to floor – they do not have to stack vertically (Figure C14.1-3). This is allowable because FVDs do not contribute stiffness to the building system and therefore staggering them does not create complications in the vertical stiffness regularity of the structure.

These specific provisions do not permit the Type II configuration due to concerns regarding the introduction of the damper to frame connection altering the moment frame. The performance of the special steel moment frame is tightly controlled by prescriptive design requirements in AISC 341, in particular requiring prequalified beam-column connections. The introduction of a damper gusset plate can affect the connection in a way that may alter its performance from what was captured in the qualification testing and the design provisions in AISC 358. This is not to say that such a configuration cannot be done in a design conducted in Chapter 18 that is subject to peer review, but for this prescriptive procedure it is not allowed.

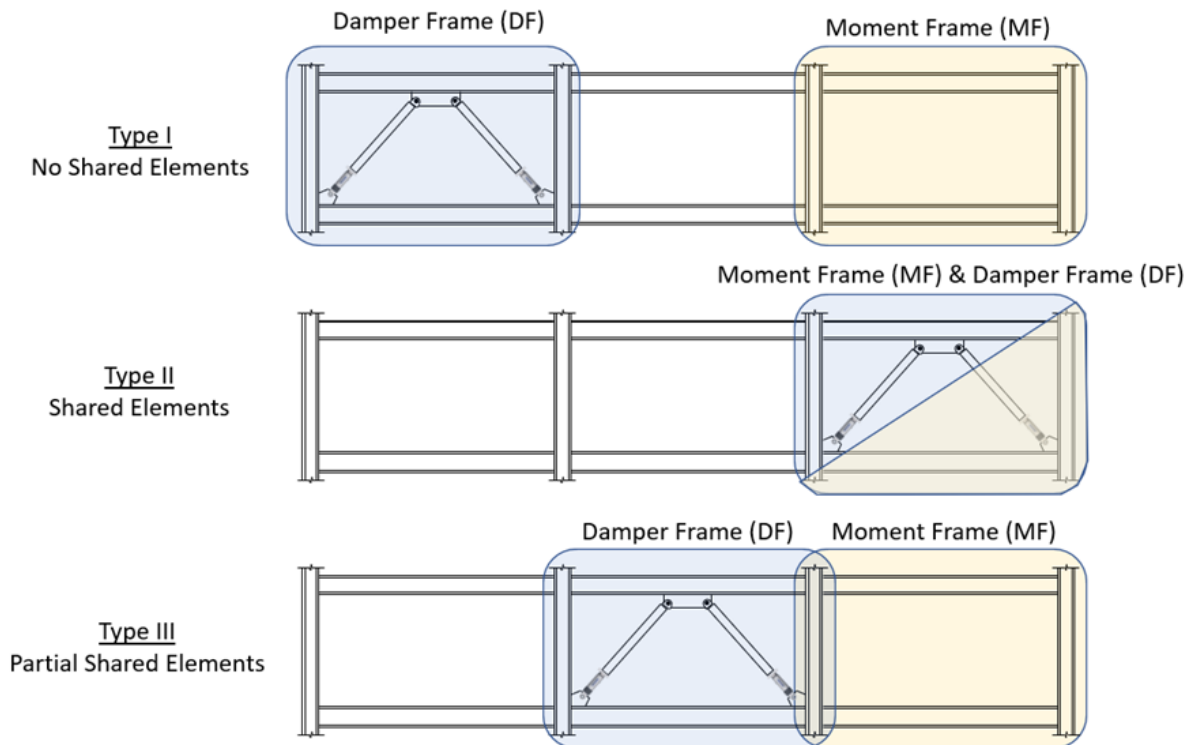


Figure C14.1-1. SMF and DF Configurations

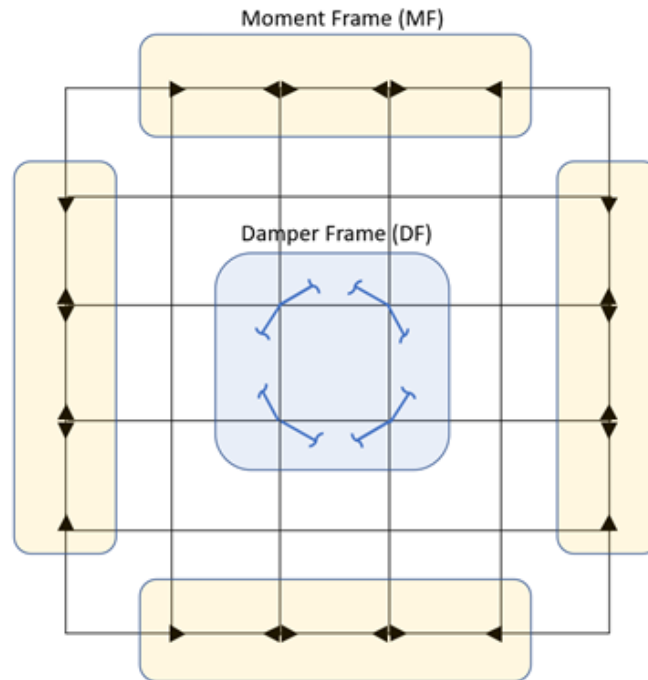


Figure C14.1-2. Plan View – Type 1 – Parallel Offset

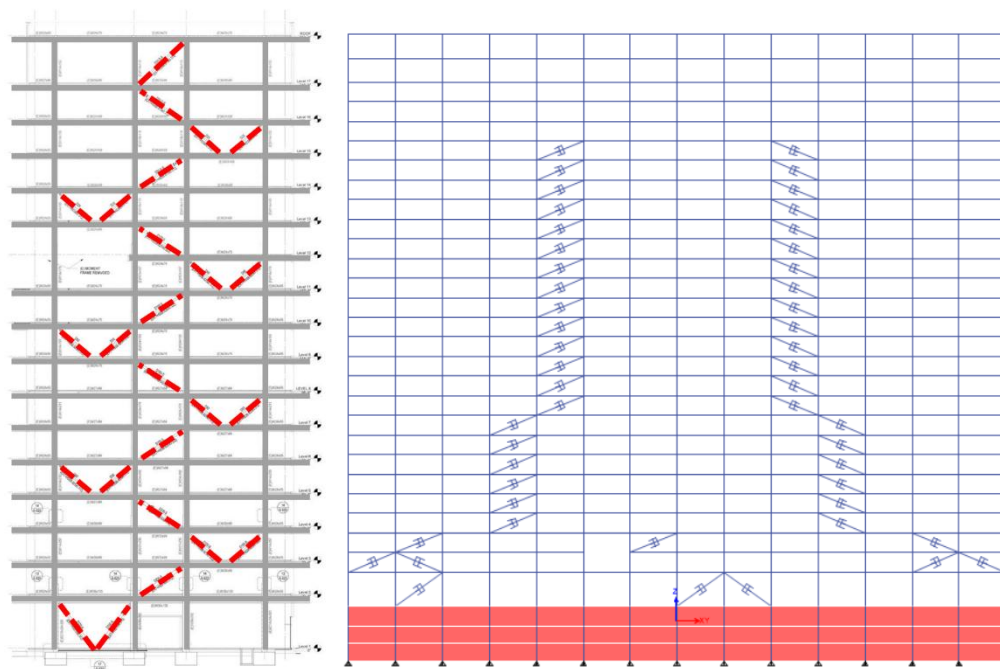


Figure C14.1-3. Examples of Staggered Vertical Distribution of Fluid Viscous Dampers

DFs are configured with Fluid Viscous Dampers (FVDs) in any of the following configurations shown in Figure C14.1-4, where dampers are in-line with the inclined braces between work points, and Figure

C14.1-5, where dampers are placed horizontally. Where dampers are placed horizontally, there may be two dampers (one on either side of the central gusset) or just a single damper on one side. The dampers may terminate at the beam-column connection or along the length of the beam. In all cases, localized effects due to the damper force and any eccentricities have to be resolved.

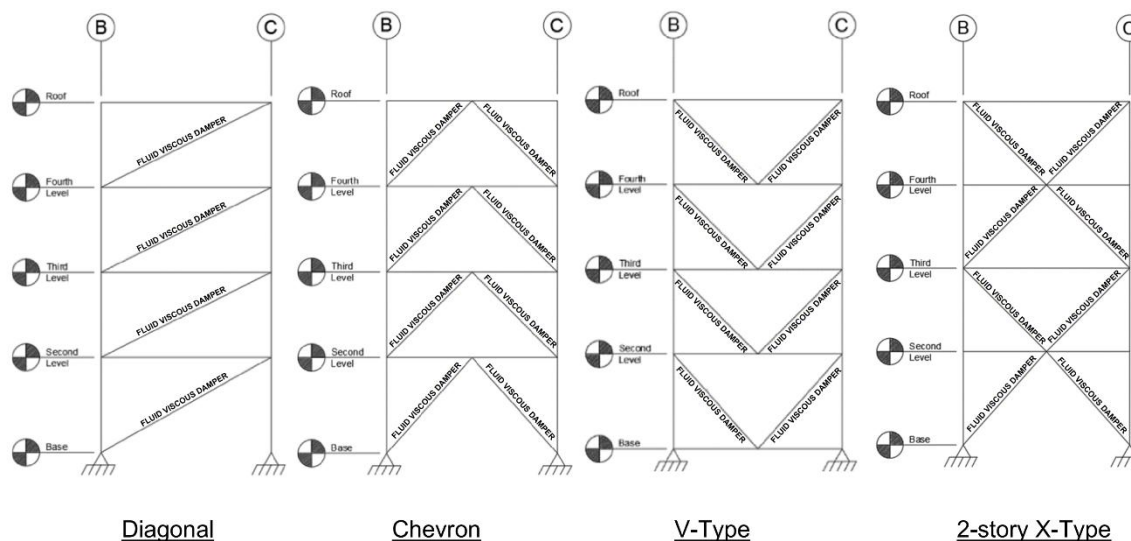


Figure C14.1-4. Common DF System Configurations

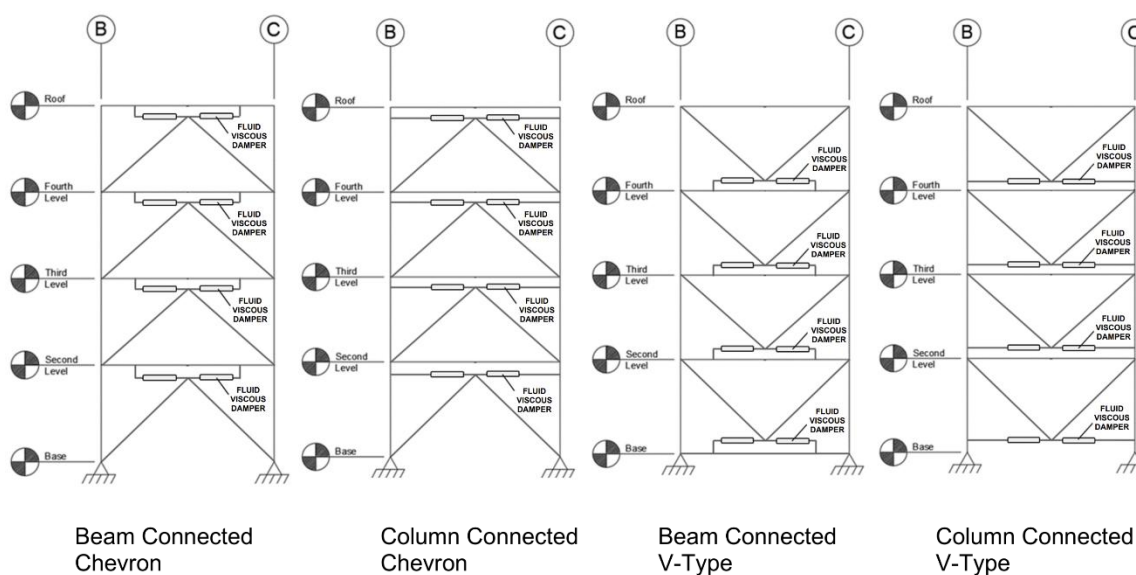


Figure C14.1-5. —Modified Chevron with Horizontal Dampers DF Configuration

C14.1.12.1.3 System Limitations

The use of the DSMF, as defined in these provisions and in contrast to a Chapter 18 design, brings certain limitations which were required in order to develop a prescriptive approach to the design of what is fundamentally a dynamic and nonlinear system. This section describes those limitations and

their purpose. There are many applications of supplemental damping allowed beyond this design procedure (e.g. alternative damping devices, applications to concrete structures, etc.), which are governed by the provisions of Chapter 18.

The height limit of 300 ft. corresponds to the limit of archetype design space that was studied to validate that the design criteria satisfy ICC-ES AC494 with Annex C, *Qualification of Building Seismic Performance of Alternative Seismic Force-resisting Systems (ICC-ES Guidance Document to FEMA P695)* (ICC-ES 2019 and 2022). Taller buildings were not studied, because the nonlinear analysis design procedure in Chapter 18 is more practical than this alternate design method for buildings taller than 300 ft.

Diaphragms serve an important role in the damped system to resist the damper-induced forces, especially when the dampers are not placed in line with the SMFs. Additionally, when dampers are offset vertically above and below a given diaphragm, the diaphragm serves to transfer the differential forces. For these reasons, flexible diaphragms are not permitted. Semi-rigid and rigid diaphragms are permissible, which can include systems such as CLT floors.

The limitation on the TIR of 1.4 aligns with existing limitations in Section 18.2.1.1.1 for the use of the reduced base shear for strength design. Similarly, the requirement in Section 18.2.1.1.2 to have at least two dampers on each floor in the direction of interest is carried through in this prescriptive approach. The evaluation of the TIR should be conducted on the SMF alone and damper eccentricities cannot be used to reduce torsional effects associated with this horizontal type irregularity.

However, it is worth noting that the redundancy penalties associated with Section 18.2.4.6 are not applicable to this system, allowing the use of just two dampers on a given floor in the direction of interest without the 1.3 factor penalty on velocity and displacement. The added requirements for amplification of force through the overstrength factor and amplification of stroke capacity to control the collapse probabilities negated the need for the redundancy penalty.

C14.1.12.2 Basis of Design

SMFs with supplemental damping designed in accordance with these provisions provide significant viscous energy dissipation, thereby reducing inelastic deformation of the SMF subject to earthquake ground shaking.

It is good practice to configure viscous dampers so that they are concentrically connected to beams and columns. However, eccentricities are allowed. The resulting moments and shears due to eccentricities are required to be fully considered in the design for strength. DF bays with eccentricities greater than half the beam depth also need to conform to the stiffness requirements of Section 14.1.12.7.4 to assure effectiveness of the damper. The configurations with horizontal dampers shown in Figure C14.1-6, for example, would impose a local moment in the beam due to the eccentricity in the damper force from the beam centerline.

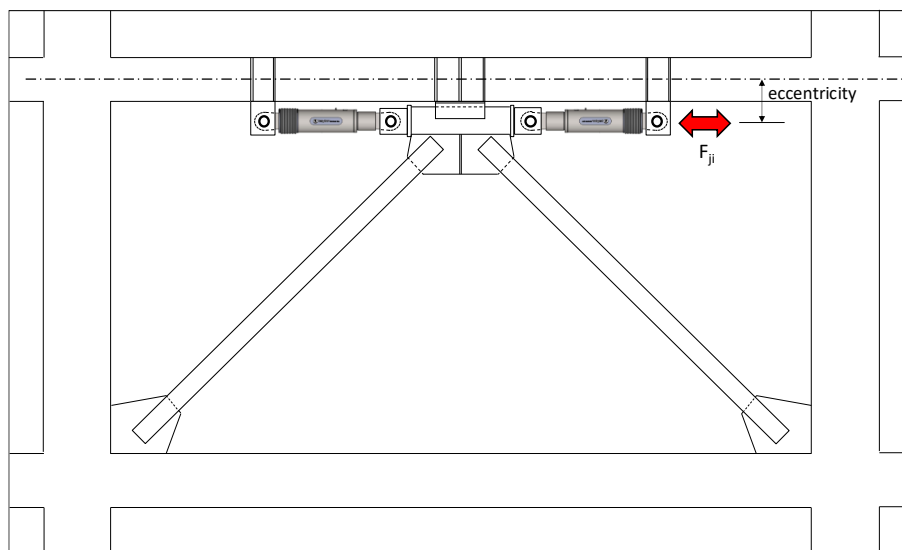


Figure C14.1-6. Schematic Drawing of the Modified Chevron with Horizontal Dampers Configuration

The assumption that MCE_R peak velocities and displacements are 1.5 times higher than DE peak velocities and displacements is supported by MCEER Report 00-0010 (Ramirez et al. 2001), although the assumption is somewhat conservative for velocity.

This design procedure does not address wind design. While it is permissible to ignore the effects of additional supplemental damping for computing wind loading, the added damping may be advantageous when computing wind load effects. For example, supplemental viscous damping can reduce the gust effect (See Section C26.11 for an example).

The DF is defined separately from the SMF to make the design procedure simpler. By incorporating the effects of the DF on the SMF directly into the design criteria in a prescriptive manner (e.g. reducing the base shear by 25%), the entire system can be designed with fewer iterations. Thus, the process for designing the SMF is very similar to the design process for other moment frames that do not have supplemental damping.

The DSMF design procedure has been validated by collapse assessment of approximately 100 archetype designs, ranging from 2- to 20-stories in height, using the FEMA P695 procedure. Many of the design requirements are based on rational engineering analysis (e.g. $C_d = 4.5$) and traditional precedent for designing buildings with supplemental damping (e.g. 25% base shear reduction for strength checks), while others are calibrated from the nonlinear response history analyses to ensure adequate collapse resistance is achieved. In the FEMA P695 validation, “collapse” was conservatively defined as the point where the first damper in the structure exceeded its stroke capacity. Sidesway collapse mechanisms could also be captured in the analysis, but did not control any of the archetype structures. While the approaches for developing various aspects of the design procedure vary (engineering analysis, precedent, engineering judgement, calibration to nonlinear

response history results), acceptability of the DSMF design procedure is ultimately based on meeting the collapse resistance requirements of FEMA P695 (Welch et al., 2022).

C14.1.12.3 Maximum Displacement Stage Analysis

C14.1.12.3.1 SMF Analysis

For conducting the SMF analysis, the SMF model should include all elements of the SMF, including members shared with the DF (such as shared columns). Dampers and members of the DF that are not shared with the SMF are not required to be included in the SMF analysis model. Moreover, the FVDs required for this system are nonlinear ($\alpha = 0.4$) and therefore a linear dynamic analysis would not “see” these elements in the analysis. The effect of damping on the SMF system response is indirectly considered through changes to the SMF design procedure listed in this section (e.g. reduced C_d and reduced minimum base shear).

Each modification to the design process for an undamped Steel Special Moment Frame are detailed and discussed herein:

Table 12.2-1 – Design Coefficients and Factors for Seismic Force-Resisting Systems:

The procedures of ICC-ES AC494 and FEMA P695 were used to qualify the DSMF as an alternative lateral force resisting system, establishing R , C_d and Ω factors to meet the collapse probability expectations of the code. The response modification coefficient, R , remains the same as a Steel Special Moment Frame. The deflection amplification factor, C_d , is 4.5 rather than the 5.5 that is specified in Table 12.2-1 for Steel Special Moment Frames (an 18% reduction). This modified C_d accounts for the reduction in drift due to supplemental viscous damping. The reduction in first-mode response determined in accordance with Equation 6-17 of FEMA 440 (FEMA, 2005) with the damping provided through this prescriptive method is approximately 45%. Based on a more conservative assumption that C_d should be equal to the response modification coefficient, R , for structures without supplemental viscous damping, the 45% reduction in response would correspond to $C_d = 4.4$ for a Steel Special Moment Frame (which uses $R = 8$). Note that the same conclusion for computing the “real” DE drifts can also be drawn from MCEER Report 00-0010 or from Table 18.7-1, assuming a total effective damping of 30%, $B_{1D}=B_{1E}=B_{1M}=B_{mD}=B_{mM}=B_R=1.8$.

Section 12.3.4 – Redundancy

The redundancy factor, ρ , is taken as 1.0 for the DSMF system, consistent with Section 12.3.4.1, Item 9, although without the Chapter 18 requirement.

Section 12.8.7 P-Delta Effects:

Chapter 18 has no explicit stability requirement for steel moment frames with viscous dampers. However, this design procedure requires a basic level of stability in order to be consistent with Chapter 12 procedures and to ensure that buildings designed with this system are not susceptible to

stability problems. The stability coefficient is not expected to be a controlling design factor for buildings in high seismic locations.

Section 12.8.4.2 Accidental Torsion

The accidental torsion moment required by this standard is modified to account for possible additional torsion induced by eccentricity in the DF system. The modified accidental torsion equation assumes that the SMF will resist 25% to 100% of the DF induced torsion, depending on the configuration of the DF, and the DF will resist the remaining 0% to 75% of the DF-induced torsion. The amount of DF-induced torsion resisted by the DF system (i.e. that does not need to be resisted by the SMF system) is based on its radius of gyration relative to the building width. For DF located on the perimeter, the SMF system is required to resist 25% of the DF-induced torsion. For DF lines with little separation (e.g. a central damper core configuration), the SMF system is required to resist virtually all of the DF-induced torsion. For simplicity, torsional resistance provided by DFs in the orthogonal direction is not considered.

Additional accidental torsion due to the damping system can be avoided by specifying a concentric damping system. Concentric damping can be achieved by making DF lines symmetric about the center of mass or by increasing or decreasing the damping constants of individual dampers such that the center of damping aligns with the center of mass. Additionally, damper properties may be varied to account for geometric eccentricities to maintain the center of damping within the allowed distance from the center of mass. This can be achieved so long as the requirements of Equations 14.1.12-8 and 14.1.12-13 are maintained. This approach can also be used to account for the contribution to damping and the center of damping from devices not aligned with one of the principal directions.

Note that per Section 12.8.4.2, buildings that are not classified as torsionally irregular do not need to include the effects of accidental torsion in the final seismic design. Therefore, most buildings with SMFs at or near the perimeter and that have little or no eccentricity in the damping system (i.e. buildings that are not classified as torsionally irregular) will be unaffected by this requirement, because accidental torsion is not required for their design. Inherent torsion is always included in the SMF design, in accordance with this standard.

Section 12.9.1.4.1

The equations for computing minimum seismic base shear are reduced by 25% to account for the reduction in response resulting from the DF. The value of 25% is consistent with the maximum permissible base shear reduction in Chapter 18. Although this reduction mirrors Chapter 18, the reduced base shear used for the DSMF has been justified through the FEMA P695 assessment.

Section 12.9.1.4.2 Scaling of Drifts

The trigger for scaling of drifts in Section 12.9.1.4.2 depends on which minimum base shear equation controls the design for strength. The minimum base shear for scaling of drifts applies, even where the strength is controlled by the alternative minimum for strength ($0.044 S_{DS}$). The alternative

minimum base shear equation for scaling drifts in this design procedure is a function of S_{D1} , so it is proportional to the MCE_R ground motion hazard and provides consistent performance from one site to the next. The modified minimum base shear equation of $0.35S_{D1}/(R/I_e)$ was found to produce designs stiff enough to consistently meet the FEMA P695 collapse resistance criteria using dampers that accommodate ~6% drift or often less. For comparison, the design requirements in Chapter 18 for systems with supplemental damping have no minimum base shear requirement for computing drifts.

The upper bound minimum base shear value of $0.5S_1/(R/I_e)$ is to ensure that moment frames with supplemental viscous damping do not have higher minimum base shear for calculating drifts than moment frames without supplemental damping. This upper bound comes into effect for sites with velocity amplification factor, F_v , greater than 2.1.

C14.1.12.3.2 Diaphragms, chords and collectors

Diaphragm inertial effects are reduced by 25% to account for the reduction in acceleration response resulting from the DF. The 25% reduction is consistent with the reduction in the base shear that is used for strength design. Diaphragm inertial effects are computed *only* in the maximum displacement-stage analysis, so they are not double-counted.

Note, the total diaphragm load can include transfer forces as well, which result from the maximum displacement stage load effect (Q_{MD}) and/or the maximum velocity stage load effect (Q_{FVD}).

Damper-induced diaphragm forces occur due to out-of-phase behavior between the dampers above and below a given diaphragm of interest. At the local level, dampers forces need a load path through connecting parts into the beam and then diaphragm. Global damper effects (internal shear and flexure) on the diaphragm are calculated based on Equation 14.1.12-6 using the cumulative overstrength damper forces on the floor above and below the diaphragm of interest. See (Canney et al. 2025) for more information.

When dampers above and below the diaphragm of interest are placed in line with the SMF, the force from those dampers do not contribute to diaphragm effects, instead the load path is directly to the SMF.

While it is critical to consider the horizontal component of the damper force in developing a complete load path from the damper into the diaphragm, it is not correct to add the lateral force into the system globally. Doing so would increase the global story shear and therefore the demand in the SMF. The impacts of the horizontal components of the dampers force on the SMF demands have already been accounted for in the development of the R , C_d and base shear reduction through the FEMA P695 process.

C14.1.12.4 Viscous Damper Constant for Target Viscous Damping

The design criteria specified in this section ensure ~25% viscous damping is provided for the fundamental mode in each principal direction for the design earthquake. Nonlinear viscous dampers

with velocity exponent, α , equal to 0.4 are required, and the vertical distribution of damping is approximately proportional to the story stiffness distribution, as determined from an ELF analysis. These constraints make the DF response and the reduction of the SMF response due to the supplemental damping consistent and predictable.

First, values of the linear damper constants, $C_{ji(L)}$, for each damper are determined for the target viscous damping using stiffness-proportional damping for the fundamental mode in each principal direction. Note that the term $\sum_{j=1}^n \cos^2 \varphi_{ji}$ is simply the number of dampers in story i , modified by the direction cosine if any dampers are not parallel to the direction being analyzed. Dampers that are orthogonal to the direction of interest drop out. Next, the nonlinear damping constant, $C_{ji(NL)}$, is determined, such that the equivalent energy dissipation from nonlinear damping at the DE shaking intensity is the same as with the linear damping constant.

C14.1.12.4.1 Target Linear Damper Constant

The linear damping constant for 25% damping in the first translational mode of vibration is computed in this section. There are many ways to compute $C_{ji(L)}$. The equation for $C_{ji(L)}$ in this section is arranged so that dampers that are only partially aligned with the principal direction will have the same damping constant as those that are aligned with the principal direction, given that their angles of inclination from the horizontal are the same. This formulation of the linear damping constant equation is guided by the notion that the dampers in the same story should also have the same damping constants, rather than other formulations which enforce principles that all dampers, even when not aligned with the principal direction, should have equal energy dissipation. The second form would generate large damping constants in dampers which are skewed. If the registered design professional desires to use dampers with varying damping constants in the same story and to resist demand in the same principal direction, an alternative formulation is provided by the exception.

For computing damper properties, the angle of inclination of the damper (θ_{ji}) is measured from horizontal, as illustrated in Figure C14.1-7. When DFs are not aligned with the principal direction, the angle of the DF from the principal direction (φ_{ji} , also illustrated in Figure C14.1-7) needs to be considered.

If alternate damper constants are desired (e.g. to align the center of damping with the center of mass and avoid damper induced torsion), any combination of damper constants that satisfy Equation 14.1.12-8 will provide the target ~25% first mode damping.

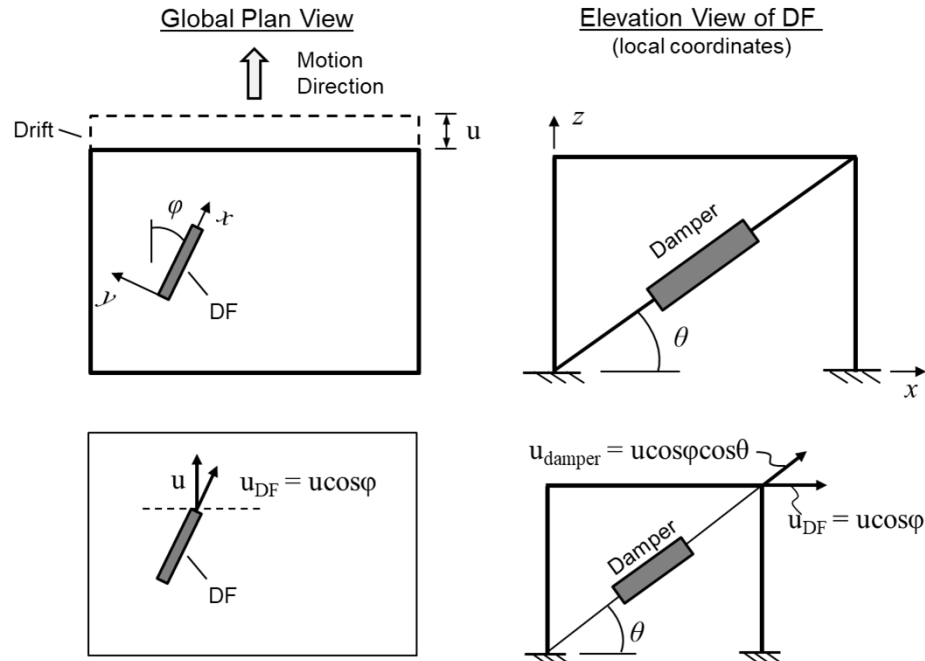


Figure C14.1-7. Illustration of Damper Orientation in Plan (Left) and Angle of Inclination (Right)

The equations for the damping constants, C , are not exact because the story stiffness is determined by the ELF procedure rather than by MRSA. Nonetheless, the damping constants resulting from these equations produce damping ratios that are consistently within a range of 24% to 26% damping. Figure C14.1-8 shows an example validation of the equations for computing the damping constants for a 4-story building.

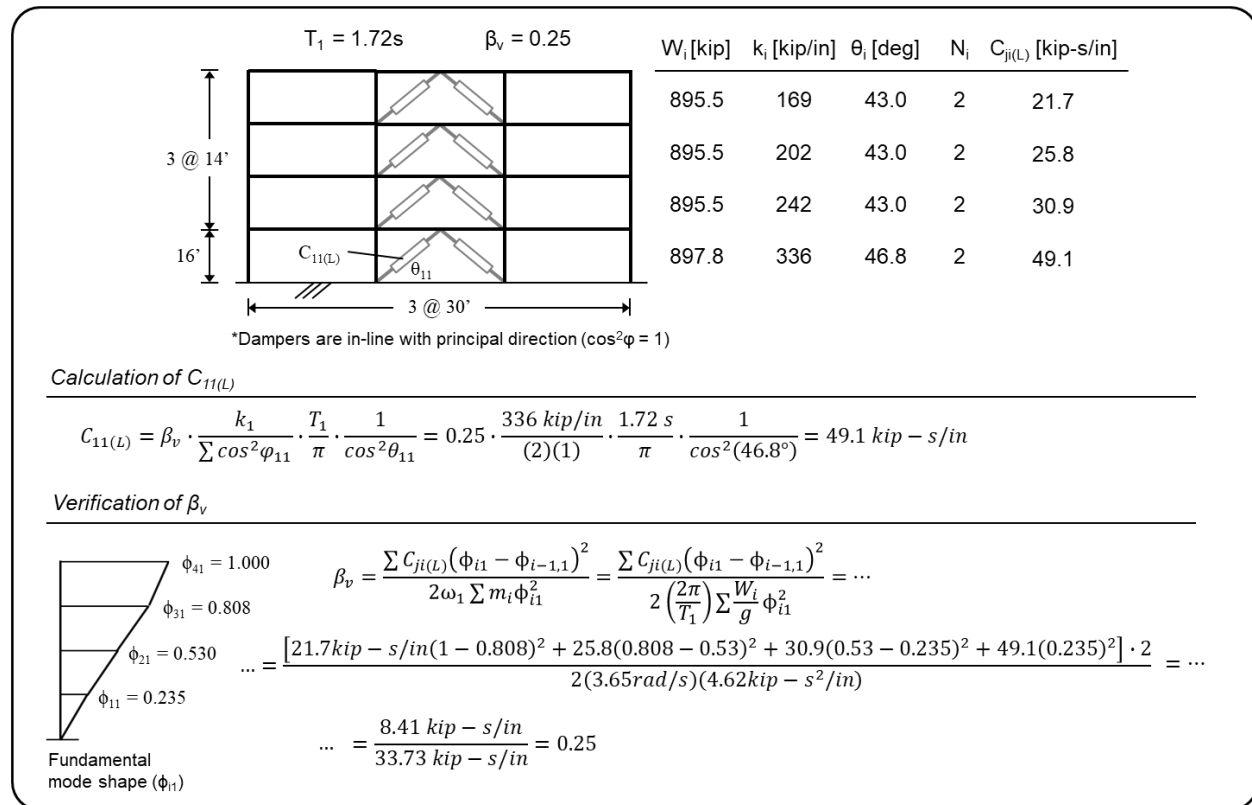


Figure C14.1-8: Sample Calculations for the Linear Damping Constant and Verification for Total Supplemental Damping in the Fundamental Mode

C14.1.12.4.2 Damper Velocity for Target Nonlinear Damping Constant

Damper velocity is approximated from peak damper stroke displacement at the DE intensity and a translational first mode period of the building.

A base shear correction factor, BF, is applied to the damper strokes to reverse the effect of minimum base shear scaling of drifts should the minimum base shear (Eqn. 12.8-7) control. This is important, because the minimum base shear for drift checks can be significantly higher than the DE base shear in mid-rise and taller buildings. Failing to account for this difference (i.e. not using the base shear correction factor, BF) could lead to first mode damping significantly greater than 25% for the DE hazard intensity. Note, the damping ratios for higher modes of vibration will be significantly greater than 25%.

Using displacements from the 3D modal analysis ensures that increases in damper stroke, and therefore velocity, due to torsion effects are accounted for in the design process. For buildings with inherent eccentricity in the SFRS, dampers on the soft side will undergo larger displacements and velocities, resulting in a larger value of $C_{ji(NL)}$, which counteracts the inherent torsion of the SFRS and reduces the torsional response of the building. To this end, it may be useful to place “dummy” elements in the analytical model to capture the location specific deformation of each damper element in the 3D model.

C14.1.12.4.3 Target Nonlinear Damping Constant

The equation for the nonlinear damping constant is adapted from MCEER 00-0010 (Ramirez et al. 2001). The nonlinear damping constant $C_{ji(NL)}$ is determined so that the energy dissipated by the nonlinear damper is the same as that of the equivalent linear damper of constant $C_{ji(L)}$ at the same peak velocity v_{ji} and same peak displacement d_{ji} . The equation for parameter λ is based on equation 4-14 of MCEER report 00-0010.

C14.1.12.4.4 Specified Nonlinear Damping Constant

To enable selection of dampers from a catalog and to support the use of the same damper in multiple locations, the specified value for C_{NL} , $C_{NL(spec)}$, needs to fall within the range of 0.9-1.3 times the target value from Section 14.1.12.4.3. The acceptable range of C_{NL} in this section is not related to testing requirements; instead, testing requirements defer back to Section 18.6.

The FEMA P695 nonlinear response history analysis (NLRHA) showed that specifying C_{NL} in the range of 0.9-1.3 of the target value results in satisfactory performance. Archetype structures with all dampers at the low end of range or all dampers at the high end of the range were both examined and were shown to meet the ICC-ES AC494 criteria.

C14.1.12.5 Maximum Velocity Stage Analysis

FVDs are velocity-dependent and therefore maximum design earthquake demands on the DF from FVDs are out-of-phase with inertial demands on the structure as determined from MRSA.

C14.1.12.5.1 DE Damper Velocity

The damper velocity computed in the previous section is based on a fundamental mode response. An amplification factor on damper velocity (A_v) is necessary to account for the increase in velocity due to higher mode effects and nonlinear behavior in the SMF. The amplification factor is a function of the number of stories of the DSMF system.

C14.1.12.5.2 FVD Forces at the Design Earthquake

The damper velocity (and corresponding force, f_{ji}) is determined for a design level earthquake (DE) response.

C14.1.12.5.3 FVD Forces at MCE_R

A maximum considered earthquake (MCE_R) level damper force, $f_{MCE,ji}$, based on a velocity 1.5 times the DE velocity, is also determined. This MCE_R level force corresponds to the force rating of the dampers that will be supplied in alignment with the required factor of safety of Section 14.1.12.7.2.

C14.1.12.5.4 Overstrength FVD Forces

Since the structure is expected to have a high probability of survival in the MCE_R event, it is necessary also to compute overstrength damper forces for designing members of the DF and transfer elements. The overstrength damper force accounts for upper bound damper properties due to manufacturing tolerances (via $R_v = 1.15$ for standard tolerances) and a velocity amplification factor of $\Omega_v = 2.5$. This results in an overall overstrength factor $\Omega_F = R_v x (\Omega_v)^\alpha = 1.66$ for DE level damper forces. The upper bound factor, R_v , is equivalent to the λ_{max} term of Section 18.2.4.5, accounting for multiple sources of variability in damper properties and is required to be justified through the testing requirements of Section 14.1.12.12.

For comparison, the design procedure in Chapter 18 requires an equivalent overstrength damper force factor ranging from 1.35 to 1.80, depending on redundancy penalties and a 20% amplification for force-controlled elements. This design procedure simplifies the approach by eliminating the redundancy factor and distinction between force-controlled and deformation-controlled elements.

Damper load effects in elements of the DF are determined from an analysis in which all dampers generate forces calculated in accordance with Sections 14.1.12.5.2-14.1.12.5.4. Figure C14.1-9 shows an example of the damper load effects in a DF with a chevron configuration. Vertical components of the damper force are resisted by the DF columns. Horizontal components of the damper force are transferred into the DF beams and, depending upon the load path to the SMF, into collectors or into the diaphragm.

Note that for the chevron or “V” configuration, there is no need to consider an imbalanced load in the central work point of the beam because the dampers and their components remain elastic at MCE_R and beyond. The imbalanced load required for brace solutions is due to the differential behavior of braces or BRBs in compression and tension, a condition which does not exist in this situation so long as the dampers on either side of the chevron have the same damping constant.

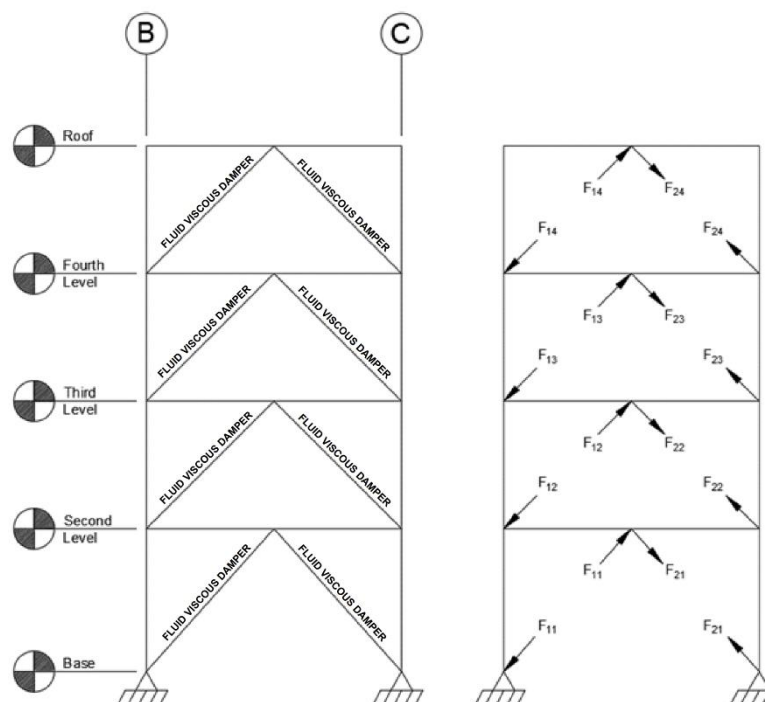


Figure C14.1-9. E_{FVD} Forces Imposed on DF with FVD in Chevron Configuration

C14.1.12.6 Steel Special Moment Frame (SMF) Design Requirements

The intent of this system is that any steel moment frame that meets AISC requirements for special moment frames is acceptable. All detailing and ductility requirements for the SMF are taken directly from the relevant AISC standards. For Type II configurations, there may be specific configuration or detailing considerations that vary depending on the SMF connection used. For example, in an SMF that employs Reduced Beam Sections (RBS), the RBS would be placed outside of the gusset plate connecting to the dampers and extender brace. This allows the ductile mechanism of the SMF to be maintained.

C14.1.12.7 Damper Frame (DF) Design Requirement

C14.1.12.7.1 Beams and Columns

Design of the DF is patterned after the design of buckling restrained braced frames, which is simple and can be done by hand if the DF member connections are treated as pinned in the structural analysis. Overstrength damper forces are used for proportioning the DF members in order to capacity protect the DF. The assumption that all of the dampers reach their peak force simultaneously (per Section 14.1.12.5) is conservative for designing columns in the lower stories; therefore, optional column axial load reduction equations are provided in Section 14.1.12.8.

A notable difference between the DF requirements in this section and buckling restrained braced frame requirements is that the minimum rotation capacity for option (a) of AISC 341 Section F4.6b is

increased to 0.050 radians for the DF. The purpose is to provide deformation compatibility with the SMF, which is capable of sustaining larger drifts than a buckling restrained braced frame.

C14.1.12.7.2 FVD Design Requirements

1. Velocity Exponent, α

All viscous dampers use the same velocity exponent, $\alpha = 0.4$. This allows the design procedure to be simplified and prescriptive.

The real force-velocity relationship for any fluid viscous damper will not have a velocity exponent of exactly 0.4 for the entire range of strokes and velocities that occur in DE and MCE_R earthquakes. Deviations in damper behavior from the idealized force-velocity behavior are handled by considering upper-bound damper forces (i.e. $R_v = 1.15$ in section 14.1.12.5.3) and are constrained by the prototype and production testing requirements of Chapter 18, which ensure that test data do not deviate too far from the idealized force-velocity behavior that is specified in design.

2. Damper Force Capacity Requirement

The damper force capacity requirement is based on the MCE_R level damper force to be consistent with the standard of practice and Section 18.2.1.2. Requiring a factor of safety of 1.6 on the MCE_R level damper force means that the damper force capacity will be required to be 1.88 times the DE damper force; this is somewhat higher than the damper overstrength factor, $\Omega_F = 1.66$, that is used for designing other members of the DF, given the value of R_v for standard production testing.

3. Minimum Required Damper Stroke, s_{req}

Damper stroke capacity significantly larger than the DE or even MCE_R stroke demands is necessary to ensure less than 10% probability of damper stroke exhaustion given MCE_R shaking, the metric used to define “collapse” in the FEMA P695 validation study. Therefore, damper strokes determined from the maximum velocity analysis are amplified by an “overstrength” factor, Ω_d . Note, this section amplifies damper stroke, not force, so the term “overstrength” is potentially misleading. Use of the term “overstrength” (rather than a more accurate term, such as “overstroke”) and application of the amplification factor to design level response (as opposed to MCE_R or some other level) is for consistency with typical applications of overstrength found throughout this design procedure.

The additional requirement for damper strokes at the first story of a multi-story building ensures that dampers on the bottom story can accommodate at least 85% of the interstory drift ratio that is required for the second story (note, damper strokes may still be very different at the first and second story, depending on the configuration and assumed base fixity). This requirement is important, because fixed-base columns can significantly reduce computed first story drifts in a linear design model; however, much of that reduction diminishes as columns begin to yield at the base in a large earthquake. The nonlinear analyses show that 85% is satisfactory to prevent premature bottoming of the dampers in the first story.

C14.1.12.7.3 Extender Brace Design Requirements

The extender brace is designed for overstrength damper forces to ensure that it does not fail prior to exhausting the provided damper stroke. The capacity of the extender brace should be determined in accordance with applicable AISC requirements, i.e. $\phi R_{Nji} \geq F_{ji}$ where ϕ and R_N are determined per AISC. Additional seismic ductility requirements for extender bracing are not required.

Extender braces need to be sufficiently stiff as to drive the damping device without significant stroke loss due to brace elastic axial deformation. If extender braces are not stiff enough, the elastic deformation of the brace takes away from the stroke of the damper, reducing the energy dissipation provided by the device. For this reason, minimum extender stiffness criteria are provided.

The extender brace stiffness equation in this section uses the full pin-pin length of the damper-brace assembly. Since the elastic stiffness of the damper is typically greater than that of the brace, the actual stiffness of the damper-brace assembly, K_A , will be somewhat higher than the value computed in this section. The minimum stiffness requirement for extender braces will rarely control the extender brace design, except for when relatively long extender braces with small dampers are used.

The intent of the minimum extender brace stiffness requirement is to ensure that the viscous damper assembly energy dissipation loss is not greater than 15% in comparison with idealized dashpot behavior, at or above seismic story drifts of 1.5%. For example, Figure C14.1-10 demonstrates the pinching of the damper hysteresis loops that occurs as the assembly stiffness, K_A , decreases, and Figure C14.1-11 demonstrates the reduction of the area enclosed by the hysteresis loops as a function of K_A .

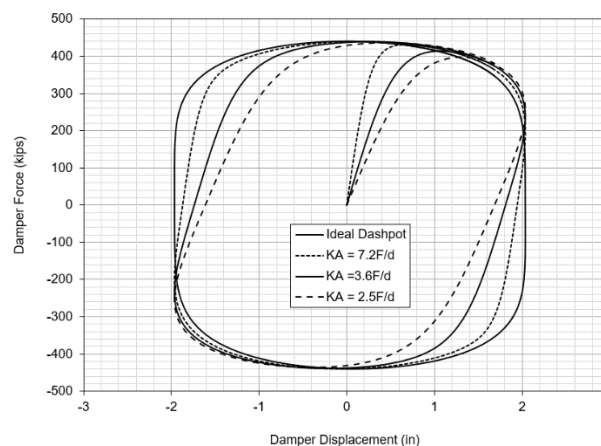


Figure C14.1-10. Viscous Damper Assembly Force-Displacement Response, $C = 200$ & $\alpha = 0.4$ (Maximum Force, $F = 440$ kips; Maximum Displacement, $d = 2$ in.)

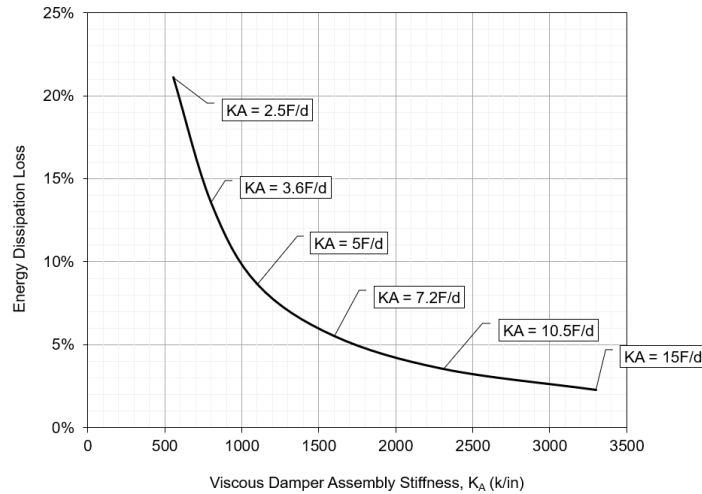


Figure C14.1-11. Viscous Damping Energy Loss (Efficiency) vs. Assembly Stiffness (15% Energy Dissipation Loss at $K_A \approx \frac{3.33F}{d}$)

Behavior of the viscous damper assembly follows the Maxwell model whereby a spring is in series with a dashpot as shown in Figure C14.1-12.

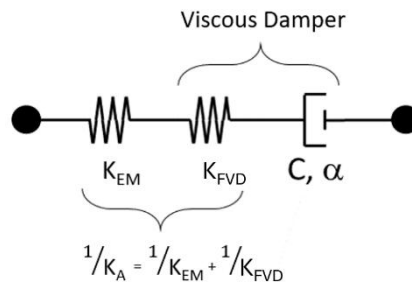


Figure C14.1-12. Viscous Damper Assembly Model

The damper Maxwell stiffness, K_{FVD} , will vary with different damper manufacturers and needs to be obtained or verified. For the devices used to validate this system, the damper Maxwell stiffness is approximately equal to $\frac{10F}{d}$, and if the required damper assembly stiffness, K_A , is $\frac{10F}{3d}$, then the minimum extender stiffness, K_{EM} , is $\frac{10F}{2d}$, or $\frac{5F}{d}$.

C14.1.12.7.4 Stiffness for DF with Eccentric Connections

Where connections are eccentric, meeting this requirement may involve increasing the stiffness of beams or columns in addition to the brace itself. The DF bay stiffness that is to be compared to K_{SM} can be modeled by replacing the brace and damper assembly with axial member(s) that have the same AE as the brace(s), pinning the column connections and applying a horizontal force to the DF bay; the stiffness is the horizontal force divided by the horizontal displacement.

C14.1.12.7.5 Connection Design Requirements

Connections are designed for overstrength damper forces to ensure that they do not fail prior the damper bottoming out. Capacity of the connections should be determined in accordance with applicable AISC requirements, i.e. $\phi R_{Nji} \geq F_{ji}$ where ϕ and R_N are determined per AISC.

C14.1.12.8 Optional Reduction of Damper-Induced Column Axial Loads for Continuously Damped Bays

The damper-induced column axial loads corresponding to the maximum velocity stage loading (E_{FVD}) can be reduced based on the more conservative of two different column force reduction factors (i.e., $CF1_{ji}$ and $CF2_{ji}$) for a given column j within story i . The factor $CF1_{ji}$ uses a cumulative “top-to-bottom” reduction approach based on the number of continuously damped bays above story i of interest (n_{ai}), with increasing load reduction for columns in stories supporting more dampers above. This reduction is referred to as cumulative since the reduction is applied to the cumulative damper-induced column axial load at the column of interest. An illustration of the definition of n_{ai} is shown in Figure C14.1-13 for a few common cases.

The second factor, $CF2_{ji}$, applies force reduction based on the local damper-induced column axial forces at a given story based on the number of stories above the local column of interest ($n_{ai,local}$), before taking the total damper-induced column axial load to be used for damper frame design. The definition of $n_{ai,local}$ is shown for different stories (i.e., columns) of interest in Figure C14.1-14.

The reduction relationships provide more accurate damper-induced column loads for taller buildings with continuously damped frame bays when compared to a simplified analysis that assumes that all dampers reach their peak force simultaneously. The relationships were derived based on a large set of nonlinear analysis archetypes where the damper-induced column axial loads could be monitored at the collapse limit state (i.e., stroke exceedance). A number of story limits are placed in the relationships to avoid unconservative force reduction for shorter buildings and limit the upper bound reduction to be consistent with the scope of the archetypes analyzed. This avoids extrapolation that is not supported by nonlinear analysis.

Two different damper-induced column axial load reduction factors (i.e., $CF1_{ji}$ and $CF2_{ji}$) are implemented to avoid unconservative loads for structures with specific details that could make one reduction factor more unconservative than the other. An illustration of two possible cases is shown in Figure C14.1-15.

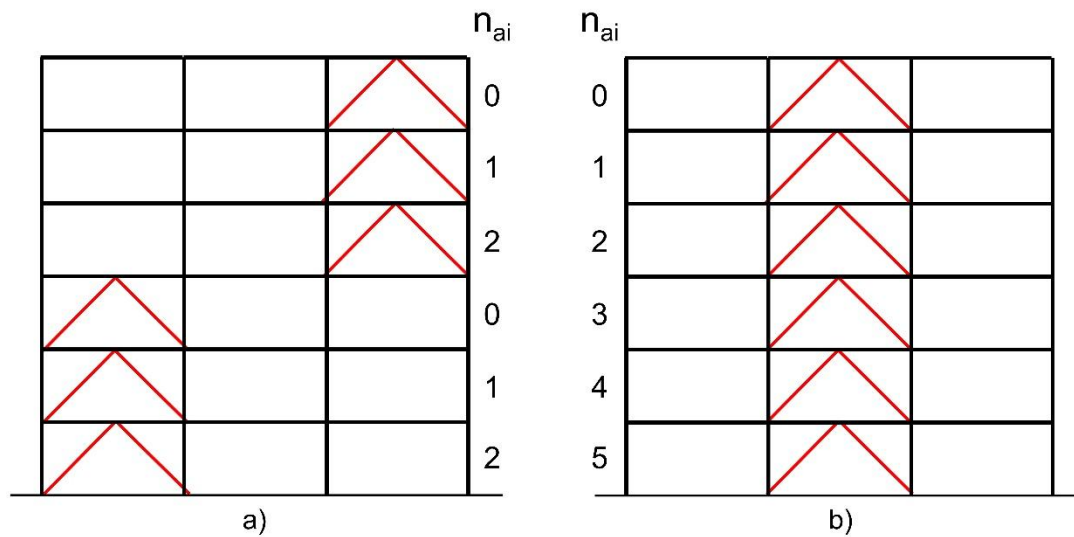


Figure C14.1-13. Illustration of the Number of Continuously Damped Bays for Damper-Induced Column Axial Load Reduction: a) Damper Bays Not Continuous in Elevation, b) Continuous Damper Bay in Elevation

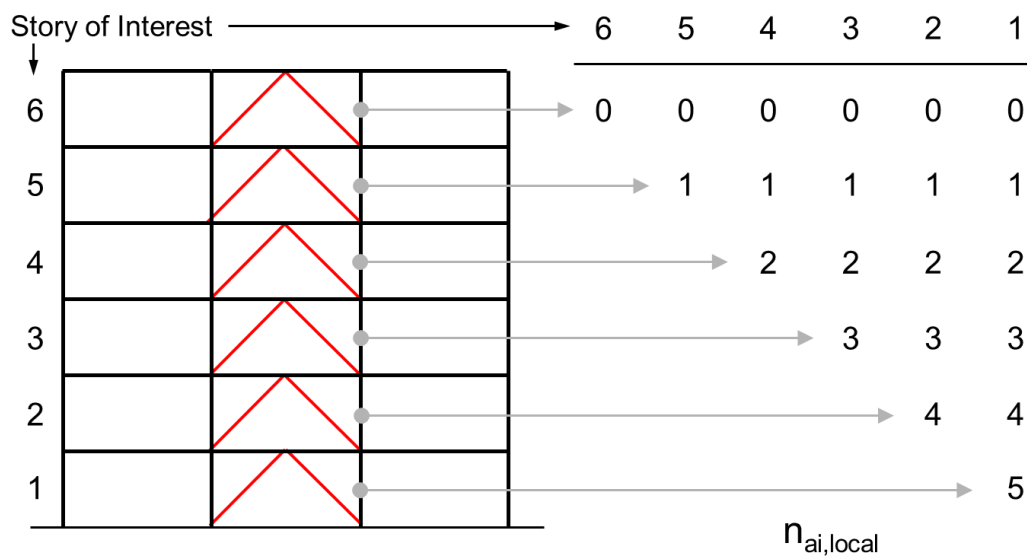


Figure C14.1-14. Illustration of Local Continuously Damped Bays Depending on the Story (Column) of Interest

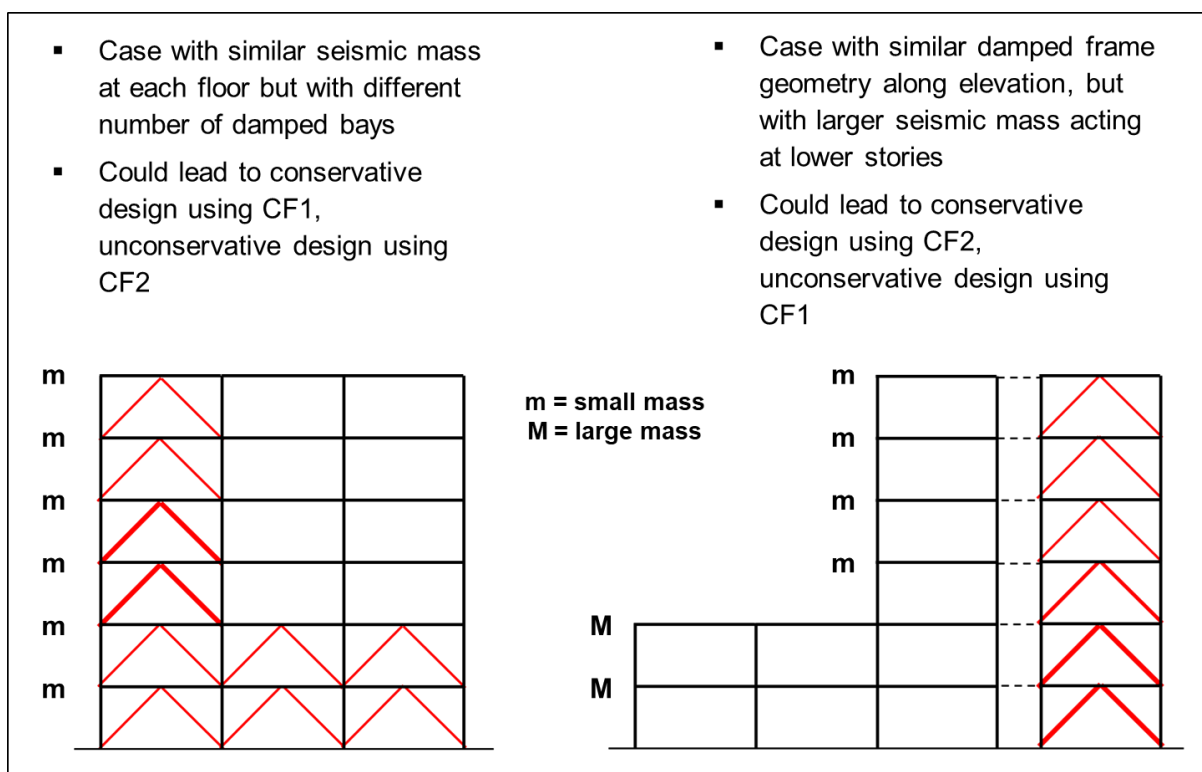


Figure C14.1-15. Illustration of Building Layouts that are Controlled by the Two Different Damper-Induced Column Axial Load Reduction Equations

The following tables show example axial load design values for a hypothetical 20-story damping frame. Table C14.1-1 illustrates the calculation of the cumulative $CF1_{ji}$ factors and corresponding reduced column axial loads ($P_{EFVD1,ji}$). Table C14.1-2 shows an example calculation for the $CF2_{ji}$ factors and reduced column axial loads ($P_{EFVD2,ji}$) that first reduce the local damper-induced column demands before taking the sum of all loads above the column of interest. Table C14.1-3 provides a summary of the unreduced (simplified analysis) loads compared with the controlling reduced axial load.

Table C14.1-1. Example Calculations for Column Axial Load Reduction Using Factor $CF1_{ji}$

	Story	Local Column Load $P_{local,ji}$	Cumulative Column Load P_{ji}	n_{aj}	$CF1_{ji}$	$P_{EFVD1,ji}$
	20	0	0.0	0	1.000	0.0
	19	84.5	84.5	1	1.000	84.5
	18	167.7	252.2	2	1.000	252.2
	17	240.8	492.9	3	1.000	492.9
	16	290.3	783.2	4	0.943	738.9
	15	297.9	1081.2	5	0.893	965.3
	14	349.5	1430.6	6	0.847	1212.4
	13	354.0	1784.6	7	0.806	1439.2
	12	403.4	2188.0	8	0.769	1683.1
	11	397.8	2585.8	9	0.735	1901.3
	10	398.2	2984.0	20	0.704	2101.4
	9	457.8	3441.7	11	0.676	2325.5
	8	461.0	3902.7	12	0.649	2534.2
	7	461.1	4363.8	13	0.625	2727.4
	6	457.2	4821.0	14	0.602	2904.2
	5	520.9	5341.9	15	0.581	3105.8
	4	523.4	5865.3	16	0.562	3295.1
	3	522.2	6387.5	17	0.543	3471.4
	2	591.7	6979.2	18	0.526	3673.3
	1	565.4	7544.6	19	0.500	3772.3

Table C14.1-2. Example Calculations for Column Axial Load Reduction Using Factor $CF2_{ji}$

Story of Interest		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
$P_{EFD2,j}$		3900	3699	3460	3274	3077	2873	2718	2542	2352	2148	1980	1784	1543	1306	1031	771	493	252	84	0
Local Story	$P_{local,j}$	$CF_{2,j}$																			
20	0	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1
19	84.5	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1
18	167.7	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1
17	240.8	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1
16	290.3	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1
15	297.9	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1
14	349.5	0.25	0.25	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1	1
13	354.0	0.25	0.25	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1	1
12	403.4	0.25	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1	1	1
11	397.8	0.25	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1	1	1	1
10	398.2	0.25	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1	1	1	1	1
9	457.8	0.25	0.25	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1	1	1	1	1	1
8	461.0	0.25	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
7	461.1	0.4	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
6	457.2	0.55	0.7	0.85	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
5	520.9	0.7	0.85	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
4	523.4	0.85	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
3	522.2	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
2	591.7	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1	565.4	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Table C14.1-3. Example Damper-Induced Column Axial Load Reduction Summary

Story	Damper-induced axial load $P_{local,j}$	Cumulative Axial load P_{ji} (no reduction)	n_{ai}	$P_{EFD1,j}$	$P_{EFD2,j}$	Reduced Axial load $P_{EFD,j}^1$	% Difference to No Reduction
20	0	0.0	0	0.0	0.0	0.0	0%
19	84.5	84.5	1	84.5	84.5	80.4	0%
18	167.7	252.2	2	252.2	252.2	234.9	0%
17	240.8	492.9	3	492.9	492.9	423.1	0%
16	290.3	783.2	4	738.9	770.6	668.2	-2%
15	297.9	1081.2	5	965.3	1030.7	935.3	-5%
14	349.5	1430.6	6	1212.4	1306.2	1181.3	-9%
13	354.0	1784.6	7	1439.2	1542.7	1439.4	-14%
12	403.4	2188.0	8	1683.1	1783.9	1653.5	-18%
11	397.8	2585.8	9	1901.3	1979.7	1832.6	-23%
10	398.2	2984.0	20	2101.4	2148.1	2050.4	-28%

9	457.8	3441.7	11	2325.5	2351.6	2266.0	-32%
8	461.0	3902.7	12	2534.2	2542.2	2468.5	-35%
7	461.1	4363.8	13	2727.4	2717.9	2658.4	-38%
6	457.2	4821.0	14	2904.2	2873.4	2873.1	-40%
5	520.9	5341.9	15	3105.8	3076.6	3071.3	-42%
4	523.4	5865.3	16	3295.1	3273.6	3257.4	-44%
3	522.2	6387.5	17	3471.4	3460.5	3431.9	-46%
2	591.7	6979.2	18	3673.3	3698.5	3630.1	-47%
1	565.4	7544.6	19	3772.3	3900.3	3800.0	-48%

¹ Based on taking envelope of $P_{EFVD1,ji}$ and $P_{EFVD2,ji}$

C14.1.12.9 Horizontal Seismic load Effect, E_h

The load combinations in this section account for acceleration at peak displacement, Q_{MD} , and maximum velocity, Q_{FVD} . While peak displacement and peak velocity are out of phase, the nonlinear nature of the force-velocity behavior of the dampers means that damper forces reach near peak force at relative low velocity. Combining $1.0Q_{MD}$ with $\pm 0.7Q_{TD}$ accounts for this combined behavior, based on Section 4.8 of MCEER report 00-0010.

At the instant of maximum acceleration, the phase lag parameter, δ , is:

$$\delta = \left(\frac{2\pi\alpha\beta_v}{\lambda} \right)^{\frac{1}{2-\alpha}} = 0.337 \quad (\text{MCEER Equation 4-35})$$

$$\alpha = 0.4$$

$$\beta_v = 0.25$$

$$\lambda = 3.582$$

Therefore, at maximum acceleration, the displacement factor is $\cos\delta = 0.94$, i.e. 94% of peak displacement. Since the structure is expected to yield, the seismic load effect due to 94% of peak displacement is approximately $1.0Q_{MD}$. The velocity factor at peak acceleration is $\sin(\delta)^\alpha = 0.64$. Assuming a velocity amplification factor of 1.25 per MCEER 00-0010 Table 4-6, the force correction is $1.25^\alpha = 1.093$, so the total load factor on peak velocity force is $0.64 * 1.093 = 0.7$. Thus, the peak acceleration horizontal load equation is $Q_{MD} \pm 0.7Q_{FVD}$.

With the exception of foundations, members in the direct load path of dampers (i.e. the DF and transfer elements between it and the SMF) are designed for overstrength damper forces, $\Omega_F Q_{FVD}$

even when overstrength is not required for the Q_{MD} effect. The overstrength earthquake load effect, E_{mh} , is required where this standard normally requires it.

C14.1.12.10 Orthogonal Load Combinations

Where orthogonal load combinations are required, the orthogonal damper load effect is multiplied by 60% rather than 30%, because the damper force is approximately 60% of peak at 30% velocity, due to the damper nonlinearity. Precisely, damper force at 30% velocity is $0.3^\alpha = 0.62$.

C14.1.12.11 Damping Devices

C14.1.12.11.5 Maximum and Minimum Damper Properties

There are three substantive changes in this section that impose requirements on DSMF dampers that are different than the current standard of practice for satisfying Section 18.2.4 requirements:

1. The range of acceptable maximum and minimum properties for dampers (Section 18.2.4.5) has been simplified significantly and a somewhat tighter tolerance is required for DSMF dampers.
2. Properties of damping devices are required to be within the allowable tolerance of the specified value, rather than a nominal value determined from testing. This is a stricter requirement for the manufacturer to meet.
3. The damping system redundancy requirement for extra displacement capacity (Section 18.2.4.5) is removed. The requirement states that damper stroke capacity is to be increased by 30% where there are fewer than four dampers in the direction of interest or fewer than two on either side of the center of mass. However, due to the conservative definition of collapse that was used for qualifying this system, the DSMF design requirements already require >30% more stroke capacity than would be required for Chapter 18 design, so this requirement is not needed for DSMF buildings.

In the exemption for the maximum and minimum design properties, there is an allowance to expand the range from $\pm 15\%$ at the discretion of the registered design professional. This could be used to accommodate FVDs with a wider spread in performance metrics or to accommodate a range of anticipated operating temperatures outside of the 50 to 90 degrees Fahrenheit (10 to 32 degrees Celsius) expected for damper application in climate-controlled environments. If wider bounds are used, specifically a higher upper bound, the upper bound factor, R_v , is to be adjusted accordingly.

Add Section C14.5.2 as follows:

C14.5.2 Requirements for Post-tensioned Timber Rocking Walls

C14.5.2.1 General

C14.5.2.1.1 Scope

Requirements for post-tensioned timber rocking walls (PTRWs) are based on emulation of post-tensioned precast concrete shear walls, as modified by timber-specific design aspects. PTRWs and post-tensioned precast concrete shear walls both respond via a reliable rocking mechanism with a generally strong and stiff panel. Design procedures for post-tensioned precast concrete shear walls are available in ACI ITG-5.1 and ITG-5.2 from which the requirements of this section are adapted. Acceptable performance of PTRWs has been demonstrated through extensive research, testing, design studies, and built projects (Palermo et al. 2006, Iqbal et al. 2007, Sarti et al. 2016a, Sarti et al. 2016b, Ganey et al. 2017, Zimmerman and McDonnell 2017, Baas et al. 2019, Pei et al. 2019, Chen et al. 2020, Amer et al. 2023, Pei et al. 2024).

PTRWs can be in uncoupled or coupled configurations subject to the requirements of Section 14.5.2.3. Typical uncoupled and coupled configurations are shown in Figure C14.5.2.1.

While these requirements pertain to strength and serviceability design in accordance with the NDS, this Standard, and the applicable building code, other considerations for the design of PTRWs such as service-level wind accelerations and deflections, especially for taller buildings, may need to be considered.

C14.5.2.1.2 Definitions

For a graphical representation of timber panel versus PTRW segment versus PTRW, see Figure C14.5.2.1.

C14.5.2.2 Materials and Components

C14.5.2.2.1 Timber Panels

Two timber panel types are permitted in this section: structural composite lumber and cross-laminated timber, each meeting the requirements of the appropriate chapter in the NDS.

Integral to the design of PTRWs is establishing the timber panel edgewise compression stress-strain relationship (including the maximum permissible compressive strain, ϵ_{mtmax}) and the inelastic compression zone height, h_c . Since the product of ϵ_{mtmax} and h_c equal an inelastic deformation capacity of the timber panel toe in compression, these two parameters should be considered simultaneously. The provisions provide two paths for establishing ϵ_{mtmax} and h_c .

The first path provides default values only applicable to cross-laminated timber. The default value of ε_{mtmax} was based on comparing an assumed elastically-perfectly plastic stress-strain relationship to experimentally-determined stress-strain curves (Akbas et al. 2017, He et al. 2018, Kaliyanda et al. 2019, Amer et al. 2023, Sinha et al. 2024). An elastically-perfectly plastic simplified stress-strain curve assumption out to three times the “yield” strain, where the “yield” strain was calculated as the NDS strength of the vertical laminations divided by the NDS elastic modulus of the vertical laminations in the CLT, was shown to be conservative compared to the experimentally-determined stress-strain curves. This conservatism is a result of both underestimation of tested strength by the NDS strength value as well as ductile, though with gradual loss of strength with increasing strain, post-peak crushing of wood. While more limited research currently exists on an appropriate h_c for timber panels (Ganey 2017, Wichman 2023, Sinha et al. 2024), a value of twice the thickness of the PTRW segment is paired with the simplified stress-strain curve to establish default assumptions.

The second path for establishing ε_{mtmax} and h_c provides alternate values based on edgewise testing of timber panels in uniform compression. This path is applicable to either structural composite lumber or cross-laminated timber. For structural composite lumber, it is the only acceptable path; for cross-laminated timber, it may be pursued where less conservative timber compression properties are sought than the default values provided in the first path. Specimens for such testing may be of the size and scale tested in Sinha et al. 2024 and may include approximately three identical specimens to assess variability. Quasi-static, monotonic testing is expected. A strain value at 80-percent of the peak stress (20-percent loss of peak stress) is used for ε_{mtmax} as a reasonable measure of substantial initiation of strength loss. The crushing zone height, h_c , would be determined as the length over which post-peak deformation localizes as judged by visual observation. It is known that h_c varies based on stress state and configuration of the wall toe. Furthermore, it is known that conservative estimates of h_c can impact the economy of PTRW designs. However, given the limitation of current experimental data, the provisions do not allow the value of h_c to exceed two times the thickness of the timber panel. It is critical that both ε_{mtmax} and h_c be determined at the same point in the test. This ensures that the parameters are mutually consistent since they are integrally tied together in the design procedure.

C14.5.2.2.2 Timber Bounding Columns

Timber bounding columns may or may not be present depending on the PTRW configuration selected. See Figure C14.5.2.1.

C14.5.2.2.3 Post-Tensioning Components

Post-tensioning bars are required to be unbonded to prevent concentration of strain due to axial elongation from gap opening at the rocking interface. This section provides a prescriptively permitted list of ASTM standards for bars.

The requirement that post-tensioning components be threaded over their full length is to ensure that post-tensioning yielding is distributed over its full length. For example, if threads are cut only at the locations of couplers or nuts and the remaining portion of a rod is left unthreaded, when the post-

tensioning component yields, the inelastic strain will concentrate in the short length of the threads and the post-tensioning component may prematurely fracture. The exception permits a case where yielding of the unthreaded portion, which corresponds to essentially the full length except for at connections and couplers, will occur because the root area of the threaded portion has equal or greater capacity to the unthreaded portion. An example that meets this exception would be threading of an upset-end rod.

Kinking of post-tensioning components could lead to premature fracture. Since the post-tensioning components provide a significant contribution to the total strength of the system, premature fracture of them would result in significant loss of system lateral strength. Kinking of post-tensioning components can occur where details localize the rotation/flexural demands on post-tensioning components to a single point. While it is not possible to eliminate some rotation/flexural demands on post-tensioning components due to story drift and PTRW rocking (except through ball joints or other mechanisms which are not recommended), details should spread such rotation/flexural demands out over a sufficient length of the post-tensioning component to keep curvatures in the post-tensioning components from localizing into a kink.

Long-term post-tensioning losses in timber is an area of on-going research. Refer to Section C14.5.2.7.13. Therefore, a requirement exists to provide access for potential re-tensioning. For example, this requirement could be met by providing hatches at the roof to access the tops of post-tensioning components for re-tensioning operations.

The requirements of Table 14.5.2.2.2 are taken from ACI 318-19 Table 20.3.2.5.1. For structures subject to corrosive conditions, consideration should be given to limiting the final stress.

Care should be taken to avoid post-tensioning force transfer through energy dissipation components during tensioning since this adds uncertainty to the actual post-tensioning force present in the timber panels (e.g., in configurations with energy dissipation components between timber panels and bounding columns) as well as the initial force in the energy dissipation components. Such uncertainty may affect the expected capacity of the rocking mechanism and the progression of yielding in the energy dissipation components.

C14.5.2.2.4 Energy Dissipation Components

These requirements define three types of energy dissipation components: U-shaped Flexural Plates (UFPs), buckling-restrained braces (BRBs), and all others. Design provisions for UFPs are provided in Section 14.5.2.8. Acceptance criteria for BRBs based on testing in accordance with ANSI/AISC 341 are adopted in this section in lieu of recreating acceptance criteria and testing requirements in this Standard. Energy dissipation components other than UFPs or BRBs are permitted when tested in accordance with the prototype testing procedures of Chapter 18. Testing in accordance with the prototype testing procedures of Chapter 18 does not imply that the other requirements of Chapter 18 apply (e.g., upper- and lower-bound properties of energy dissipation components, independent review, etc.).

Discussion of fatigue considerations for energy dissipation components under wind loading is discussed in Section C14.5.2.6.

C14.5.2.3 System Requirements

C14.5.2.3.1 LFRS Configuration

The requirements in this section limit the configurations of PTRWs that can be used when applying this section. Typical uncoupled and coupled configurations are shown in Figure C14.5.2.1. The limitations exist for the following reasons:

1. PTRWs with more than one rocking interface have not been studied sufficiently for incorporation into a prescriptive method.
2. The presence of rocking interfaces at different elevations for different PTRWs creates a deformation incompatibility when the PTRWs rock that cannot be captured by the analysis procedures of this section. This requirement only applies to all PTRWs within the same building; PTRWs in two, structurally separate buildings may have their rocking interfaces placed at different elevations so long as all PTRWs in each building use the same elevation. Where steps in the floor below the rocking interface occur, particularly at grade, a varying height concrete curb wall can be used to set a constant elevation for all PTRWs.
3. This requirement identifies acceptable elements and components where inelastic response may occur in the PTRW structural vertical lateral force-resisting system. Inelastic response outside of the structural vertical lateral force-resisting system covered by this appendix (e.g., at soil-foundation interface) is not restricted where permitted elsewhere in this Standard. Restriction of inelastic response to the timber panels at the rocking interface and the energy dissipation components plus designing all other components to remain essentially elastic ensures that the inelastic mechanism intended by the provisions reliably forms. Requiring post-tensioning components to remain essentially elastic up to and including θ_{design} ensures that the minimum restoring equation of Section 14.5.2.5.6 is valid.
4. PTRW segments in shapes other than a planar PTRW segment (e.g., C- or L-shaped PTRW segments composed of planar timber panels with vertical splices at the corners) are not permitted. Multiple PTRW segments may be arranged to form PTRW configurations such as C- or L-shapes in plan with or without energy dissipation components in-between. However, each PTRW segment must be planar. In non-planar PTRW configurations, energy dissipation elements may impose weak-axis demands on timber panels and bounding columns, which should be considered.
5. PTRW segments in other than balloon-framed construction (e.g., platform-framed construction) do not provide a level of strength and stiffness through interfaces commensurate with the adjacent PTRW segments and are therefore not permitted. However, the exception clarifies that it is acceptable for the topmost level which attaches to the PTRW

segment to frame into and/or bear on the top of the PTRW in a platform-framed condition since no forces are being transferred from above that level. In other words, the typical concerns with the strength and stiffness of a platform-framed condition are not applicable at the topmost level when the conditions of the exception are met.

6. Horizontal splices are required to be located at the same vertical location for each PTRW segment. This requirement prevents a horizontal splice from stepping up/down on either side of a vertical PTRW splice. See Figure C14.5.2.2. Horizontal splices running the full PTRW segment length in the same location allow for the possibility of load sharing/redistribution which may be necessary when estimating the horizontal splice demands from the provisions in this appendix (as opposed to nonlinear analysis). It should be noted that horizontal splices will be necessary when PTRW dimensions exceed those of timber panel maximum dimensions based on fabrication capabilities. In the United States, timber panel dimensions are typically 4ft to 12ft by 16ft to 60ft.
7. Vertical splices are required to be located at the same horizontal location for each PTRW segment. This requirement prevents a vertical splice from jogging over above/below a horizontal splice. See Figure C14.5.2.2. Vertical splices running the full PTRW height in the same location allow for the possibility of load sharing/redistribution which may be necessary when estimating the vertical splice demands from the provisions in this appendix (as opposed to nonlinear analysis).
8. Post-tensioning components and forces must be balanced about each PTRW segment length to avoid initial post-tensioning forces creating imposed, static edgewise bending in the timber panels. Static edgewise bending in a PTRW due to initial post-tensioning would result in lateral deflection of the PTRW.

While the requirements in this section are necessary for use of the prescriptive provisions in this section, the adequacy of PTRWs which take exception to one or more of these limitations can be confirmed through alternate, rational means such as performance-based design.

C14.5.2.3.2 Non-LFRS Configuration

Element and components not part of the lateral force-resisting system need not be considered in contributing to resistance to lateral forces. However, they must be able to accommodate the inter-story drifts. This requirement is also contained in Section 12.12.4 but only for structures in certain Seismic Design Categories. This section enforces deformation compatibility of the non-LFRS components for any structure using PTRWs.

Timber wall panels that are not part of the designated LFRS are not permitted unless they can be demonstrated to not significantly affect the strength or stiffness of the LFRS. This requirement exists to ensure that the analysis and design considering only the LFRS, as is required by this section, is sufficiently accurate. For example, the presence of non-LFRS timber wall panels which significantly affect strength or stiffness could create weak-story, soft-story, torsional, in-plane discontinuity or out-

of-plane offset irregularities. Load bearing timber wall panels not part of the LFRS are assumed to significantly affect the strength or stiffness of the LFRS and are therefore directly prohibited.

Note that the requirements of this section do not preclude the use of other non-LFRS systems (e.g., light-frame wood stud bearing walls) so long as the non-LFRS system selected can be demonstrated to meet the deformation compatibility requirements of this Standard.

Testing for deformation compatibility of glulam beam-column connections which can be used to demonstrate appropriate drift capacity of standardized connections exists in the literature (Williams 2018, Madland et al. 2023a, Madland et al. 2023b, Pryor et al. 2024). Where proprietary connections are used, information on the drift capacity of the connection should be sought from the manufacturer/supplier directly.

C14.5.2.4 Modeling and Analysis

C14.5.2.4.1 General

Modeling and analysis of PTRWs for earthquake loads generally follow the same requirements as for other LFRSs in accordance with Chapter 12. A linear analytical model is developed which incorporates the appropriate extent, component stiffness, and seismic as required by Section 12.7. Then either the equivalent lateral force or the modal response spectrum analysis procedure is selected and applied.

Checks herein are in addition to, not in lieu of, the requirements elsewhere in this Standard. For example, the redundancy factor should still be determined in accordance with Chapter 12. While the redundancy check in Chapter 12 is procedurally assessed through analytical “removal” of a portion of a wall, it should be noted that, for PTRWs, this may actually be representing other actions which could result in loss of lateral load resistance (e.g., loss of one or more post-tensioning components).

With respect to modeling and analysis, the approach of this section is to determine the response of the PTRWs without consideration for the gap opening which occurs across the rocking interface. Instead, the PTRW response is determined for the structure prior to rocking (i.e., essentially elastic response) and then that is modified to represent the true response including rocking (i.e., inelastic response). This approach is described further in Section C14.5.2.5.1.

For modeling and analysis, it is often acceptable to neglect the stiffness contributions of the post-tensioning and energy dissipation components. Post-tensioning components are often located near the center of the PTRW segment length, minimizing their contribution to flexural stiffness. Energy dissipation components (e.g., between PTRW segments in coupled configurations or between timber panels and bounding columns) are often very flexible in comparison to the timber panels. Where there is concern over the additional stiffness provided by either the post-tensioning or energy dissipation components (e.g., where the stiffness of the system increases by more than 10-15% when considering the post-tensioning and/or energy dissipation components), separate analytical models with and without these components can be developed and compared.

C14.5.2.4.2 Effective Stiffness for Linear Modeling

When linear analysis is conducted in accordance with Chapter 12, an analytical model is typically built using the initial, linear stiffness of components without consideration for gap opening due to rocking or post-yield change in stiffness. This linear model is then used to establish force and displacement demands in accordance with Chapter 12 and these provisions which, with modifications, approximate the actual nonlinear response.

The effect of loading direction with respect to grain orientation and lamination is significant for most wood products. For example, parallel- versus perpendicular-to-grain compression, rolling shear, and glue line torsion play a major role in the flexural and shear stiffness of CLT. It is often convenient to use transformed section properties to account for these effects (Chapter 3 of the CLT Handbook, Zimmerman and McDonnell 2018).

The provisions require that the flexibility of vertical and horizontal splices in PTRW segments be considered. However, a high stiffness is typically targeted for vertical and horizontal splices or is achieved as a result of the force design requirements in Sections 14.5.2.7.6 and 14.5.2.7.7, respectively. Note also that any inelasticity of vertical and horizontal splices is prevented by the force design requirements. Where vertical and/or horizontal splices are sufficiently stiff such that their contribution to deflection is minimal, it may be acceptable to neglect their contribution to system flexibility and exclude splice stiffness from the analytical model.

C14.5.2.5 Design for Earthquake Loads

C14.5.2.5.1 Design Rotation at Rocking Interface

The calculation of θ_{design} is based on amplifying the deflection from linear analysis to that expected for nonlinear response of a PTRW. This permits the provisions of this section to be applied without the burden of nonlinear analysis. Instead, it requires the use of a simplified factor, C_1 , which represents the ratio of deflection from a structure responding inelastically to one responding elastically. In these provisions, that factor has been adopted from ASCE 41-17, a standard which also uses this amplification approach for its linear analysis procedures. The value of C_1 increases with ductility (as represented by the response modification factor, R) and decreases with period. Therefore, C_1 is greatest for short-period buildings with large ductility demands. When the fundamental period exceeds 1.0 sec, $C_1 = 1.0$. This is often referred to as the “equal displacement rule” for long-period structures (i.e., an inelastically and elastically responded structure of the same period undergo the same displacement). The value of C_1 also changes with site class through the parameter a which is also adopted directly from ASCE 41-17.

Note that C_1 is not similar to the deflection amplification factor, C_d . C_d multiplies up the deflections due to the earthquake design forces which have been reduced by R whereas C_1 amplifies the deflections due to the earthquake design forces not reduced by R .

In calculating δ_{DE} using the equivalent lateral force procedure, it is important to neglect Equation 12.8-6 as stipulated in the exception to Section 12.8.6.1. Additionally, it is important to use the

computed fundamental period rather than the upper limit on the calculated period as stipulated in Section 12.8.6.2. If either of the above are violated, δ_{DE} and θ_{design} will be overestimated which may skew the remaining calculations in Section 14.5.2.5 (e.g., overestimation of the nominal moment capacity of the rocking mechanism which is unconservative and overestimation of strain demands on energy dissipation and post-tensioning components which is conservative).

After establishing the inelastic displacement demand at the top of the PTRW, $C_1 \times \delta_{DE}$, the design gap opening rotation at the rocking interface, θ_{design} , can be calculated by subtracting out the elastic deflection of the PTRW itself when subject to forces corresponding to the moment capacity of the rocking mechanism, $\delta_{DE,y}$ (i.e., PTRW deflection at initiation of base rocking). Since the forces corresponding to the moment capacity of the rocking mechanism is unknown until the design is complete, the forces corresponding to the design base shear (i.e., earthquake forces reduced by R but considering scaling of modal response spectrum analysis to equivalent lateral force analysis base shear, when required) are used as an approximation. For a diagrammatic derivation of the equation for θ_{design} , see Figure C14.5.2.3.

Since δ_{DE} and $\delta_{DE,y}$ are computed at the plan location of the PTRW, they do not consider diaphragm deflection directly. However, diaphragm flexibility modeling may still be necessary in appropriately determining the distribution of forces to each PTRW, which would affect δ_{DE} and $\delta_{DE,y}$ indirectly.

C14.5.2.5.2 Maximum Rotation at Rocking Interface

The difference between the equations for θ_{max} versus θ_{design} is the incorporation of the factor of 1.5 which represents the ratio of Risk-Targeted Maximum Considered Earthquake to Design Earthquake. The 1.5 factor is applied on δ_{DE} , not on θ_{design} . Therefore, the ratio of θ_{max} to θ_{design} will be less than or equal to 1.5.

C14.5.2.5.3 Nominal Moment Capacity of the Rocking Mechanism

The nominal moment capacity of the rocking mechanism for PTRWs is calculated in much the same way as for a reinforced concrete section subject to combined compression and bending (i.e., a neutral axis is determined, then strains and stresses are calculated considering plane sections remain plane and material stress-strain relationships, then the sectional force and moment are determined based on summing stresses). See Figure C14.5.2.4. However, the moment capacity is determined for the entire rocking mechanism, not simply the section at the rocking interface, which may include distributed energy dissipation components up the PTRW height. Example calculations of the nominal moment capacity of the rocking mechanism can be found in literature (Busch et al. 2022, Wichman 2023). The four requirements of this section include:

1. The use of nominal properties and stress-strain material relationships. It is not the intent that the exact stress-strain curves are required. Rather, simplified stress-strain curves are acceptable. For example, steel energy dissipation components and wood compression may each be represented using a linear elastic perfectly plastic relationship if such a relationship can be reasonably fit to the experimentally derived curves. Additionally, since the purpose of

calculating the nominal moment capacity of the rocking mechanism is primarily to check against the moment demand in accordance with Section 14.5.2.7.2, it is conservative to make assumptions which underestimate the nominal moment capacity of the rocking mechanism (e.g., neglecting strain hardening in energy dissipation components).

2. Plane sections remain plane in compression such that strain is linearly proportional to distance from the neutral axis. This assumption is the basis of many other provisions in the NDS (e.g., calculation of flexural section modulus, S). Since θ_{design} is calculated by the procedures of Section 14.5.2.5.1, it must be converted to a curvature for use in sectional analysis in this section by dividing by an effective crushing zone height, h_c .
3. Gap opening in uplift is linearly proportional to distance from the neutral axis. This assumption is similar to that described in Item #2 above but for the uplift/tension side of the section.
4. Displacements/deformations need to consider the combined effect of θ_{design} at the rocking interface and the timber panel flexibility itself. This is necessary since the nominal moment capacity of the rocking mechanism can often include contributions from components not immediately at the rocking interface (e.g., energy dissipation components distributed up the building height as shown in Figure C14.5.2.1 for the uncoupled configuration with bounding columns). For such cases, the displacements/strains and resulting forces/stresses in these components will not simply be related to θ_{design} and the distance from the neutral axis. Rather, the displacement due to the timber panel flexibility itself needs to be included. This requirement is essentially ensuring that the displacements/strains and resulting forces/stresses in all components are consistent with the displaced shape shown in Figure C14.5.2.3 which differs from the displaced shape from analysis.

C14.5.2.5.4 Probable Moment Capacity of the Rocking Mechanism

The probable moment capacity of the rocking mechanism is intended to reflect an upper-bound strength at displacements corresponding to the Risk Targeted Maximum Considered Earthquake (MCE_R). It is calculated using similar assumptions as described in Section 14.5.2.5.3 for the nominal moment capacity of the rocking mechanism except for the following:

- The use of expected material properties and stress-strain relationships instead of nominal. For steel components, the use of R_y and R_u in accordance with ANSI/AISC 341 may be used to convert between nominal and expected yield and ultimate stress, respectively. Additionally, since the purpose of calculating the probable moment capacity of the rocking mechanism is primarily to establish the overstrength factor in accordance with Section 14.5.2.7.3, it is conservative to make assumptions which overestimate the probable moment capacity of the rocking mechanism (e.g., assuming all energy dissipation components are resisting force equal to their expected ultimate capacity instead of attempting to determine their degree of strain hardening).

- The use of θ_{max} instead of θ_{design} to reflect the greater gap opening at the MCE_R than the DE earthquake.

Item 4 of Sections 14.5.2.5.3 and 14.5.2.5.4 are identical. Although the increased moment capacity of the rocking mechanism at the MCE_R compared to the DE does result in increased PTRW segment displacements above the rocking interface (approximately equal to the ratio of probable to nominal moment capacity), this effect is neglected for simplicity in the provisions. Furthermore, while the timber panel stiffness may differ between DE and MCE_R levels, the change is expected to be small and likely appropriately neglected given the other, more gross assumptions made in estimating the probable moment capacity.

C14.5.2.5.5 Component Limits

Limits on timber panels at the rocking interface, post-tensioning components, and energy dissipation components are included at both the design earthquake (DE) per Section 14.5.2.5.3 and the Risk-Targeted Maximum Considered Earthquake (MCE_R) or probable capacity per Section 14.5.2.5.4. At the DE, the strain in the timber panel at the rocking interface cannot exceed the maximum useable strain of the timber panel. Exceeding that strain would result in unacceptably large amounts of damage in the timber panel and potential for significant strength degradation with cycling during the earthquake. Additionally, the post-tensioning components should not yield at the DE. Yielding of the post-tensioning components precludes re-centering and makes the minimum restoring force equation of Section 14.5.2.5.6 invalid. Stresses in post-tensioning components are calculated considering (a) the initial prestress, considering the requirements of Section 14.5.2.7.13, and (b) the stress induced by deformations associated with PTRW rocking at the rocking interface as well as the elastic flexibility of the PTRW above the rocking interface, the latter two effects which are calculated in accordance with Section 14.5.2.5.3.

At the MCE_R or the probable capacity of the PTRW, the energy dissipation and post-tensioning components must not fracture. For post-tensioning components, this is ensured by requiring that the strain stay below 85-percent of the strain at tensile strength. ACI ITG-5.2-09 Section 5.6.3.3, which as described in Section C14.5.2.1.1 forms the basis of these provisions, has a similar requirement when designing post-tensioned precast concrete shear walls. Prevention of fracture for energy dissipation components is ensured through design procedures provided in Section 14.5.2.8 (for UFPs) and qualification testing (for BRBs and other energy dissipation components).

C14.5.2.5.6 Minimum Restoring

The minimum restoring provisions exist so that each PTRW re-centers after the design earthquake (DE). They are also predicated on the post-tensioning components not yielding at DE as required by Section 14.5.2.5.5. Re-centering for earthquakes beyond the DE is not ensured by these provisions. The requirements of this section are based on those of ACI ITG-5.2-09 Section 5.3.

The equation in this section is derived based on summing moments about the PTRW segment toe for typical configurations (i.e., post-tensioning components and gravity loads at center of PTRW segment

length and energy dissipation components at PTRW segment edges). An exception is provided where geometry differs from that assumed. For example, if all energy dissipation components are located at the center of the PTRW segment length, $l_{w,j}$ becomes $l_{w,j}/2$ on the right-hand side of the equation.

Only the energy dissipation components not at the toe end of the PTRW segment are considered. As described above in deriving the equation, energy dissipation components at the toe end of the PTRW segment produce no moment about the toe of the PTRW segment. For the two-panel coupled configuration of Section C14.5.2.1, the portion of the summation on the right-hand side of the equation that includes the trailing PTRW segment will be zero since all of the energy dissipation components for that PTRW segment are located at the toe end of the PTRW.

Note that placing energy dissipation components at both edges of a PTRW segment versus concentrating those components at the PTRW segment centerline does not change satisfaction of the minimum restoring moment provisions for uncoupled single panel configurations. In the latter case, the right-hand side of the equation has twice the force (i.e., all energy dissipation components are not at the PTRW segment toe instead of half) but half the lever arm (i.e., located at PTRW segment center versus full PTRW segment length).

The following assumptions are made to derive a simplified form of the equation in this section. These assumptions are fairly consistent with ACI ITG-5.2-09 Section 5.3.

- Contribution to moment of wood in compression as shown in Figure C14.5.2.4 is neglected. This can be unconservative where the wood compression zone is a significant proportion of the PTRW segment length but, for typical designs, is considered a sufficient assumption for this check.
- Post-tensioning components forces are taken at initial post-tensioning (i.e., excluding the additional forces generated due to PTRW gap opening at the rocking interface). This is conservative where additional force in post-tensioning components due to gap opening at the rocking interface is significant.
- Forces in the energy dissipation components are taken at nominal yield level. This may be unconservative where strain hardening in energy dissipation components is significant.

It is noted that even systems which do not meet the minimum restoring moment requirements of this section may self-center sufficiently due to other phenomena like “shake-down” which occurs after the high intensity portion of the earthquake, restoring provided by the gravity system, etc. This helps justify several of the potentially unconservative assumptions made in deriving a simple equation.

C14.5.2.5.7 Minimum Energy Dissipation

Minimum energy dissipation is required to prevent the design of an essentially elastic PTRW system. Essentially elastic systems tend to be more sensitive to ground motion variability and may result in unconservative designs without further modifications not included in these provisions. A PTRW without sufficient energy dissipation components would be considered an essentially elastic, albeit

nonlinear, system (i.e., stiffness changes with deformation but loading and unloading follows the same path such that negligible energy is dissipated). The limit for minimum energy dissipation (i.e., energy dissipation contribution greater than one-eighth of the nominal moment capacity) was validated as part of the FEMA P-695 study of PTRWs (Zimmerman et al. 2026).

Because establishing the contribution to the nominal moment capacity of the rocking mechanism from energy dissipation components directly is not straightforward for all PTRW configurations, the provisions allow it to be determined based on the difference between the nominal moment capacity and the nominal moment capacity neglecting the energy dissipation components. The latter represents the contribution of post-tensioning components and gravity load and can be calculated following the procedures of Section 14.5.2.5.3 but for a hypothetically identical PTRW except without the energy dissipation components present.

C14.5.2.6 Design for Wind Loads

While other non-seismic and non-wind lateral loads may act on PTRWs, it is assumed that they are secondary to or considered in load combinations with either seismic or wind lateral loads. For example, where flood loads may act on PTRWs, they would act in combination with wind loads and thus the design procedures of this section would be followed.

C14.5.2.6.1 Nominal Moment Capacity of the Rocking Mechanism

The nominal moment capacity of the rocking mechanism for PTRWs for wind loads is taken less than the nominal moment capacity of the rocking mechanism for earthquake loads. This is a result of the fact that greater gap opening rotation is permitted under earthquake than wind loading. For wind loading, the nominal moment capacity of the rocking mechanism represents the capacity prior to large gap opening as represented by (a) the requirement to keep the wood in compression essentially elastic (i.e., triangular stress distribution) and (b) the requirement to use the initial post-tensioning force (i.e., no additional elongation due to gap opening permitted to be used even though some may occur). Forces in energy dissipation components are permitted to be taken equal to their nominal strengths to reflect the fact that the mobilized strength of the PTRW for wind loads would be realized with the energy dissipation components yielding.

Compared to seismic loading, wind loading involves many more cycles of force. However, forces are generally at or below yield for almost all elements and components (e.g., timber panels, post-tensioning, etc.), with the exception of energy dissipation components. Energy dissipation components are permitted to yield under ultimate wind loads but corresponding displacements are expected to be much less than for seismic loading. Thus, fatigue of energy dissipation components under wind loading is assumed to be addressed by the use of energy dissipation components which have been tested for more onerous seismic loading. For example, the large fatigue life of U-shaped Flexural Plates (UFPs) has been extensively experimentally studied (Deng et al. 2013, Jiao et al. 2015, Gonzalez et al. 2025).

Note that, although not typically required by the governing building code, accelerations and drifts under service-level wind events may need to be considered in addition to providing sufficient

strength for wind loading. Such checks may include limiting wind drifts to less than $1/400$ to $1/600$ of the story height under service-level wind events. Service-level wind events are often taken between 10-year and 25-year mean recurrence interval events. Additional information on service-level wind considerations can be found in ASCE 7 Appendix CC.

C14.5.2.6.2 Component Limits

Compressive stress in the wood at the rocking interface is required to remain less than its nominal value for wind loading.

C14.5.2.7 Strength and Serviceability

C14.5.2.7.1 General

The requirements of this section amend the requirements contained in Chapter 12 for earthquake loads. Unless explicitly stated, the requirements of this section do not permit exceptions to Chapter 12.

C14.5.2.7.2 Flexural Strength of the Rocking Mechanism

While the NDS often characterizes strength acceptability in terms of either allowable stress design (ASD) versus load and resistance factor design (LRFD) and in terms of either stress versus sectional strength, this section requires that LRFD and sectional strength be used in lieu of ASD and stresses. This is based on the rationale that the rocking mechanism (i.e., a sectional strength) is the controlling mode of behavior rather than any individual material stress, although individual material stress and strain limits are required by the provisions.

The capacity of the rocking mechanism is not just determined from the component strengths at the rocking interface. Rather, in the case where energy dissipation components are distributed up the height of PTRW (e.g., uncoupled single-panel configuration with bounding columns), the capacity of the rocking mechanism includes all components which deform when rocking occurs at the rocking interface. Because of this, the demand of the rocking mechanism must be carefully extracted from analysis software, especially where coupled panel configurations are used, so as to include forces generated at the rocking interface as well as in energy dissipation components not at the rocking interface if such energy dissipation components are explicitly modeled. For greater discussion on consideration of energy dissipation components in analytical models, see Section C14.5.2.4.1.

C14.5.2.7.3 Amplification of Forces and Moments

Amplification is not required to be considered for wind loading since significant inelastic action is not expected under wind loads.

Amplification of shear and moment for earthquake loads in structures which respond in cantilever-mode (e.g., tall concrete shear walls, spine and rocking systems, etc.) is well known. Amplification is typically measured as the increase in expected demands compared to demands resulting from a

code-prescribed equivalent lateral force or modal response spectrum analysis. Amplification is generally largest near mid-height of the building. See Figure C14.5.2.5. The amplification is generally a result of two phenomena:

1. Overstrength at the rocking interface. This results directly from the design process which requires that the capacity at the rocking interface exceeds the code-prescribed demand by a resistance factor. Furthermore, nominal material properties are assumed in calculating the capacity. Therefore, this overstrength can amount to, at a minimum, the product of (a) the demand-capacity ratio at the rocking interface (i.e., overdesign), (b) the inverse of the resistance factor, and (c) the ratio of capacity determined based on expected properties to that determined based on nominal properties.
2. Reduced inelasticity in higher modes as compared to code-prescribed procedures. In the equivalent lateral force procedure or the modal response spectrum procedure performed in accordance with this Standard, the entirety of the seismic forces is reduced by the response modification coefficient, R . However, it is known that, for taller and more flexible structures, the second, third, and higher modes contribute significantly to force demands. Yet, because hinging or rocking at the base does not greatly affect the second, third, and higher modes, the force reduction assumed by R does not occur in these modes. Therefore, these modes respond more as if they were not reduced by R even though the code-prescribed procedures assume they are. As a result, forces generated from the higher modes exceed those in the code-prescribed procedures.

In these provisions, overstrength in the rocking mechanism is quantified by ω_b (i.e., the first phenomena described above). The second phenomena (i.e., elastically responding higher modes) is addressed through superimposing additional forces which represent the higher modes on top of the conventional forces otherwise used for design.

Amplification of horizontal earthquake forces is achieved by replacing the horizontal seismic load typically used in this Standard's load combination, E_h , with an amplified horizontal seismic load, $E_{h,amp}$. The amplified horizontal seismic load, $E_{h,amp}$, is similar to horizontal seismic loads including overstrength used in this Standard except that, where horizontal seismic loads including overstrength simply scale up the magnitude of the horizontal seismic loads but retain their general shape with height, the amplified horizontal seismic load, $E_{h,amp}$, has both a different magnitude and shape compared to E_h .

The equation for $E_{h,amp}$ explicitly addresses the two phenomena discussed above, namely overstrength of the rocking mechanism and reduced inelasticity in higher modes. The equation takes the form of a modified modal combination or modified square root sum-of-the-squares (SRSS) procedure.

1. The first term in the equation addresses overstrength by taking this Standard's seismic effects, Q_E , (e.g., shear, moment, etc. resulting from conventional equivalent lateral force or

response spectrum analysis) and multiplying them by ω_b . Note that Q_E taken alone is what is used to design the rocking mechanism in accordance with Section 14.5.2.7.2.

2. The second term in the equation includes a factor of 2.5 times an additional seismic effect. This seismic effect represents the approximate elastic response of the second mode at the design earthquake level. The equation for approximating horizontal seismic forces for the second mode of a cantilever wall that is rocking at its base is taken from Steele and Wiebe (2016). This horizontal seismic force can be applied to an analytical model to determine Q_{hm} . The factor of 2.5 is based on calibration using FEMA P-695 to ensure a sufficiently low likelihood of failure of brittle actions (i.e., non-simulated collapse).

Recognizing that the procedure for amplification of forces and moments is non-trivial for application by the design engineer, a simplified approach is provided in Exception 1 for shorter buildings where higher mode amplification is not as significant. This simplified approach does not require the use of additional forces to approximate the higher modes. Instead, a single amplification factor equal to $2 \times \omega_b$ may be applied to this Standard's seismic effects, Q_E . However, when this exception is invoked, the largest value of $2 \times \omega_b \times Q_E$ at any point up the height must be used for all locations for certain design checks. For example, for checking timber panel in-plane shear or combined edgewise bending and axial loading, the largest value of shear or moment, respectively, occurring at any story must be used to check all stories. Since the largest shear and moment occur at the base, this means that the PTRW segment dimensions and grade/strength would need to remain constant up the full height of the building. This requirement is necessary since the amplification value of $2 \times \omega_b$ was determined to capture the maximum value up the height as a ratio to the base moment or shear. Due to higher modes, the maximum value can occur other than at the base. Therefore, without this requirement, a PTRW segment could be shortened/thinned up the height in a building design exactly where the critical section might be. For a more realistic estimate of the shape of amplified seismic load effects, the general procedure of this section, rather than Exception 1, is recommended.

Since it is acknowledged that the general procedure of this section is not the only method for approximating amplification of seismic loads, Exception 2 provides the design engineer with a path to use another rational analysis that explicitly addresses the two phenomena discussed above.

Note that other standards have taken a more prescriptive approach to consideration of amplification of forces. For example, ACI 318-19 Section 18.10.3 establishes simplified amplification factors only for shear in reinforced concrete shear walls. The provisions here instead allow more freedom to determine amplification of seismic forces via a general method with exceptions but require that amplification be performed for not just PTRW shear but other force demands. This is because for PTRWs, unlike special reinforced concrete shear walls, wall shear is not the only non-ductile mechanism; PTRW segment shear and flexure, horizontal and vertical splices, etc. can all exhibit non-ductile response and are therefore susceptible to increased forces due to amplification effects.

C14.5.2.7.4 Timber Panel In-Plane Shear

The timber panel must be capable of resisting amplified in-plane shear demands. The three-halves factor in the equation adjusts between the average shear stress and the maximum shear stress assuming a parabolic distribution of shear over the cross-section depth akin to the three-halves factor in NDS Equation 3.4-2. The check of shear stress versus strength then follows NDS procedures.

Shear capacity of a timber panel composed of CLT is available from some manufacturers via testing in accordance with PRG-320. General equations for shear capacity of CLT are also available in the literature (Flaig and Blass 2013).

C14.5.2.7.5 Timber Panel Combined Edgewise Bending and Axial Loading

The timber panel must be capable of resisting amplified edgewise bending and axial loading demands. Design checks are separated for timber panels composed of structural composite lumber for which design stresses are available in accordance with NDS Chapter 8 and for timber panels composed of cross-laminated timber for which design section resultant forces are, for the most part, available in accordance with NDS Chapter 10.

Checks are required to be performed for instances where the extreme fiber stress is both tensile and compressive. For sections where tensile stresses occur, Equations 14.5-11 and 14.5-12 are adapted from NDS Equation 3.9-2. For sections where compressive stresses occur:

- Equation 14.5-13 is adapted from NDS Equation 3.9-3.
- Equation 14.5-14 is a generalized version of the F_{CE1} equation in NDS Section 3.9.2 and is derived from the U.S. CLT Handbook Chapter 3 Section 2.2.2.
- Equation 14.5-15 is a generalized version of NDS Equation 3.7-1 and is derived from the U.S. CLT Handbook Chapter 3 Section 2.2.2.
- Equation 14.5-16 is a generalized version of the F_{bE} equation in NDS Sections 3.3.3.6 and 3.9.2 and is derived from AWC Technical Report 14 Equation B10 (AWC 2003).

With the modified equations in these provisions, combined edgewise bending and axial loading can be checked for the timber panel using the referenced NDS procedures commonly applied to a sawn lumber or glulam column. In some cases for CLT, the equations in this section require the use the laminate stress properties (e.g., F_b) listed in the manufacturer's report or PRG-320 rather than the composite sectional properties (e.g., $F_b(S_{eff})$) from NDS Chapter 10.

C14.5.2.7.6 Timber Panel Horizontal Splice

Horizontal splices in timber panels must be capable of resisting amplified shear and moment demands. The provisions require that the shear and moment demands for splices be taken as the larger of the forces at the splice location and 50% of the maximum forces at any story. The latter

means that 50% of the maximum PTRW segment shear and 50% of the maximum PTRW segment moment must be applied at every story. This was deemed necessary since, as seen in the FEMA P-695 study of PTRWs (Zimmerman et al. 2026), the $E_{h,amp}$ forces can underestimate shear and moment away from the peak value (e.g., in the uppermost stories) under collapse-level ground motions. At design- and MCE_R -level ground motions, the $E_{h,amp}$ forces were shown to predict shear and moment demands from nonlinear response history analysis better than at collapse level. Underestimation of splice demands under collapse-level ground motions leads to undesirable performance (i.e., splice failure). Hence, the requirement to design splices for no less than 50% of the maximum forces at any story is included.

Horizontal splices are often made with screwed external plates or with glued-in rods. A sectional analysis under the assumption of plane sections remaining plane plus appropriate material stress-strain relationships can be used to determine the bending capacity of horizontal splices. For an example, see Zimmerman and McDonnell 2018. See Section C14.5.2.4.2 for discussion of horizontal splice flexibility.

C14.5.2.7.7 Timber Panel Vertical Splice

Vertical splices in timber panels must be capable of resisting amplified shear demands. Vertical splices are often screwed half-lap connections. See Section C14.5.2.4.2 for discussion of vertical splice flexibility.

C14.5.2.7.8 PTRW Segment In-Plane Shear Connection at the Rocking Interface

The connection of the PTRW segment at the rocking interface must be capable of resisting sliding including amplification of forces. This is typically configured with shear keys, either at each end or in a notch in the center of the PTRW segment. An example of shear keys at each end of a PTRW segment is provided in Pei et al. (2024). While the use of friction at the rocking interface to resist a portion of the shear may be justified, it is cautioned against relying completely on friction to resist the full sliding shear demand due to the dynamic nature of earthquake shaking and PTRW rocking.

C14.5.2.7.9 PTRW Segment to Diaphragm Connection

PTRW segment to diaphragm connections must be capable of resisting amplified forces resulting from the maximum of the difference in shear in the PTRW segment above and below the diaphragm under lateral forces and the diaphragm forces. It is noted that this Standard requires force amplification for collectors in the higher Seismic Design Categories via the overstrength factor, Ω_o . Since this amplification and the amplification factor from these provisions address some of the same sources of amplification, only the larger (not simultaneous application) of the two needs to be considered.

When CLT diaphragms are used, the provisions of Section 4.5 are also applicable. In such cases, the WoodWork's *CLT Diaphragm Design Guide* (Breneman et al. 2023) may be useful for designing the PTRW segment to diaphragm connection.

PTRW segment to diaphragm connections which cause uplift of the diaphragm as the PTRW rocks should consider the kinematics and corresponding forces which may result from such movement when designing the PTRW segment to diaphragm connection. Alternatively, PTRW segment to diaphragm connections which allow free uplift and rotation of the PTRW with respect to the diaphragm while simultaneously transferring the horizontal shear may be detailed.

C14.5.2.7.10 Bounding Columns

The exception here is similar to the approach used for the design of columns in other high ductility seismic force-resisting systems (e.g., buckling-restrained braced frames) where the force demands are determined based on capacity-based forces from the deformation-controlled components in the system. For PTRWs, the probable forces delivered by the energy dissipation components essentially determine the maximum seismic forces on bounding columns (e.g., sum of UFP probable strengths is used to determine the maximum seismic axial force in a bounding column). Note that the exception states that the probable forces from “all energy dissipation components” shall be used. For the example of a corner bounding column at the intersection of two, orthogonal PTRWs, the probable forces delivered by the energy dissipation components in both directions must be used.

C14.5.2.7.11 Post-Tensioning and Energy Dissipation Connections

Post-tensioning and energy dissipation connection forces are required to be determined by the ultimate/probable strength of the post-tensioning or energy dissipation components, respectively. This is necessary to ensure that the connections do not fail before the inelastic action of the post-tensioning and/or energy dissipation components.

C14.5.2.7.12 Other Structural Components and Connections

Other structural components and connections must be capable of resisting forces only amplified by the PTRW bending overstrength factor, not the full amplification corresponding to higher modes. It is assumed that all components and connections significantly affected by higher modes are explicitly addressed in Sections 14.5.2.7.4 through 14.5.2.7.9. Other structural components and connections include those not otherwise identified in previous sections (e.g., foundations).

It is noted that this Standard requires force amplification for certain components in the higher Seismic Design Categories via the overstrength factor, Ω_o . Since this amplification and the amplification factor from these provisions address some of the same sources of amplification, only the larger (not simultaneous application) of the two needs to be considered.

An exception is provided to allow certain soil and geotechnical actions (e.g., bearing pressures on shallow foundations, geotechnical pile capacities for deep foundations, passive pressure, footing sliding friction, etc.), global overturning, diaphragms, and diaphragm chords to be designed for the design forces without further amplification (i.e., ω_b taken as unity). For soil and geotechnical actions, this reflects the standard-of-practice that geotechnical checks of foundations are not designed for overstrength forces and have their own sources of overstrength/ductility which may balance structural overstrength. However, note that structural checks of foundations (e.g., flexure or shear in

spread footings, axial in piles, etc.) are not included in the exception list and must be capable of resisting forces amplified by the PTRW bending overstrength factor. Exclusion of diaphragms and diaphragm chords acknowledges the standard-of-practice for diaphragm design in this Standard which does not necessarily ensure nor require essentially elastic diaphragm performance. Diaphragm collectors are not included in the exception and are addressed explicitly in Section 14.5.2.7.9.

C14.5.2.7.13 Post-Tensioning Losses

Two cases of post-tensioning stresses are required to be considered. This is necessary to consider the potential long-term loss of post-tensioning due to creep and shrinkage of the wood and relaxation of post-tensioning components and anchorages in addition to the case of initial post-tensioning stress. In general, when post-tensioning stress decreases, the flexural capacity of the rocking mechanism in Section 14.5.2.7.2 decreases and the check of minimum restoring moment of Section 14.5.2.5.6 becomes more difficult. The provisions permit the effect of the creep component of the timber panel on the post-tensioning stress to be determined following AWC NDS provisions for long-term loading. While those provisions are written for bending members, they are permitted to be applied here for timber panels acting primarily in compression. More accurate estimation of the loss in post-tensioning stress can be found in the available literature (Baas et al. 2021, He et al. 2022, Gesh 2023). A set loss may also be prescribed and designed for in concert with an in-situ monitoring and re-tensioning plan.

C14.5.2.8 Design of U-Shaped Flexural Plates

C14.5.2.8.1 General

U-shaped Flexural Plates (UFPs) are a common energy dissipation component used in PTRWs. Explicit provisions are therefore provided. However, any energy dissipation component meeting the requirements of Section 14.5.2.2.3 may be used.

C14.5.2.8.2 Configuration Limits

The ratio of plate thickness to U-shaped Flexural Plate (UFP) bend diameter defines the maximum strain experienced in the UFP during its “rolling” action under relative deformation between one “leg” of the UFP and the other. This strain shall not exceed strain limits intended to prevent fracture which are translated from the requirements of Section 14.5.2.5.5. Note that the geometry of the UFP defines its strain demand after yield rather than the earthquake hazard level.

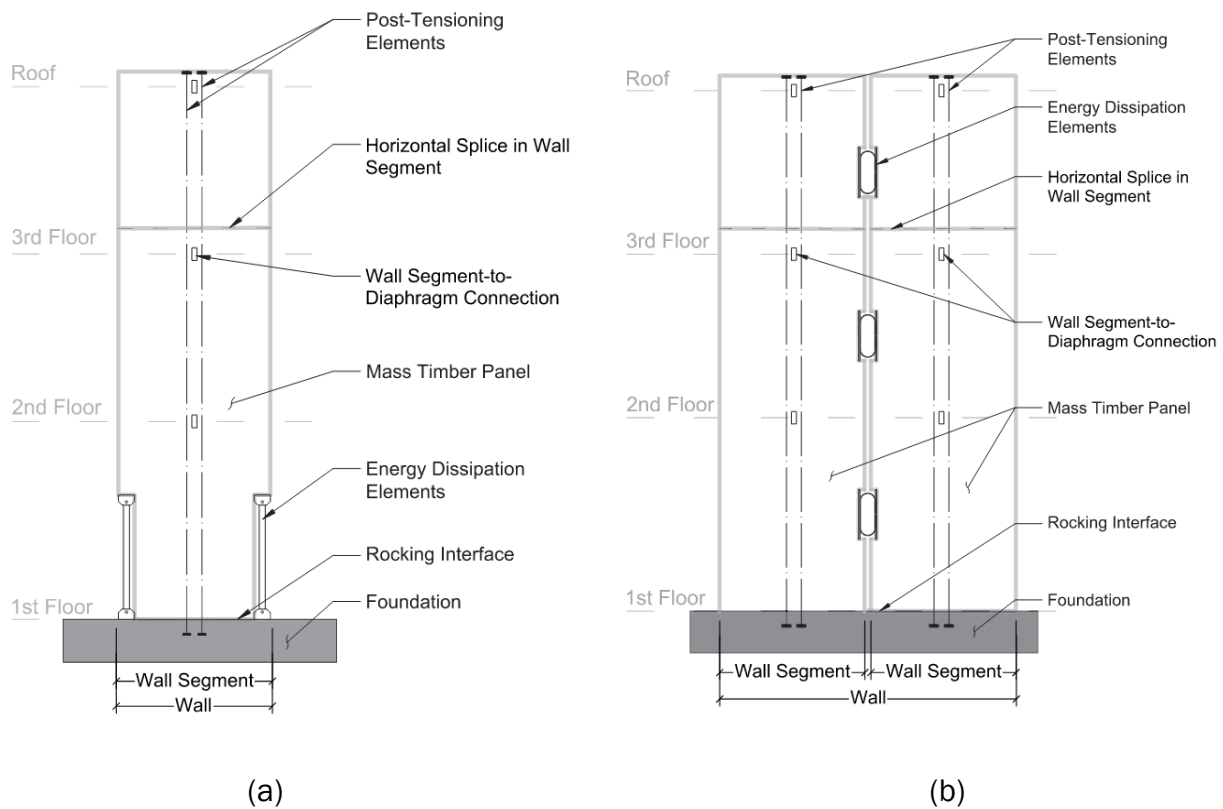
However, the required length between the nearest attachment point and the beginning of the 180-degree bend is dependent on the earthquake hazard level and must be sufficient to allow “roll-out” of the UFP for the maximum considered earthquake. The two-thirds factor in the provisions is determined by multiplying the four-thirds factor from ACI ITG-5.2 Section 7.4 (i.e., four-thirds equals “one-third greater”) for UFPs by one-half. The one-half represents how half of the UFP relative shear displacement is shared with each UFP leg. Sufficient clearance for the full four-thirds of the relative shear displacement at the maximum considered earthquake needs to be provided. To determine

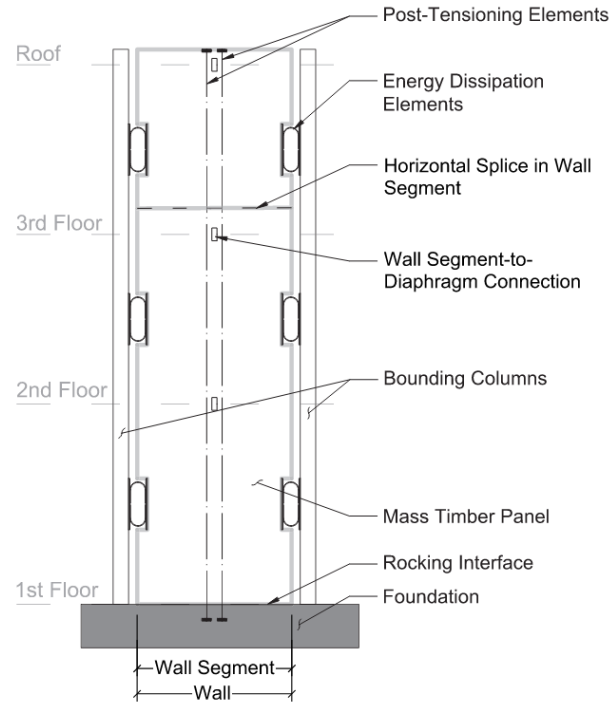
necessary clearances, it is often helpful to draw a deformed shape diagram focused specifically on the UFP area and including PTRW rocking deformations and UFP rolling. See Figure C14.5.2.6.

C14.5.2.8.3 Stiffness and Strength

The stiffness and strength equations for U-shaped Flexural Plates (UFPs) are adapted from Baird et al. 2014.

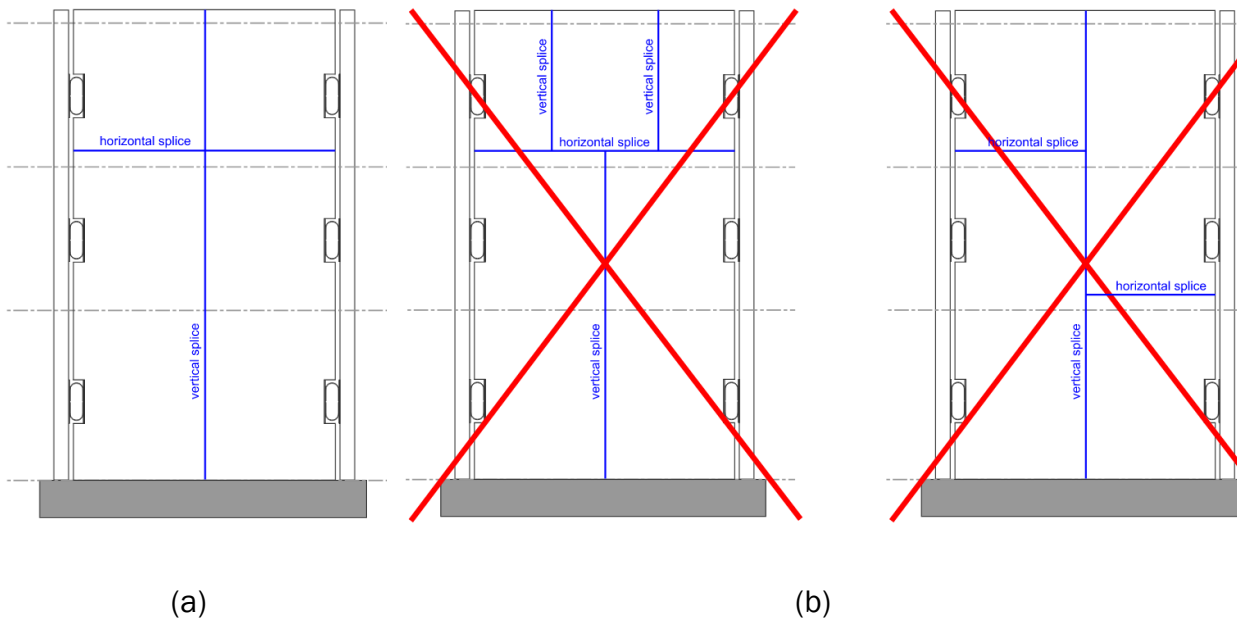
Figure C14.5.2.1 Typical uncoupled and coupled PTRW configurations, a) uncoupled single PTRW segment configuration, b) two PTRW segment coupled configuration, and c) uncoupled single PTRW segment configuration with bounding columns. Figures adapted from Busch et al. 2022.





(c)

Figure C14.5.2.2 Example of horizontal and vertical splice locations (a) permitted by this appendix since horizontal splices align full length of PTRW segment and vertical splices align full height, and (b) not permitted by this appendix since horizontal and/or vertical splices do not align.



(a)

(b)

Figure C14.5.2.3 Derivation of equation for θ_{design}

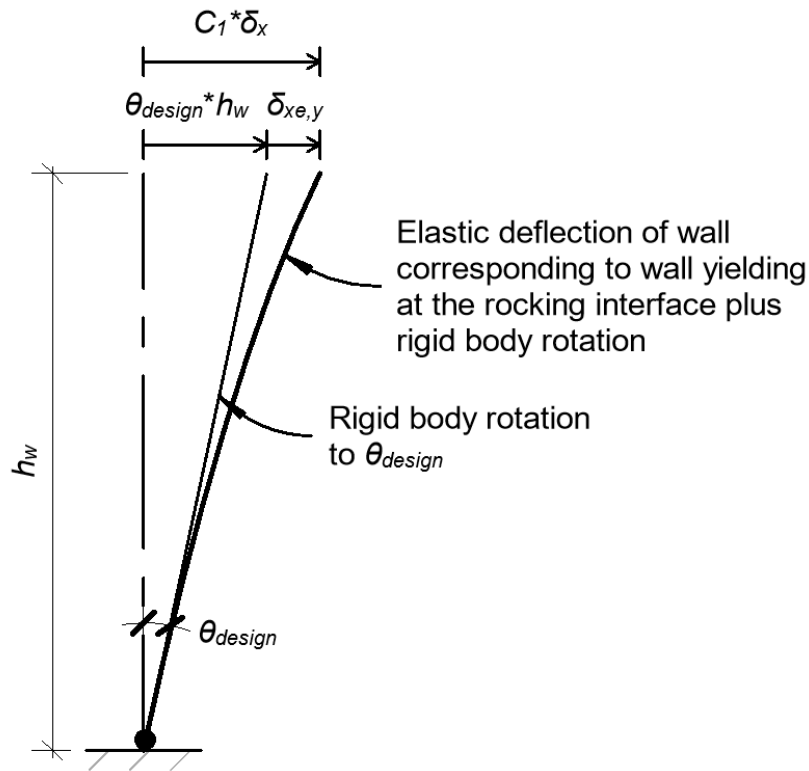


Figure C14.5.2.4 Sectional analysis of rocking interface. Figure adapted from Busch et al. 2022.

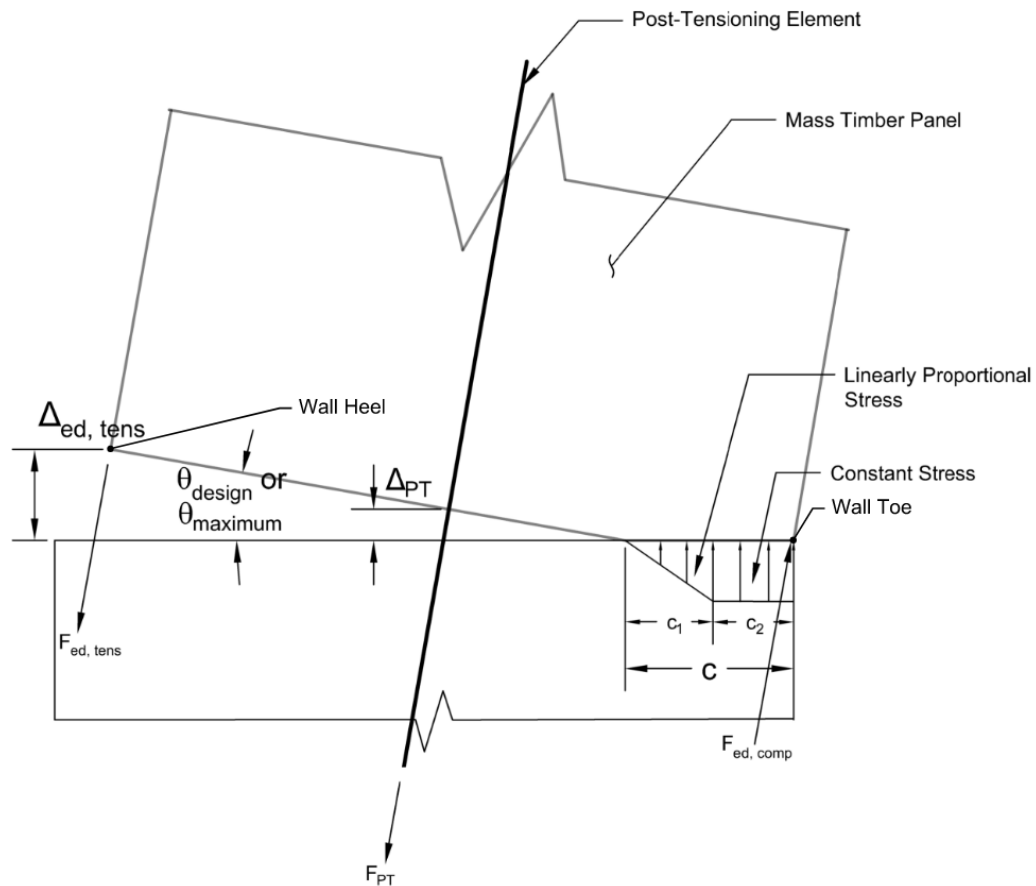


Figure C14.5.2.5 Example of amplification of (a) shears and (b) moments between modal response spectrum and nonlinear response history analysis for a 10-story PTRW building

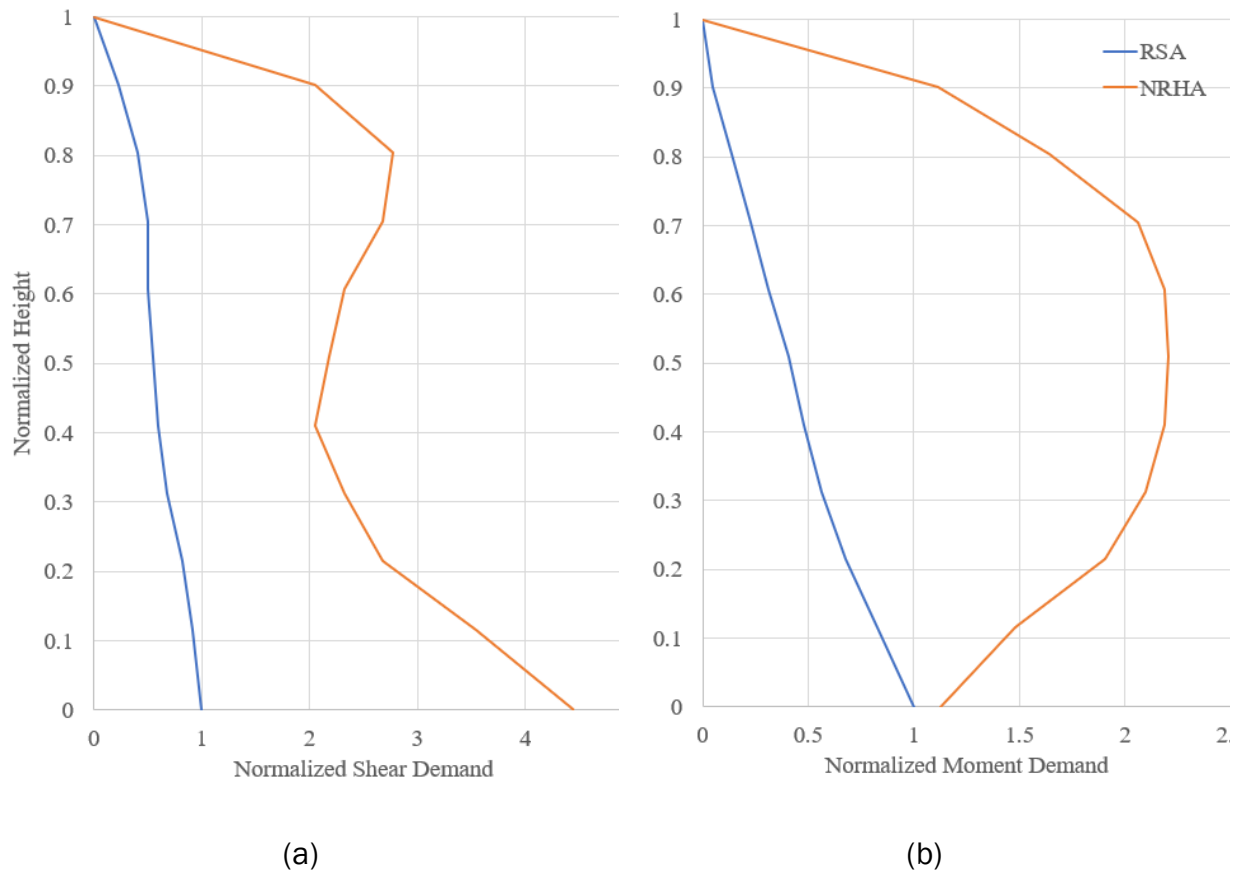
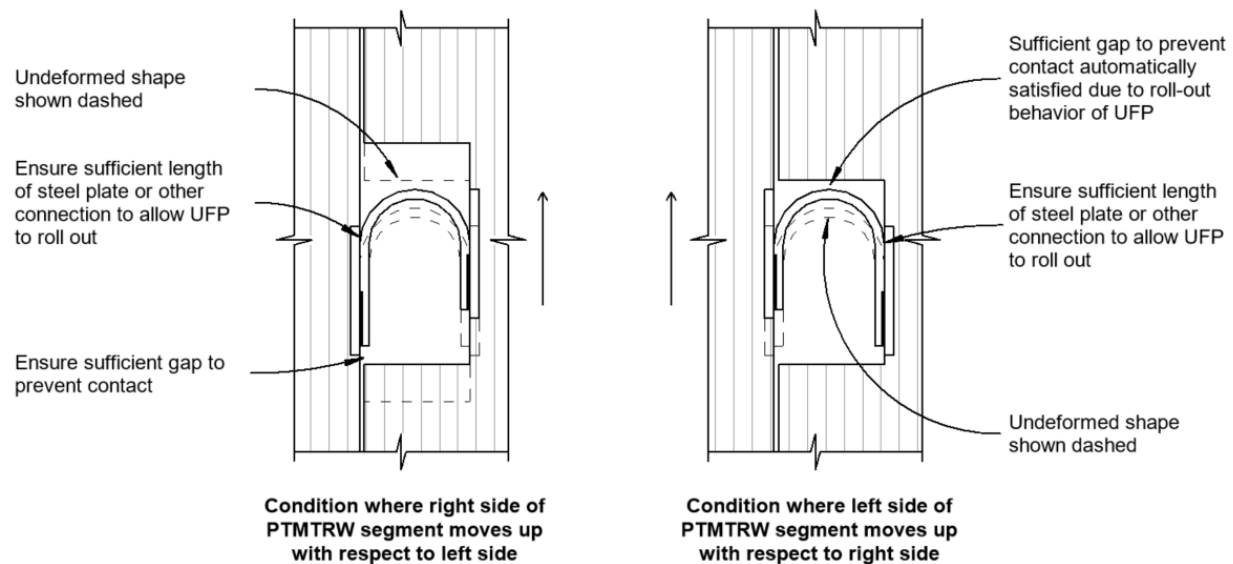


Figure C14.5.2.6 Representative deformed shapes of PTRW rocking deformations and UFP rolling



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COMMENTARY TO CHAPTER 15, SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURES

Revise Section C15.4.10 as follows:

C15.4.10 Requirements for Nonbuilding Structure Foundations on Liquefiable Sites

Section 12.13.9 allows shallow foundations to be built on liquefiable soils, with a number of restrictions. Many nonbuilding structures are sensitive to large foundation settlements. This sensitivity is caused by restraint imposed by interconnecting piping and equipment and the buckling sensitivity of shell structures. Therefore, to build these structures on shallow foundations on liquefiable soils, it must be demonstrated that the foundation, the nonbuilding structure not similar to buildings, and the connecting systems, can be designed for the soil strength loss, the anticipated settlements from lateral spreading, and the total and differential settlements induced by MCE_L ground motions as defined in Section 11.8.3.

Revise Section C15.6.1 as follows:

C15.6.1 Earth-Retaining Structures

Section C11.8.3 presents commonly used approaches for the design of nonyielding walls and yielding walls for bending, overturning, and sliding, taking into account the varying soil types, importance, and site seismicity.

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COMMENTARY TO CHAPTER 16, NONLINEAR RESPONSE HISTORY ANALYSIS

Revise Chapter C16 as follows:

C16.1.1 Scope

Response history analysis is a form of dynamic analysis in which response of the structure to a suite of ground motions is evaluated through numerical integration of the equations of motions. In nonlinear response history analysis, the structure's stiffness matrix is modified throughout the analysis to account for the changes in element stiffness associated with hysteretic behavior and P-delta effects. When nonlinear response history analysis is performed, the R , C_d , and Ω_0 coefficients considered in linear procedures are not applied because the nonlinear analysis directly accounts for the effects represented by these coefficients.

Nonlinear response history analysis is permitted to be performed as a stand alone method for the design of any structure and is specifically required to be performed for the design of certain structures incorporating seismic isolation or energy dissipation systems. Nonlinear response history analysis is also frequently used for the design of structures that use alternative structural systems or do not fully comply with the prescriptive requirements of the standard in one or more ways. Risk Category I, II, and III structures require analysis at the Risk-Targeted Maximum Considered Earthquake (MCE_R) level and to be more consistent with the target reliabilities indicated in Section 1.3.1.3. Risk Category IV structures also require analysis at the Design Earthquake (DE) per the requirements of Section 1.3.3.

The target collapse reliabilities given in Table 1.3-2 are defined such that when a building is subjected to MCE_R ground motion, not greater than a 10% probability of collapse exists for Risk Category I and II structures. For Risk Category III and IV structures, these maximum collapse probabilities are reduced to 5% and 2.5%, respectively.

There are additional performance expectations for Risk Category III and IV structures that go beyond the collapse safety performance goals (e.g., limited damage and postearthquake functionality for lower ground motion levels). These enhanced performance goals are addressed in this chapter by augmenting demands on force-controlled actions by $I_e > 1.0$, not permitting any unacceptable responses for Risk Category III or IV structures when subjected to MCE_R ground motion records and requiring deformation-controlled actions to be checked against the ASCE 41 Immediate Occupancy performance level under Design Earthquake shaking intensity in all Risk Category IV structures.

It is conceptually desirable to create a Chapter 16 response history analysis (RHA) design process that explicitly evaluates the collapse probability and ensures that the performance goal is fulfilled. However, explicit evaluation of collapse safety is a difficult task requiring (a) a structural model that is able to directly simulate the collapse behavior, (b) use of numerous nonlinear response history analyses, and (c) proper treatment of many types of uncertainties. This process is excessively complex and lengthy for practical use in design. Therefore, Chapter 16 maintains the simpler approach of *implicitly* demonstrating adequate performance through a prescribed set of analysis rules and acceptance criteria. Even so, this implicit approach does not preclude the use of more advanced procedures that explicitly demonstrate that a design fulfills the collapse safety goals. Such more advanced procedures are permitted by Section 1.3.1.3 of this standard. An example of an advanced explicit procedure is the building-specific collapse assessment methodology in Appendix F of FEMA P-695 (FEMA 2009b).

C16.1.2 Structural Design Basis

This chapter is intended to be a stand-alone procedure. However, there are important design requirements in Chapter 12 that are still required. The first are the basic requirements in Section 12.1 and its subsections that set forth requirements on continuous load path and interconnectedness of all elements of the structure, not just those that are part of the designated seismic force resisting system. Second, this chapter is intended to be used with structures that are already permitted in Table 12.2-1. However, the section permits the use of structural systems taller than the height limits specified in Table 12.2-1 because of the added rigor in the nonlinear analysis procedure when compared to the linear procedures in Chapter 12. Elsewhere in this standard, other requirements are referenced, such as those for diaphragm design and out-of-plane wall anchorage.

C16.1.3 Vertical Response Analysis Most structures are not sensitive to the effects of response to vertical ground shaking, and there is little evidence of the failure of structures in earthquakes resulting from vertical response. However, some nonbuilding structures and building structures with long spans, cantilevers, prestressed construction, or vertical discontinuities in their gravity-load-resisting systems can experience significant vertical earthquake response that can cause failures. The linear procedures of Chapter 12 account for these effects in an approximate manner, through use of the $0.2S_{DS}D$ term in the load combinations. When nonlinear response history analysis is performed for structures with sensitivity to vertical response, direct simulation of this response is more appropriate than the use of the approximate linear procedures. However, in order to properly capture vertical response to earthquake shaking, it is necessary to accurately model the stiffness and distribution of mass in the vertical load system, including the flexibility of columns and horizontal framing. This effort can considerably increase the complexity of analytical models. Rather than requiring this extra effort in all cases where vertical response can be significant, this chapter continues to rely on the approximate approach embedded in Chapter 12 for most cases. However, where the vertical load path is discontinuous and where vertical response analysis is required by Chapter 15, Chapter 16 does require explicit modeling and analysis of vertical response. Since in many cases the elements sensitive to vertical earthquake response are not part of the seismic force-

resisting system, it is often possible to decouple the vertical and lateral response analyses, using separate models for each.

Appropriate accounting for the effects of vertical response to ground shaking requires that horizontal framing systems, including floor and roof systems, be modeled with distributed masses and sufficient vertical degrees of freedom to capture their out-of-plane dynamic characteristics. This increased fidelity in modeling of the structure's vertical response characteristics will significantly increase the size and complexity of models. As a result, the chapter requires direct simulation of vertical response only for certain structures sensitive to those effects and relies on the procedures of Chapter 12 to safeguard the vertical response of other structures.

C16.1.4 Documentation By its nature, most calculations performed using nonlinear response history analysis are contained within the input and output of computer software used to perform the analysis. This section requires documentation, beyond the computer input and output, of the basic assumptions, approaches, and conclusions so that thoughtful review may be performed by others including peer reviewers and the Authority Having Jurisdiction (AHJ). This section requires submittal and review of some of these data before the analyses are performed, in order to ensure that the engineer performing the analysis/design and the reviewers are in agreement before substantive work is performed.

C16.3 MODELING AND ANALYSIS

C16.3.1 Modeling

Nonlinear response history analysis offers several advantages over linear response history analysis, including the ability to model a wide variety of nonlinear material behaviors, geometric nonlinearities (including P-delta and large displacement effects), gap opening and contact behavior, and nonlinear viscous damping, and to identify the likely spatial and temporal distributions of inelasticity. Nonlinear response history analysis has several disadvantages, however including an increased effort to develop the analytical model, increased time to perform the analysis (which is often complicated by difficulties in obtaining converged solutions), sensitivity of computed response to system parameters, large amounts of analysis results to evaluate, and the inapplicability of superposition to combine live, dead, and seismic load effects.

While computation of collapse probability is not necessary, it is important to note that mathematical models used in the analysis should have the capability to determine if collapse occurs when the structure is subjected to MCE_R level ground motions. The ability to predict collapse is important because the global acceptance criteria in Section 16.4.1.1 allow collapse (or unacceptable response) to occur for only one of the 11 ground motions for Risk Category I and II buildings and allows no such responses for Risk Categories III and IV buildings. Development of models with the ability to predict collapse requires attributes such as cyclic loss of strength and stiffness, low cycle fatigue failure, and geometric nonlinearity.

Although analytical models used to perform linear analysis, in accordance with Chapter 12, typically do not include representation of elements other than those that compose the intended lateral-force-resisting system, the gravity-load-carrying system and some nonstructural components can add significant stiffness and strength. Because the goal of nonlinear response history analysis is to accurately predict the building's probable performance, it is important to include such elements in the analytical model and also to verify that the behavior of these elements will be acceptable. This inclusion may mean that the contribution of stiffness and strength from elements considered as nonparticipating elements in other portions of this standard should be included in the response history analysis model.

Expected material properties are used in the analysis model, attempting to characterize the expected performance as closely as possible. It is suggested that expected properties be selected considering actual test data for the proposed elements. Where test data are not readily available, the designer may consider estimates as found in ASCE 41 and the PEER TBI Guidelines (Bozorgnia et al. 2009). Guidance on important considerations in modeling may also be found in *Nonlinear Structural Analysis for Seismic Design*, NIST GCR 10-917-5 (NIST 2010).

Two-dimensional structural models may be useful for initial studies and for checking some specific issues in a structure; however, the final structural model used to confirm the structural performance should be three-dimensional.

For certain structures, the response under both horizontal and vertical ground motions should be considered. NIST GCR 11-917-15 (NIST 2011) provides some guidance to designers considering the application of vertical ground motions. To properly capture the nonlinear dynamic response of structures where vertical dynamic response may have a significant influence on structural performance, it is necessary to include vertical mass in the mathematical model. Typically, the vertical mass must be distributed across the floor and roof plates to properly capture vertical response modes. Additional degrees of freedom (e.g., nodes at quarter points along the span of a beam) need to be added to capture this effect, or horizontal elements need to be modeled with consistent mass. Numerical convergence problems caused by large oscillatory vertical accelerations have been noted (NIST 2012), where base rotations caused by wall cracking in fiber wall models are the primary source of vertical excitation. See also the Commentary on Chapter 22.

Consideration of the additional vertical load of $(0.2S_{DS}) * D$, as per Section 12.4.2, is inappropriate for response history analysis. Response history analyses are desired to reflect actual building response to the largest extent possible. Applying an artificial vertical load to the analysis model before application of a ground motion results in an offset in the yield point of elements carrying gravity load because of the initial artificial stress. Similarly, applying an artificial vertical load to the model at the conclusion of a response history analysis is not indicative of actual building response. If vertical ground motions are expected to significantly affect response, application of vertical shaking to the analysis model is recommended. It should be noted that vertical response often occurs at higher frequencies than lateral response, and hence, a finer analysis time-step might be required when vertical motions are included.

For structures composed of planar seismic force-resisting elements connected by floor and roof diaphragms, the diaphragms should be modeled as semirigid in plane, particularly where the vertical elements of the seismic force-resisting system are of different types (such as moment frames and walls). Biaxial bending and axial force interaction should be considered for corner columns, nonrectangular walls, and other similar elements.

Nonlinear response history analysis is load path-dependent, with the results depending on combined gravity and lateral load effects. The MCE shaking and design gravity load combinations required in ASCE 7 have a low probability of occurring simultaneously. Therefore, the gravity load should instead be a realistic estimate of the expected loading on a typical day in the life of the structure. In this chapter, two gravity load cases are used. One includes an expected live loading characterizing probable live loading at the time of the MCE shaking, and the other, no live load. The case without live load is required to be considered only for those structures where live load constitutes an appreciable amount of the total gravity loading. In those cases, structural response modes can be significantly different, depending on whether the live load is present. The dead load used in this analysis should be determined in a manner consistent with the determination of seismic mass. When used, the live load is reduced from the nominal design live load to reflect both the low probability of the full design live load occurring simultaneously throughout the building and the low probability that the design live load and MCE shaking will occur simultaneously.

The reduced live load values of $0.8L_0$ for live loads that exceed $100 \text{ lb} / \text{ft}^2$ ($4.79 \text{ kN} / \text{m}^2$) and $0.4L_0$ for all other live loads, were simply taken as the maximum reduction allowable in Sections 4.7.2 and 4.7.3.

Gravity loads are to be applied to the nonlinear model first and then ground shaking simulations are applied. The initial application of gravity load is critical to the analysis, so member stresses and displacements caused by ground shaking are appropriately added to the initially stressed and displaced structure.

C16.3.3 Seismic Loads:

The primary objective of this chapter is to provide design requirements for structures subject to MCE_R shaking intensity. Additional design provisions are required for Risk Category IV structures to provide enhanced performance at the Design Earthquake intensity. This necessitates an explicit evaluation of the structure using ground motions selected and scaled to the DE target spectrum. These ground motions can either be taken as two-thirds of the MCE_R ground motion acceleration records or a new suite can be selected and scaled.

C16.3.4 P-delta Effects

P-delta effects should be realistically included, regardless of the value of the elastic story stability coefficient $\theta = P\Delta_e / (Vh)$. The elastic story stability coefficient is not a reliable indicator of the importance of P-delta during large inelastic deformations. This problem is especially important for

dynamic analyses with large inelastic deformations because significant ratcheting can occur. During these types of analyses, when the global stiffness starts to deteriorate and the tangent stiffness of story shear to story drift approaches zero or becomes negative, P-delta effects can cause significant ratcheting (which is a precursor to dynamic instability) of the displacement response in one direction. The full reversal of drifts is no longer observed, and the structural integrity is compromised. To ascertain the full effect of P-delta effects for a given system, a comparison of static pushover curves from a P-delta model and non-P-delta model can be compared.

When including P-delta effects, it is important to capture not only the second-order behavior associated with lateral displacements but also with global torsion about the vertical axis of the system. In addition, the gravity load used in modeling P-delta effects must include 100% of the gravity load in the structure. For these reasons, the use of a single “leaning column,” where much of a structure’s vertical weight is lumped at a single vertical coordinate, is discouraged, and instead, the structure’s vertical load should be distributed throughout the structure in a realistic manner, either through direct modeling of the gravity system or by appropriately distributed “leaning columns.”

In some structures, in addition to considering P-delta effects associated with global structural deformation, it is also important to consider local P-delta effects associated with the local deformation of members. This is particularly important for slender elements that are subject to buckling.

C16.3.5 Torsion

Inherent torsion is actual torsion caused by differences in the location of the center of mass and center of rigidity throughout the height of the structure. Accidental torsion effects, as per Section 12.8.4.2, are artificial effects that attempt to account for actual variations in load and material strengths during building operation that differ from modeling assumptions. Some examples of this difference would be nonuniformity of the actual mass in the building, openings in the diaphragm that are unaccounted for, torsional foundation input motion caused by the ground motion being out of phase at various points along the base, the lateral stiffness of the gravity framing, variation in material strength and stiffness caused by typical construction tolerances, and incidental stiffness contribution by the nonstructural elements.

When the provision for accidental torsion was first introduced, it was to address buildings that have no inherent torsion but are sensitive to torsional excitation. Common examples of this type of configuration are cruciform core or I-shaped core buildings. In reality, many things can cause such a building to exhibit some torsional response. None of the aforementioned items are typically included in the analysis model; therefore, the accidental torsion approach was introduced to ensure that the structure has some minimum level of resistance to incidental twisting under seismic excitation.

The accidental torsion also serves as an additional check to provide more confidence in the torsional stability of the structure. When there is no inherent torsion in the building, accidental torsion is a crucial step in the design process because this artificial offset in the center of mass is a simple way

to force a minimum level of twisting to occur in the building. The accidental torsion step (i.e., the required 5% force offsets) is also important when checking for plan irregularities in symmetric and possibly, torsionally flexible buildings. Where there is already inherent torsion in the building, additional accidental torsion is not generally a crucial requirement because the building model will naturally twist during analysis, and no additional artificial torsion is required for this twisting to occur. However, for buildings exhibiting either torsional or extreme torsional irregularities, inclusion of accidental torsion in the nonlinear analysis is required by this standard to assist in identification of potential nonlinear torsional instability.

Accidental torsion may be excluded from the nonlinear analysis when it can be demonstrated that the structural response is not significantly affected by accidental torsion (PEER TBI, DeBock et al., 2013, and De la Llera and Chopra, 1994). A method is provided to determine the significance of accidental torsion on the structural response based on a torsional index parameter J . Where the torsional index is less than 1.2 on all floors, it implies that the structural response is not significantly sensitive to accidental torsion.

C16.3.6 DAMPING

The provisions for mass and stiffness proportional equivalent viscous damping methods are based on PEER/ATC-72-1, PEER TBIv2.03, Chopra and McKenna, 2016a and NZS 1170.5:2004 Part 5: Earthquake actions—New Zealand (SNZ 2004).

Equivalent viscous damping can be represented by combined mass and stiffness (Rayleigh) damping. To ensure that the equivalent viscous damping does not exceed the target level in the primary response modes, the equivalent viscous damping is typically set at the target level for two periods, one above the fundamental period and one below the highest mode frequency of significance. This results in Rayleigh damping ascribing different damping ratios to different modes (Figure C16.3.5-0). Typically, the calculated damping for modes in between the anchor time periods, such as the fundamental period, will be less than the target; outside of these, such as for higher modes, will be higher than the target. In fact, even in between the anchor time periods, the damping for the fundamental and higher modes can differ significantly (Figure C16.3.5-0). Thus, Rayleigh damping risks severely overdamping the higher modes relative to the fundamental modes and artificially diminishing their effects. To overcome this risk, the recommendation of NZS 1170.5:2004 is adapted and the damping ascribed to the highest mode necessary to achieve 90% mass participation is limited to 8 times the damping ascribed to the fundamental mode. Additionally, since the damping for various modes necessary to achieve 90% modal mass participation can vary significantly, it's required to ensure that the average of the damping ratios for each mode weighted by the mode's mass participation not exceed the equivalent viscous damping ratio.

For very tall buildings, the second and even third modes can have significant contributions to response; in this case, the lower multiple on T_1 may need to be reduced to avoid excessive damping in these modes. Consistent with the ground motion modification procedures in Section 16.2.3, the period range for scaling specified in Section 16.2.3.1 is recommended for anchoring Rayleigh damping models at the target equivalent viscous damping ratio. However, the expected nonlinear

response of the structure should be considered to avoid overdamped solutions. If the period of the structure is expected to lengthen significantly, then the equivalent viscous damping ratio should also be limited to not greater than the target equivalent viscous damping ratio at longer periods (e.g., $1.5T_1$ to $2.0T_1$).

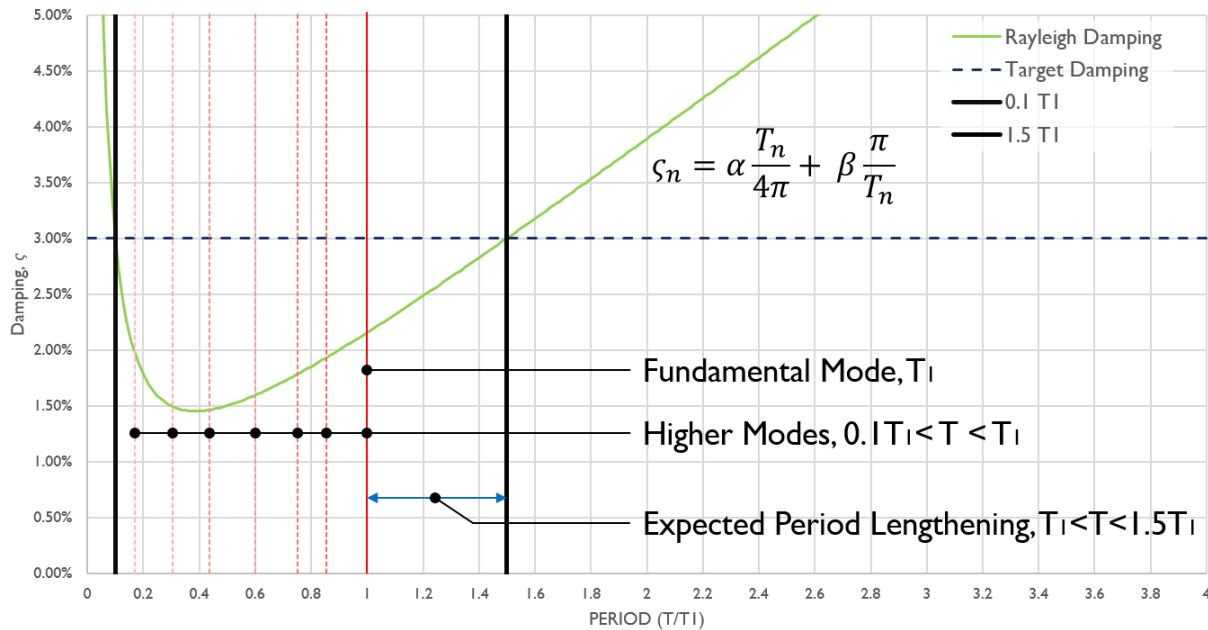


Figure C16.3.5-0. Example of Uneven Damping Ratios Ascribed by Rayleigh Damping to Various Modes

Equivalent viscous damping may alternatively be represented by modal damping, which allows for the explicit specification of the target damping in each mode.

Various studies have shown that the system damping may vary with time as the structure yields, and in some cases, damping well above the target levels can temporarily exist. Zareian and Medina (2010) provide recommendations for the implementation of damping in such a way that the level of viscous damping remains relatively constant throughout the response.

The level of structural damping caused by component-level hysteresis can vary significantly based on the degree of inelastic action. Typically, hysteretic damping provides a contribution less than or equal to 2.5% of critical damping.

Damping and/or energy dissipation caused by supplemental damping and energy dissipation elements should be explicitly accounted for with component-level models and not included in the overall viscous damping term.

It is noted that in the context of choosing a damping model to employ in response history analysis, consideration should be given to plasticity models and viscous damping assumptions. Response of models employing concentrated or lumped plasticity (zero length) elements has been shown to be

sensitive to damping model assumptions. Spurious damping forces have been observed in concentrated plasticity models when used in conjunction with initial-stiffness-proportional Rayleigh damping models (Chopra and McKenna 2016b). Such spurious damping forces can result in effective viscous damping exceeding the target equivalent viscous damping ratio on the order of three times the yield moment of adjoining structural elements (Chopra and McKenna 2016b), which can in turn lead to an underestimation of dynamic response (Figure C16.3.5-1). Although the use of tangent-stiffness-proportional Rayleigh damping may substantially diminish such spurious damping forces, this approach is not recommended as it lacks physical basis and has difficult conceptual implications such as negative damping coefficients associated with the negative tangent stiffness of degrading components.

Use of modal damping, which employs a damping matrix constructed by superposition of modal damping matrices, may eliminate spurious damping forces. When such modal damping is employed, damping ratios must be specified for all modes that are expected to contribute significantly to structural response (Chopra and McKenna 2016b). Response of distributed plasticity elements has been shown to be less sensitive to damping assumptions compared to concentrated plasticity elements. However, even such elements may become more sensitive to damping assumptions at deformation responses approaching collapse (Chopra and McKenna 2016b, Hall 2016). Some other methods to avoid spurious damping forces are condensing-out the degrees of freedom that would generate spurious damping forces (Bernal, 1994 via PEER/ATC 72) or enforcing upper-bounds on the stiffness proportional damping terms (Hall, 2005, Powell, 2008 via PEER/ATC 72). The user is referred to PEER/ATC 72 for a more detailed discussion on spurious damping forces and methods to avoid them.



Beam w/ Concentrated Plasticity Hinges

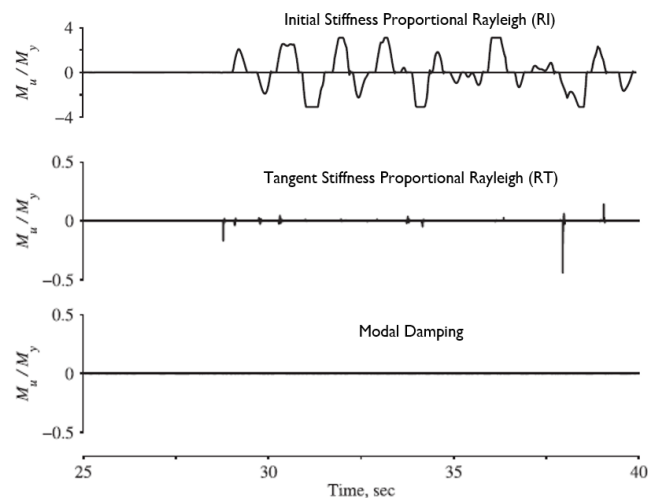


Figure C16.3.5-1. Example of Spurious Damping Forces

Use of damping models that include a mass-proportional term may lead to unrealistically large forces, and an underestimation of response, in structures with large rigid body motion. This effect may be significant in analyses of tall buildings, where drifts in the upper portions of the building are due, in part, to deformations that occur in lower levels of the building (Hall 2005 via PEER/ATC 72).

C16.3.7 Diaphragms, Chords, and Collectors

Often diaphragms, chords, and collectors are not explicitly modeled in the nonlinear analysis. The provisions permit the forces used to design diaphragms and chords to be based on the tributary diaphragm weight times the mean floor acceleration from the nonlinear analysis. These floor accelerations are what the diaphragm design force equations in Section 12.10 are attempting to predict. Because the user can assess them directly from the nonlinear model, they can be used in lieu of Section 12.10. However, the minimum diaphragm design force from Section 12.10.1 is still required when the forces from the nonlinear analysis floor accelerations are used. This equation represents a minimum design force irrespective of analysis procedures that has historically been enforced on structures and the committee elected to retain them. Diaphragm spanning between vertical seismic force resisting elements without transfers can be considered noncritical because they likely could sustain some nonlinearity if the elements were modeled explicitly, so overstress is not as significant as in a transfer diaphragm. Transfer diaphragms are those that transfer forces to or from vertical seismic force resisting elements due to discontinuities in vertical elements, some

vertical elements stopping at a floor, or changes in relative stiffness of the vertical seismic force resisting system elements. Transfer diaphragms are often considered critical unless explicitly modeled to account for nonlinear behavior in the transfer diaphragm.

Determining forces on transfer diaphragms often involves combining the diaphragm inertial response with the forces transferred between vertical, lateral force-resisting elements. One way to obtain the transfer forces is to subtract the forces in the elements on each side of the transfer diaphragm at each time step and take the maximum value of that from each record. If the diaphragm elements are explicitly modeled with elastic stiffness, section cuts in the diaphragm may also be used to arrive at the transfer forces. The provisions permit component actions in transfer diaphragms to be explicitly modeled and assessed as deformation-controlled.

The provisions permit the diaphragm and chords to be designed for Design Earthquake demands using the linear procedures in Chapter 12, using a separate linear analysis model with forces from either the equivalent lateral force procedure or linear dynamic analysis and the demand equations in Section 12.10.1 or 12.10.3.

The provisions require collector elements to be designed explicitly for the forces in the nonlinear analysis, with all their component actions being considered force-controlled, as opposed to being based on the linear design. This is done because the nonlinear analysis may predict different horizontal or vertical distributions for forces in the analysis model. There are a number of ways to extract collector forces from a nonlinear analysis, such as the use of section cuts or explicit collector elements in the diaphragm. Another possible way is to subtract the forces in the structural element(s) the collector is delivering load to below the diaphragm from the force in the element(s) above at each time step.

C16.3.8 Structural Walls and Their Anchorage

Properly modeling the out-of-plane behavior of structural walls is very complex and infrequently done in practice. Therefore, the provisions of Section 12.11 should be used without any modification from the nonlinear analysis model.

C16.3.9 Foundation Design

The design of foundation elements for structures designed to this chapter should be in accordance with Section 12.13, except for two sections. Section 12.13.4 reduces overturning because of the ELF procedure potentially overestimating it. Since overturning is explicitly determined based on nonlinear actions in the superstructure, the results from the nonlinear analysis are presumed to be a better estimate.

It is common to model the connection of the structure to the foundation and soil with a pinned or fixed base. When this is done, the soil actions are checked independently using the forces from the nonlinear analysis, sometimes using a separate analysis program specific to the design of mat foundations and combined footings. When the soil actions are not explicitly modeled, the provisions

require they be considered force-controlled actions. Soil actions are generally ductile, so when not modeled they can still be considered non-critical, meaning they only have to be evaluated for the mean response from the nonlinear analysis using their expected capacities. Similarly, stability for overturning can be treated as a non-critical action because rocking of the structure, when explicitly modeled, has generally shown to be an acceptable action.

However, there are times when both rocking and soil actions should be modeled. Specifically, where the flexibility associated with linear or nonlinear response of the soil or rocking could lead to changes in the structure's response or exacerbate deformation compatibility issues. See Section C8.4 of ASCE 41 for a detailed discussion of conditions where foundation flexibility and nonlinear soil actions are important to model.

The provisions permit soil actions to be modeled explicitly as deformation-controlled actions, enabling direct assessment of the superstructure's response—including footing plunging (unrecoverable downward settlement) from bearing capacity overstress and foundation element rocking. The stiffness of the soil actions should be determined based on Section 12.13.3 and the strength by Section 12.13.5.1. The provisions do not limit the nonlinear deformations at the subsurface-foundation interface because it is presumed that the superstructure's response will indicate whether those foundation or soil deformations are acceptable for the design objective. If the soil actions are treated as force-controlled actions resistance factors shall be per Section 12.13.5.2.

Where ASCE 41 or the applicable material design standards permit foundation components or the connection of the structure to the foundation to be modeled as deformation-controlled actions, those elements' actions can be explicitly modeled as deformation-controlled. If those standards do not have provisions, then the foundation elements' actions and the connection between the foundation and the structure should be considered force-controlled.

Fixed-based modeling is not always conservative and there are cases when it may not be appropriate (see also Section C12.7.1). Some common cases include but are not limited to the following situations where vertical or horizontal soil springs may need to be included in the analysis model:

- 1) When the building foundation type or configurations vary significantly across the building footprint. One example of this is a building supported by both deep and shallow foundation elements. The difference in vertical and lateral stiffness of these two types of foundation elements can result in substantial differences in the lateral force distribution depending on whether or not foundation flexibility is modeled. Additional examples are
 - Buildings with nontrivial elevation differences at the foundation level. A fixed based model will distribute most of the lateral force to the upper foundation level which can be substantially different if foundation flexibility is modeled.

- Buildings for which the SFRS is supported on softer soils beneath one side of the building and stiffer soils beneath others, which results in potentially different levels of foundation rotations and horizontal displacements
- 2) When the building's superstructure's seismic force-resisting system includes elements which are sensitive to base rotations. This generally occurs for stiff lateral elements such as cantilever shear wall or braced frame elements. Fixed-base modeling might not be appropriate for these types of structures because it underestimates deformation demands on secondary lateral elements, such as gravity frames and their connections.
 - 3) Moment resisting frames. Fixed base modeling of moment frame columns could underpredict drifts when significant foundation rotation caused by soil flexibility occurs prior to column base yielding.
 - 4) Where foundation base diaphragm is designed outside of the analysis model and elements not in line with the seismic force resisting system are required to resist more than 20% of the seismic base shear from a given seismic force resisting element above.
 - 5) A rule of thumb test where $2/3 * h_n / (v_s * T) > 0.10$ is satisfied. This test was originally presented in NIST (2012) and adopted in FEMA P-2091 (Section 3.2).

Where:

h_n is the building height as defined in section 11.2 in ft

v_s is the effective shear wave velocity for site soil conditions in ft/s

T is the fundamental period calculated using a fixed base (in seconds)

Both versions 1 and 2 of the PEER TBI guidelines (Bozorgnia et al. 2009; PEER 2017) and NIST GCR 12-917-21 (NIST 2012) recommend the inclusion of subterranean building levels in the mathematical model of the structure. The modeling of the surrounding soil has several possible levels of sophistication, two of which are depicted below in (b) and (c) of Figure C16.3-1, which are considered most practical for current practice. For a MCE_R -level assessment, which is the basis for the Chapter 16 RHA procedure, the rigid bathtub model is preferred by PEER TBI (Bozorgnia et al. 2009) and NIST (2012) (Figure C16.3-1c). This model includes soil springs and dashpots, and identical horizontal ground motions are input at each level of the basement. Such a modeling approach, where the soil is modeled in the form of springs and/or dashpots (or similar methods) placed around the foundation, is encouraged but is not required. When spring and dashpot elements

are included in the structural model, horizontal input ground motions are applied to the ends of the horizontal soil elements rather than being applied to the foundation directly. A simpler but less accurate model is to exclude the soil springs and dashpots from the numerical model and apply the horizontal ground motions at the bottom level of the basement (Figure C16.3-1b), which is fixed at the base. Either the fixed-base (Figure C16.3-1b) or bathtub (Figure C16.3-1c) approach is allowed, but the bathtub approach is encouraged because it is more accurate.

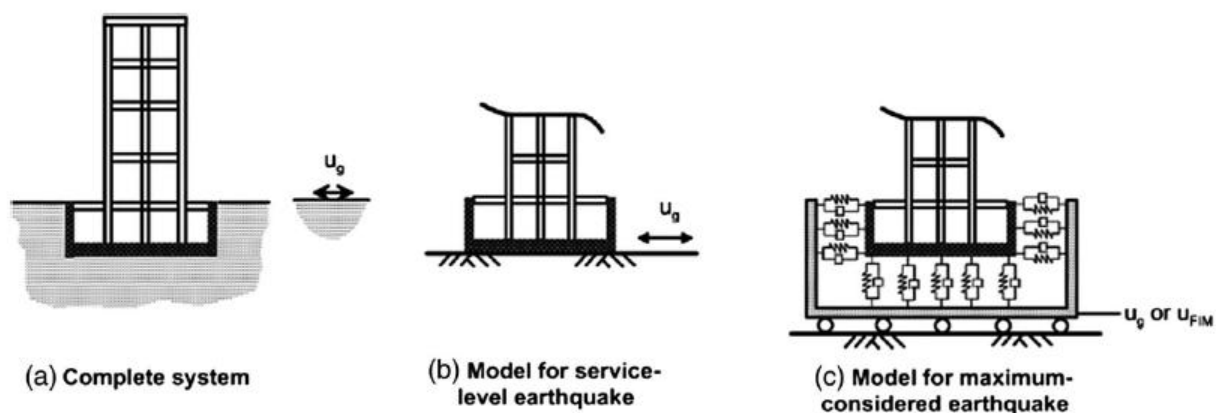


Figure C16.3-1. Illustration of the method of inputting ground motions into the base of the structural model.

Source: NIST (2012).

For the input motions, the PEER TBI (Bozorgnia et al. 2009, PEER 2017) guidelines allow the use of either the free-field motion, which is the motion defined in Section 16.2.2, or a foundation input motion modified for kinematic interaction effects. Guidelines for modeling kinematic interaction are contained in NIST (2012).

More sophisticated procedures for soil-structure interaction modeling, including the effects of multi-support excitation, can also be applied in RHA. Such analyses should follow the guidelines presented in NIST (2012).

Approximate procedures for the evaluation of foundation springs are provided in Chapter 19 of this standard.

C16.3.10 Prohibited Irregularities

Chapter 12 prohibits extreme weak story irregularities in almost all cases and weak story irregularities in structures assigned to Seismic Design Categories E and F. While a nonlinear analysis can better capture the performance of a building and the consequences of a weak-story irregularity, the standards committee felt that such irregularities have serious consequences and complete reliance on a nonlinear analysis to justify their existence may be misguided. Nonlinear analysis, while

often more accurate than linear analysis at predicting drifts, is still an analytical tool, not a simulation and may not fully capture the consequence of significant issues.

The provisions do permit an exception for low-rise buildings assigned to Seismic Design Category B and C, which is also present in Chapter 12. Another exception is provided for weak stories in the upper portion of buildings. This exception is intended to permit the use of outrigger trusses or walls in tall buildings, where it is common to have an outrigger to transfer some of the overturning forces from a core to specially designed and detailed columns. Because the columns under an outrigger should be classified as a Type 3 discontinuous system vertical irregularity, they should be designed with all their actions considered force-controlled. When either of these two exceptions is taken, the permission to have an unacceptable response is eliminated. This is done because an unacceptable response may indicate that the weak-story irregularity is a potential collapse mechanism.

C16.4.1 Global Acceptance Criteria

C16.4.1.1 Unacceptable Response This section summarizes the criteria for determining unacceptable response and how the criteria were developed. It must be made clear that these unacceptable response acceptance criteria are not the primary acceptance criteria that ensure adequate collapse safety of the building but rather the story drift criteria and the element-level criteria discussed later in Section C16.4. The unacceptable response acceptance criteria were developed to be a secondary protection to supplement the primary criteria. Unacceptable responses result in instabilities and loss of gravity load support. Consequently, if it can be shown that after a deformation-controlled element reaches its (collapse prevention) limit, the model is able to redistribute demands to other elements, this would not constitute an unacceptable response. The acceptance criteria were intentionally structured in this manner because there is high variability in unacceptable response (as described in this section) and the other primary acceptance criteria are much more stable and reliable (because they are based on mean values of 11 motions rather than the extreme response of 11 motions).

When performing nonlinear analysis for a limited suite of ground motions, the observance of a single unacceptable response (or, conversely, the observance of no unacceptable responses) is statistically insignificant. That is, it is reasonably probable that no collapses will be observed in a small suite of analyses, even if the structure has a greater than 10% chance of collapse at MCE_R shaking levels.

It is also possible that a structure with less than a 10% chance of collapse at MCE_R shaking levels will still produce an unacceptable response for one ground motion in a small suite. In order for statistics on the number of unacceptable responses in a suite of analyses to produce a meaningful indication of collapse probability, a very large suite of analyses must be performed. Furthermore, the observance or nonobservance of an unacceptable response depends heavily on how the ground motions were selected and scaled (or spectrally matched) to meet the target spectrum.

Since the observance or nonobservance of an unacceptable response is not statistically meaningful, the standard does not rely heavily on the prohibition of unacceptable responses in the attempt to “prove” adequate collapse safety. The many other acceptance criteria of Section 16.4 are relied

upon to implicitly ensure adequate collapse safety of the building. If one desired to expand the unacceptable response acceptance criteria to provide true meaningful collapse safety information about the building, a more complex statistical inference approach would need to be used. This is discussed further below.

The statistical insignificance of unacceptable response in a small suite of analyses leaves a large open question about how to interpret the meaning of such responses when they occur. Even though occurrence of a single unacceptable response is statistically meaningless, the occurrence of many unacceptable responses (e.g., 5 of 11) indicates that the collapse probability is significantly in excess of 10%. In addition, a conscientious structural designer is concerned about such occurrences, and the occurrences of unacceptable responses may provide the designer some insight into possible vulnerabilities in the structural design.

Some engineers presume that the acceptance criteria related to *average* response effectively disallow any unacceptable responses (because you cannot average in an infinite response scenario), while others presume that *average* can also be interpreted as *median*, which could allow almost half of the ground motions to cause an unacceptable response.

The statistics presented below are provided to help better interpret the meaning of observance of a collapse of the model analyzed or other type of unacceptable response in a suite of analyses. These simple statistics are based on predicting the occurrence of collapse (or other unacceptable response) using a binomial distribution, based on the following assumptions:

- The building's collapse probability is exactly 10% at the MCE_R level.
- Collapse probability is lognormally distributed and has a dispersion (lognormal standard deviation) of 0.6. This value includes all sources of uncertainty and variability (e.g., record-to-record variability, modeling uncertainty). The value of 0.6 is the same value used in creating the risk-consistent hazard maps for ASCE 7-10 (FEMA 2009a) and is consistent with the values used in FEMA P-695 (FEMA 2009b).
- The record-to-record variability ranges from 0.25 to 0.40. This is the variability in the collapse capacity that would be expected from the analytical model. This value is highly dependent on the details of the ground motion selection and scaling; values of 0.35 to 0.45 are expected for motions that do not fit tightly to the target spectrum, and values of 0.2 to 0.3 are expected for spectrally matched motions (FEMA 2009b).

Figure C16.4-1 shows collapse fragility curves for a hypothetical building that has a 10% collapse probability conditioned on MCE_R motion ($P[C | MCE_R] = 10\%$), with an assumed record-to-record collapse uncertainty of 0.40 and a total collapse uncertainty of 0.60. The figure shows that the median collapse capacity must be a factor of 2.16 above the MCE_R ground motion level, that the probability of collapse is 10% at the MCE_R when the full variability is included (as required), but

that the probability of collapse is only 2.7% at the MCE_R when only the record-to-record variability is included. This 2.7% collapse probability is what would be expected from the structural model that is used in the RHA assessment procedure.

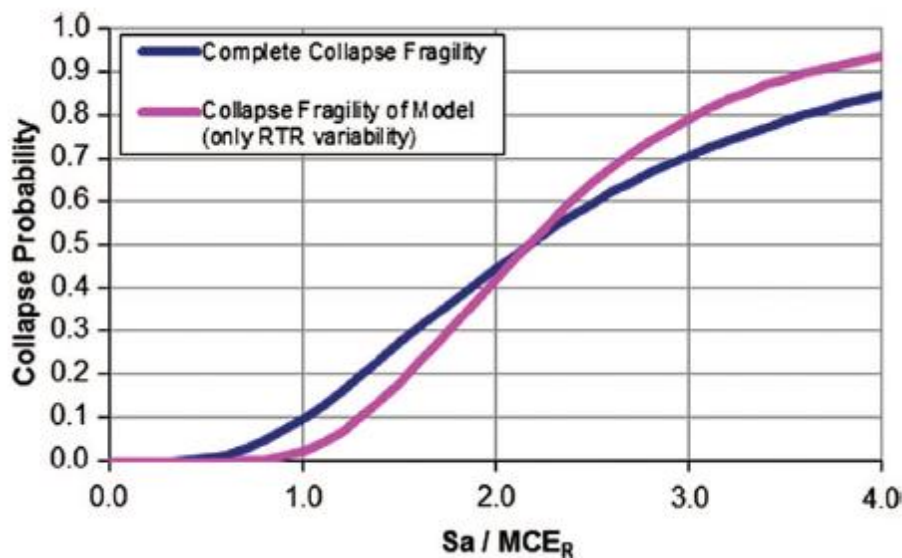


Figure C16.4-1. Collapse fragilities for a building with $P[C | MCE_R] = 10\%$ and $\beta_{COL,RTR} = 0.40$.

Table C16.4-1 shows the probability of observing n collapses in a suite of 11 ground motions for a structure that has different values of $P[C | MCE_R]$.

Table C16.4-1. Likelihood of Observing Collapses in 11 Analyses, Given Various MCE_R Collapse Probabilities and a Record-to-Record Uncertainty of 0.4.

Number of Collapses	Likelihood for Various $P[C MCE_R]$ Values				
	0.05	0.10	0.15	0.20	0.30
0 of 11	0.93	0.74	0.51	0.30	0.07
1 of 11	0.07	0.23	0.36	0.38	0.21
2 of 11	0	0.03	0.11	0.22	0.29
3 of 11	0	0	0.02	0.08	0.24
4 of 11	0	0	0	0.02	0.13
5 of 11	0	0	0	0	0.05

Table C16.4-1 shows that for a building meeting the $P[C | MCE_R] = 10\%$ performance goal, there is a 74% chance of observing no collapses, a 23% chance of observing one collapse, a 3% chance of observing two collapses, and virtually no chance of observing more than two collapses. In comparison, for a building with $P[C | MCE_R] = 20\%$, there is a 30% chance of observing no collapses, a 38% chance of observing one collapse, a 22% chance of observing two collapses, and a 10% chance of observing more than two collapses.

This table illustrates that

- Even if no collapses are observed in a set of 11 records, this does not, in any way, prove that the $P[C | MCE_R] = 10\%$ performance goal has been met. For example, even for a building with $P[C | MCE_R] = 20\%$, there is still a 30% chance that no collapses will be observed in the analysis. Therefore, the other noncollapse acceptance criteria (e.g., criteria for drifts and element demands) must be relied on to enforce the 10% collapse probability goal.
- If the $P[C | MCE_R] = 10\%$ performance goal is met, it is highly unlikely (only a 3% chance) that two collapses will be observed in the set of 11 records. Therefore, an acceptance criterion that prohibits two collapses is reasonable.

The collapse likelihoods shown in Table C16.4-1 are based on a relatively large record-to-record variability value of 0.40. Table C16.4-2 illustrates similar statistics for the case when the record-to-record variability is suppressed in ground motion selection and scaling, such as occurs with spectral matching. This table shows that, for a building meeting the $P[C | MCE_R] = 10\%$ performance goal and with record-to-record variability taken as 0.25, the likelihood of observing a collapse response is very low. This is why no unacceptable responses are permitted in the suite of analyses when spectral matching is used.

Table C16.4-2. Likelihood of Observing Collapses in 11 Analyses, Given Various MCE_R Collapse Probabilities and a Record-to-Record Uncertainty of 0.25.

Number of Collapses	Likelihood for Various $P[C MCE_R]$. Values				
	0.05	0.10	0.15	0.20	0.30
0 of 11	1.00	0.99	0.93	0.79	0.30
1 of 11	0	0.01	0.07	0.19	0.38
2 of 11	0	0	0	0.02	0.22
3 of 11	0	0	0	0	0.08
4 of 11	0	0	0	0	0.02

5 of 11	0	0	0	0	0
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For Risk Categories I and II structures, if more than 11 ground motions are used for analysis, then additional unacceptable responses may be permissible. Two unacceptable responses would be permissible if 20 or more motions are used, and three unacceptable responses would be permissible if 30 or more motions are used. For Risk Categories III and IV structures, the collapse probability goals are 6% and 3%, respectively, at the MCE_R $P[C | MCE_R]$ vel. When the above computations are redone using these lower collapse probability targets, this shows that the acceptance criteria should require that no motions of the 11 produce an unacceptable response for these categories.

Typically, mean building response values (story drifts, element deformations, and forces) are used in acceptance evaluations, where the “mean” is the simple statistical average for the response parameter of interest. When an unacceptable response occurs, it is not possible to compute a mean value of the building response values because one of the 11 response quantities is undefined. In this case, rather than the mean, the standard requires use of the counted median response multiplied by 1.2 but not less than the mean response from the remaining motions.

To compute the median value, the unacceptable response is assumed as larger than the other responses and then, assuming that 11 analyses were performed, the counted median value is taken to be the 6th largest value from the set of 11 responses. The 1.2 factor is based on a reasonable ratio of mean to median values for a lognormal distribution ($\beta = 0.4$ results in mean / median = 1.08, $\beta = 0.5$ results in mean / median = 1.13, $\beta = 0.6$ results in mean / median = 1.20, and $\beta = 0.7$ results in mean / median = 1.28).

The requirement to also check the mean of the remaining 10 response results is simply an added safeguard to ensure that the 1.2 — median value does not underpredict the mean response values that should be used when checking the acceptance criteria.

Although currently the purpose of this acceptance criterion is not meant to quantify the structure’s collapse probability under MCE_R ground motions, the acceptance criterion can be recast to do so in future provisions. The collapse probability can be inferred from analysis results and compared to the target value (e.g., 10% for structures in Risk Category I or II). In this alternate light, existing statistical inference theories can be used to determine the number of acceptable responses, and the number of ground motions required to conclude that the proposed design may have an acceptable collapse probability.

As discussed in the previous section, analysis results can be thought of as following a binomial distribution. Based on this distribution, one could use the observed counts of collapsed and noncollapsed responses (indicated by unacceptable and acceptable responses) to estimate the collapse probability of the proposed design in a manner that accounts for the uncertainty in the estimated collapse probability. This uncertainty depends on the total number of ground motions. If few ground motions are used, there is a large uncertainty in the collapse probability. If many ground motions are used, there is a small uncertainty. For example, compare a set of 11 ground motions

with one unacceptable response to a set of 110 ground motions with ten unacceptable responses. Both sets most likely have a unacceptable response probability of 9.1%. The design with one unacceptable and ten acceptable responses has only a 34% chance that its unacceptable response probability is 10% or less. The design with 10 unacceptable and 100 acceptable responses has a 56% chance that its unacceptable response probability is 10% or less.

In the current acceptance criterion, the choice to require 11 ground motions follows from the need to have confidence in the average values of the resulting element-level and story-level responses (Section C16.2.3.1). These element-level and story-level responses are then used to *implicitly* demonstrate adequate collapse safety. If future provisions seek to *explicitly* ensure that the proposed design has an acceptable collapse probability, then this unacceptable response acceptance criterion should be revised using statistical inference theory to establish the number of required ground motions and the maximum number of unacceptable responses, as well as the element- and story-level response limits.

C16.4.1.2 Minimum Design Strength

The provisions require that the structure have a minimum strength in each direction equal to the design strength used in the design procedures in Chapter 12. Prior to the 2028 edition of the standard, the provisions required a full linear design per Section 12.8 or 12.9 in addition to the nonlinear response history analysis. That has been eliminated, but the committee felt that the structure should not have mean base shear in each direction less than what would be required to design a building kept using Chapter 12. The linear analysis design strength has been validated through detailed studies (NIST GCR 10-917-8).

C16.4.1.3 Transient Story Drift

The limit on mean story drift was developed to be consistent with the linear design procedures of this standard. To this end, the basic Table 12.12-1 story drift limits are adopted with the following adjustments:

- Increased by a factor of 1.5, to reflect the analysis being completed at the MCE_R ground motion level rather than at 2/3 of the MCE_R level, and
- Increased by another factor of 1.25, to reflect an average ratio of R/C_d .

These two increases are the basis for the requirement that the mean story drift be limited to 1.9 (which was rounded to 2.0) of the standard Table 12.12-1 limits.

The masonry-specific drift limits of Table 12.12-1 are not enforced in this section because the component-level acceptance criteria of Section 16.4.2 are expected to result in equivalent performance (i.e., a masonry building designed in accordance with Chapter 16 is expected to have similar performance to a masonry building designed using linear analysis methods and the more stringent drift limits of Table 12.12-1).

For tall structures, more restrictive drift limits are adopted than permitted under Table 12.2.1. When performing nonlinear response history analysis, the model is required to include all elements that significantly affect the stiffness and strength of the structure to resist lateral forces. This is in stark contrast to the intent of Chapter 12 analysis procedures, which consider only the seismic force-resisting system in the analysis. As a result, nonlinear response history analyses will include elements normally considered to carry only gravity loads, but which, particularly in tall buildings, add substantial stiffness to the analytical model. Analysis of these models will result in lower predictions of drift than the typical models used for linear analysis. If the same drift limits were applied, the effect would be to permit more flexible structures than permitted by Chapter 12. The drift limit applied for buildings over 240 ft is the same as that in the 2017 PEER Tall Buildings Initiative, Guidelines for Performance-Based Seismic Design of Tall Buildings, which is commonly used for the design of tall structures in the US. For buildings with heights between 100 ft (30 m) and 240 ft (73 m), Equation (16.4-1) applies and allows for a smooth transition between the requirements for tall buildings and the doubled limits of Table 12.12-1 requirement, as applied to Risk Category I and II mid-rise structures, other than masonry. For building heights above 240 ft (73 m), the story drift limit of 0.03 hsx controls over the value calculated per Equation (16.4-1).

For Risk Category IV structures, the provisions require the drift of the structure to be kept to within 130% of the values in Table 12.12-1 under DE hazard intensity. The increase from the values in Table 12.12-1 is representative of nonlinear analysis' higher degree of accuracy in predicting drifts. The drift limits for Risk Category IV structures in Table 12.12-1 are more than I_e divided by the drift limits for Risk Category I and II structures. Therefore, Risk Category IV structures are designed to more significant drift limits than the increase in design forces. This additional margin of drift limit is intentional. Limiting drifts to low values reduces damage in the structural system and the nonstructural components and systems, providing better performance.

C16.4.1.4 Residual Story Drift

The standard does not require checks on residual drift for Risk Category I, II, and III structures less than 240 ft (73 m) in height. Residual drift is the unrecoverable portion of transient drift that commonly occurs when a structure undergoes inelastic response. It is computed as the absolute value of the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any edge of the structure, following completion of the structure's response to earthquake motion. Residual drifts are an indicator of potential incipient dynamic instability, and the prudent engineer checks for this instability. Limiting residual drifts is an important consideration for post-earthquake operability and for limiting financial losses, but such performance goals are not included in the scope of the ASCE 7 standard. For Risk Category I and II buildings, the ASCE 7 standard is primarily meant to ensure the protection of life safety.

Design practice for tall buildings, however, has routinely included limits on residual drift. One important reason for this is that many tall buildings are located in dense urban areas. Residual drift is commonly used to judge the post-earthquake safety of damaged buildings. If tall buildings exhibit large residual drifts after an earthquake and consequently appear to be unstable, this could prompt

building officials to cordon off large areas, imposing hardship on many people and businesses. The tall building community therefore designs to reduce the probability of such occurrence.

C16.4.1.5 Members Spanning Between Structures

When members span between structures, unseating or loss of support can lead to collapse. The provisions require that unseating of structural members supported on seismically separated portions of the structure not occur by confirming that there is sufficient support length under the maximum deformation of each record in the ground motion suite or at least twice the mean movement of the structural elements. Haselton et al. (2017) discusses the reliability study that arrived at the factor of two on the median, providing approximately a 10% probability of failure.

C16.4.1.6 Structural Separation

The provisions presume that the structure is designed to not impact adjacent structures as discussed in C12.12.2. It is not expected that an explicit pounding analysis would be performed. Therefore, the provisions invoke the requirements of C12.12.2, but with the use of drifts and deformations that approximate those at the Design Earthquake level.

C16.4.2 Element-Level Acceptance Criteria

The element-level acceptance criteria requires classification of each element action as either force-controlled or deformation-controlled, similar to the procedures of ASCE 41. Note that this is done for each *element action*, rather than for each *element*. For example, for a single column element, the flexural behavior may be classified as a deformation-controlled action, whereas the axial behavior may be classified as a force-controlled action.

Deformation-controlled actions are those that have reliable inelastic deformation capacity. Force-controlled actions pertain to brittle modes where inelastic deformation capacity cannot be ensured. Based on how the acceptance criteria are structured, any element action that is modeled elastically must be classified as being force-controlled.

Some examples of force-controlled actions are

- Shear in reinforced concrete (other than diagonally reinforced coupling beams);
- Axial compression in columns;
- Punching shear in slab-column joints without shear reinforcing;
- Connections that are not explicitly designed for the strength of the connected component, such as some braces in braced frames;
- Displacement of elements resting on a supporting element without rigid connection (such as slide bearings); and

- Axial forces in diaphragm collectors.

Some examples of deformation-controlled actions are

- Shear in diagonally reinforced coupling beams;
- Flexure in reinforced concrete columns and walls;
- Axial yielding in buckling restrained braces; and
- Flexure in special moment frames.

As discussed in Section 12.3.3.4, the failure of elements supporting discontinuous walls or frames has led to significant collapses in past earthquakes. Regardless of whether component actions can be treated as deformation-controlled, any actions in elements supporting discontinuous walls or frames should be treated as force-controlled.

When constructing a nonlinear model, if any of a component's actions are not explicitly modeled as nonlinear, then all of that component's actions must be treated as force-controlled. Limiting the actions to force-controlled is necessary because the nonlinear model will not adapt to overstress in those actions as it would if those actions had been modeled explicitly with nonlinear behavior. Therefore, these actions must remain elastic in order for the analyses to be a valid representation of the force distribution within the structure.

Section 16.4.2 further requires categorization of component actions as critical, ordinary, or noncritical based on the consequence of their exceeding strength or deformation limits. Because of the differences in consequence, the acceptance criteria are developed differently for each of the above classifications of component actions. An element's criticality is judged based on the extent of collapse that may occur, given the element's failure, and is also judged on whether the effect of the element's failure on seismic resistance is substantive. An element's failure could be judged to have substantial effect on the structure's seismic resistance if analysis of a model of the building without the element present predicts unacceptable performance, while analysis with the element present does not.

Limits placed on response quantities are correlated to building performance and structural reliability. In order for compliance with these limits to meaningfully characterize overall performance and reliability, grouping of certain component actions for design purposes may be appropriate. For example, while symmetrical design forces may be obtained for symmetrical structures using equivalent lateral force and modal response spectrum analysis procedures, there is no guarantee that component actions in response history analysis of symmetrical models will be the same—or even similar—for identical components arranged symmetrically. Engineering judgment should be applied to the design to maintain symmetry by using the greater demands (that is, the demands on the more heavily loaded component determined using the appropriate factor on its mean demand) for the design of both components. For this purpose, using the mean demands of the pair of

components would not be appropriate because this method would reduce the demand used for design of the more heavily loaded component.

Though this point is perhaps trivial in the case of true symmetry, it is also a concern in nonsymmetrical structures. For these buildings, it may be appropriate to group together structural components that are highly similar either in geometric placement or purpose. The demands determined using the suite mean (the mean response over all ground motions within a suite) may be very different for individual components within this grouping. This is a result both of the averaging process and the limited explicit consideration of ground motion to structure orientation in the provisions. Although the analysis may indicate that only a portion of the grouped components do not meet the provisions, the engineer ought to consider whether such nonconformance should also suggest redesign in other similar elements. Thus, response history analysis places a higher burden on the judgment of the engineer to determine the appropriate methods for extracting and interpreting meaningful response quantities for design purposes. For

One of the design objectives for Risk Category IV structures per Section 1.3.3 is that they have a low probability of not returning to function following DE shaking intensity. One way of accomplishing this is to limit the nonlinear response of the deformation-controlled actions to a level that would not cause the building to receive a red or yellow tag. That is the intent of the Immediate Occupancy performance level in ASCE 41. Therefore, the provisions require Risk Category IV structures be evaluated for the Immediate Occupancy performance level under a suite of ground motions at the DE intensity. This can be accomplished by simply reducing the MCE_R suite by multiplying the records by 2/3rds or by selecting and scaling a suite of records to the DE spectrum.

C16.4.2.1 Force-Controlled Actions The application of the load combinations and resistance factors in Equations (16.4-2), (16.4-2), (16.4-3), and (16.4-4) are limited to load effects determined using NLRHA for structures subject to an MCE_R level event. These load and resistance factors should not be used for structural design with the design earthquake effects, E , determined in accordance with Chapter 12 of the standard. When evaluating earthquake effects in accordance with Chapter 12 of the standard, the load combinations of Chapter 2 apply.

The acceptance criteria for force-controlled actions Equations 16.4-1 and 16.4-2 follow the same framework that underlies the load combinations contained in Chapter 2 of the standard, except that both the load factors and the capacity (resistance) factors have been adjusted for consistency with the NLRHA approach and to maintain compatibility with the target reliability in Table 1.3-2. The load factor of 1.3le on seismic demands was determined assuming a demand dispersion associated with record-to-record variability of 0.3, based on values observed for several real buildings and additional modeling uncertainty of 0.2, which was selected based on engineering judgment consistent with approaches used in FEMA P695. Based on data presented in National Bureau of Standard Special Publication 577, the typical bias in resistance, (i.e. the ratio of the expected strength of an element to the nominal strength), was taken as 1.1, while a 0.15 coefficient of variation on resistance was assumed. The materials standards have resistance factors that generally vary from values of

approximately 0.7 to 0.9. An average value of 0.85 was used in computing the 1.3 load factor on seismic demand.

Exception 2 to the section permits the use of an alternative set of load combinations (16.4-3 and 16.4-4) to evaluate force-controlled actions when demands are limited by the development of a plastic mechanism. For example, shear in a column in a moment frame cannot exceed the sum of the plastic moments at the ends of the column, divided by the free height of the column. A load factor of unity is used on the capacity-limited earthquake demand in this case, recognizing the very low probability that demand can exceed the computed value and also for consistency with similar criteria in ACI 318 and AISC 341.

To determine appropriate values of λ , we begin with the collapse probability goals of Table 1.3-2 (for Risk Categories I and II) for MCE_R motions. These collapse probability goals include a 10% chance of a total or partial structural collapse and a 25% chance of a failure that could result in endangerment of individual lives. For the assessment of collapse, we then make the somewhat conservative assumption that the failure of a single critical force-controlled component would result in a total or partial structural collapse of the building.

Focusing first on the goal of a 10% chance of a total or partial structural collapse, we assume that the component force demand and component capacity both follow a lognormal distribution and that the estimate of $F_{n,e}$ represents the true expected strength of the component. We then calibrate the λ value required to achieve the 10% collapse probability goal. This value is depicted in Figure C16.4-2, which shows the lognormal distributions of component capacity and component demand.

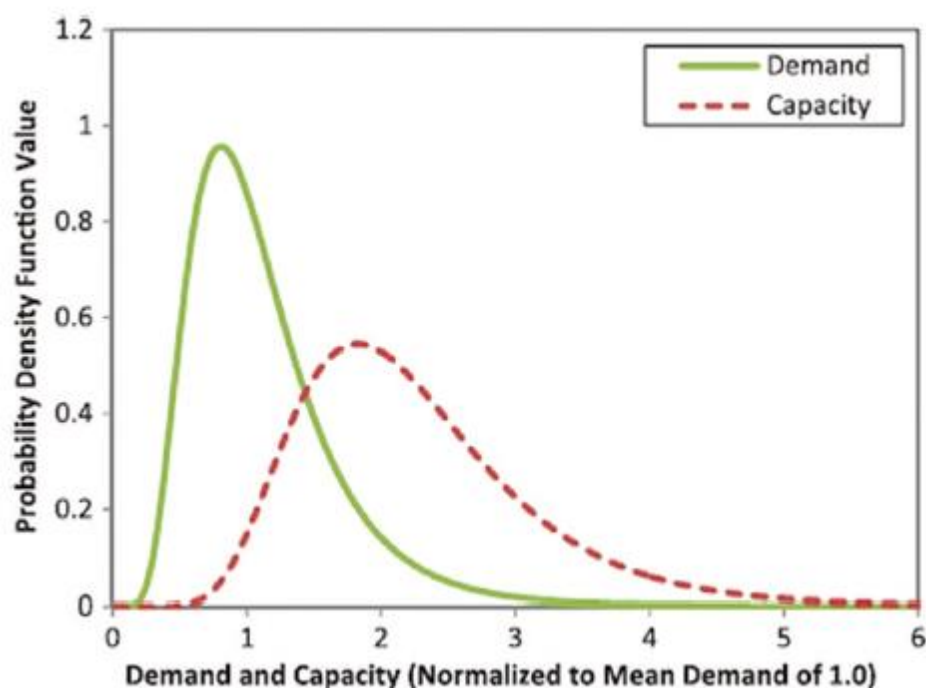


Figure C16.4-2. Illustration of component capacity and demand lognormal distributions (normalized to a mean capacity of 1.0); the mean component capacity is calibrated to achieve $P[C | MCE_R] = 10\%$.

The calibration process is highly dependent on the uncertainties in component demand and capacity. Table C16.4-3a shows typical uncertainties in force *demand* for analyses at the MCE_R ground motion level, for both the general case and the case where the response parameter is limited by a well-defined yield mechanism. Table C16.4-3b shows typical uncertainty values for the component *capacity*. The values are based on reference materials, as well as the collective experience and professional judgment of the development team.

Table C16.4-3a Assumed Variability and Uncertainty Values for Component Force Demand.

Demand Dispersion (β_D)		
General	Well-Defined Mechanism	Variabilities and Uncertainties in the Force Demand
0.40	0.20	Record-to-record variability (for MCE_R ground motions)
0.20	0.20	Uncertainty from estimating force demands using structural model
0.13	0.06	Variability from estimating force demands from mean of only 11 ground motions

0.46	0.29	$\beta_{D-Total}$
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In the calibration process, the λ and ϕ values both directly affect the required component strength. Therefore, the calibration is completed to determine the required value of λ/ϕ needed to fulfill the 10% collapse safety objective. This calibration is done by assuming a value of λ/ϕ , convolving the lognormal distributions of demand and capacity and iteratively determining the capacity required to meet the 10% collapse safety objective by adjusting λ/ϕ . Table C16.4-4 reports the final λ/ϕ values that come from such integration.

It should be clearly stated that this approach of calibrating the λ/ϕ ratio means that the final acceptance criterion is independent of the ϕ value specified by a material standard. If it is desirable for the acceptance criteria to be partially dependent on the value of ϕ , then the uncertainty factors of Table C16.4-3b would need to be made dependent on the ϕ value in some manner.

Table C16.4-3b. Assumed Variability and Uncertainty Values for Component Force Capacity.

Capacity Dispersion (β_c)		
General	Well-Defined Mechanism	Variabilities and Uncertainties in the Final As-Built Capacity of the Component
0.30	0.30	Typical variability in strength equation for $F_{n,e}$ (from available data)
0.10	0.10	Typical uncertainty in strength equation for $F_{n,e}$ (extrapolation beyond available data)
0.20	0.20	Uncertainty in as-built strength because of construction quality and possible errors
0.37	0.37	$\beta_{C-Total}$

Since the values in Table C16.4-4 are similar, for simplicity, the acceptance criterion is based on $\lambda/\phi = 2.0$ for all cases, and a separate criterion for the existence of a well-defined mechanism is not included. In addition, the strength term is defined slightly differently. For Risk Categories III and IV, this full calculation was redone using the lower collapse probability goals of 6% and 3%, respectively, and it was found that scaling the force demands by I_e sufficiently achieves these lower collapse probability goals.

Table C16.4-4. Required Ratios of λ/ϕ to Achieve the 10% Collapse Probability Objective.

Dispersion	Required Ratios of λ/ϕ
------------	-----------------------------------

General	2.1
Well-Defined Mechanism	1.9

This statistical calculation was then repeated for the goal of 25% chance of a failure that could result in endangerment of individual lives. This resulted in a required ratio of 1.5 for such force-controlled failure modes; deemed as “ordinary.”

Force-controlled actions are deemed noncritical if the failure does not result in structural collapse or any meaningful endangerment to individual lives. This occurs in situations where gravity forces can reliably redistribute to an alternate load path and no failure will ensue. For noncritical force-controlled components, the acceptance criteria allow the use of $\lambda = 1.0$.

Where an industry standard does not define expected strength, expected (or mean) strength, F_e , is computed as follows. First, a standard strength-prediction equation is used from a material standard, using a strength reduction factor, ϕ , of 1.0; the expected material properties are also used in place of nominal material properties. In some cases, this estimate of strength ($F_{n,e}$) may still be conservative in comparison with the mean expected strength shown by experimental tests (F_e) caused by inherent conservatism in the strength equations adopted by the materials standards. If such conservatism exists, the $F_{n,e}$ value may be multiplied by a “component reserve strength factor” greater than 1.0 to produce the estimate of the mean expected strength (F_e). This process is illustrated in Figure C16.4-3, which shows the $F_e / F_{n,e}$ ratios for the shear strengths from test data of reinforced concrete shear walls (Wallace et al. 2013). This figure shows that the ratio of $F_e / F_{n,e}$ depends on the flexural ductility of the shear wall, demonstrating that $F_e = 1.0 F_{n,e}$ is appropriate for the shear strength in the zone of high flexural damage and $F_e = 1.5 F_{n,e}$ may be appropriate in zones with no flexural damage.

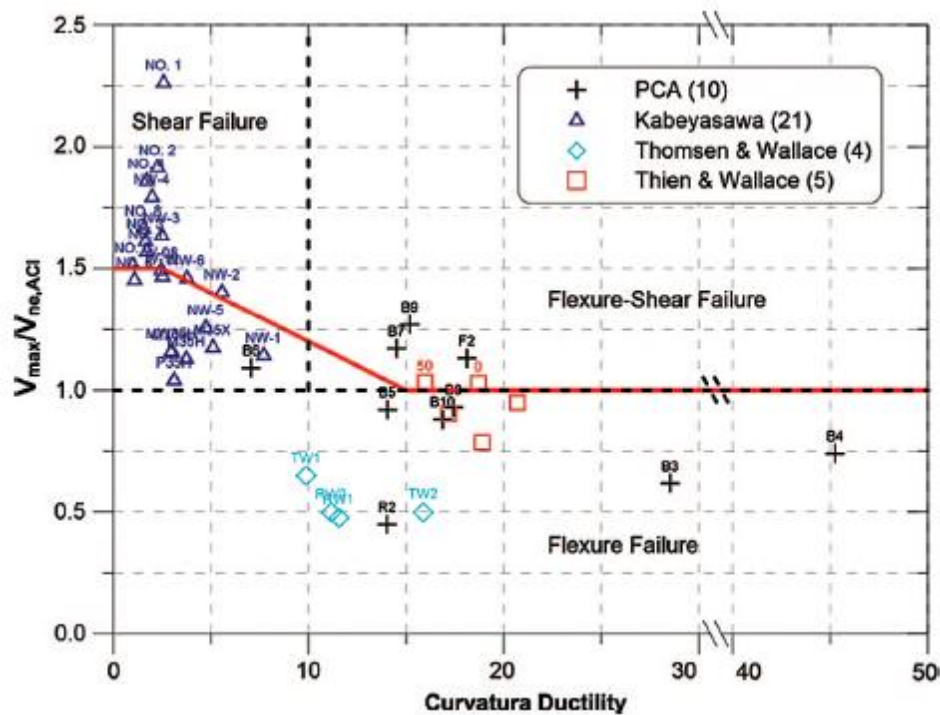


Figure C16.4-3. Expected shear strengths (in terms of $F_e / F_{n,e}$) for reinforced concrete shear walls when subjected to various levels of flexural ductility.

Source: Courtesy of John Wallace.

For purposes of comparison, Equation (C16.4-1) is comparable to the PEER TBI acceptance criteria (Bozorgnia et al. 2009) for the case that $\phi = 0.75$ and $F_e = 1.0 F_{n,e}$.

The exception allows for the use of the capacity design philosophy for force-controlled components that are “protected” by inelastic fuses, such that the force delivered to the force-controlled component is limited by the strength of the inelastic fuse.

The following are some examples of force-controlled actions that are deemed to be critical actions:

- Steel Moment Frames (SMF):
 - Axial compression forces in columns caused by combined gravity and overturning forces
 - Combined axial force, bending moments, and shear in column splices
 - Tension in column base connections (unless modeled inelastically, in which case it would be a deformation-controlled component);

- Steel Braced Frames (BRBF - Buckling Restrained Braced Frame, SCBF - Special Concentrically Braced Frames):
 - Axial compression forces in columns caused by combined gravity and overturning forces
 - Combined axial force, bending moments, and shear in column splices
 - Tension in brace and beam connections;
 - Column base connections (unless modeled inelastically)
- Concrete Moment Frames:
 - Axial compression forces in columns caused by combined gravity and overturning forces
 - Shear force in columns and beams;
- Concrete or Masonry Shear Walls:
 - Shear in concrete shear wall, in cases when there is limited ability for the shear force to transfer to adjacent wall panels. For cases of isolated shear walls (i.e., WALL #1 in Figure C16.4-4), the shear force in this isolated wall is deemed as a critical action. In contrast, the shear force in a one-wall pier that is in a group of wall piers (e.g., panel #2 of Figure C16.4-5) need not be deemed a critical action (especially when determining whether an analysis is deemed to represent an unacceptable response). For this case of a group of wall piers, it may be appropriate to consider the sum of the wall shears to be the critical action (e.g., the sum of wall shears in PANELS #1, #2, and #3 of Figure C16.4-5).
 - Axial (plus flexural) compression in concrete shear wall (for most cases)
 - Axial compression in outrigger columns
 - Axial (plus flexural) tension in outrigger column splices;
- Other Types of Components:
 - Shear forces in piles and pile cap connections (unless modeled inelastically)
 - Shear forces in shallow foundations (unless modeled inelastically)
 - Punching shear in slabs without shear reinforcing (unless modeled inelastically)
 - Diaphragms that transfer a substantial amount of force (from more than one story)

- Elements supporting discontinuous frames and walls;



Figure C16.4-4. Plan view of sample building showing arrangement of concrete shear walls.

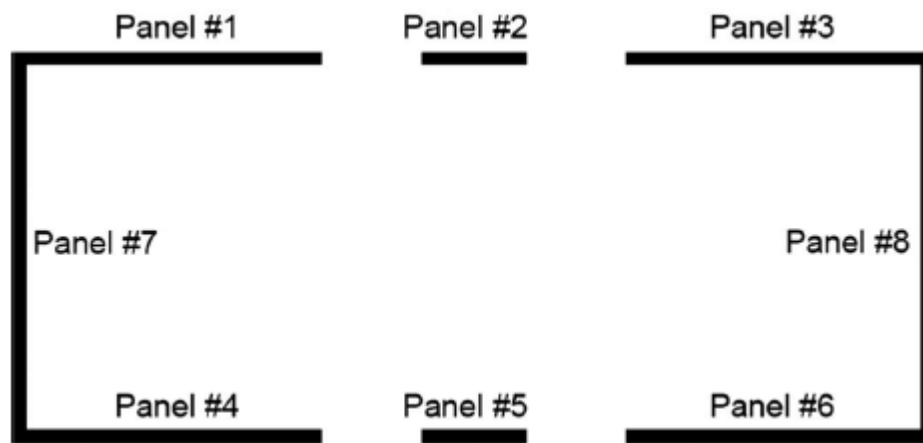


Figure C16.4-5. Plan view of sample building showing components of a reinforced concrete core shear wall.

The following are some examples of force-controlled actions that are deemed to be ordinary actions:

- Steel Moment Frames (SMF):
 - Shear force in beams and columns
 - Column base connections (unless modeled inelastically)
 - Welded or bolted joints (as distinct from the inelastic action of the overall connection) between moment frame beams and columns;
- Steel Braced Frames (BRBF, SCBF):
 - Axial tension forces in columns caused by overturning forces (unless modeled inelastically);
- Concrete Moment Frames:

- Splices in longitudinal beam and column reinforcement;
- Concrete or Masonry Shear Walls:
 - An ordinary classification would only apply in special cases where failure would not cause widespread collapse and would cause minimal reduction in the building seismic resistance.
- Other Types of Components:
 - Axial forces in diaphragm collectors (unless modeled inelastically)
 - Shear and chord forces in diaphragms (unless modeled inelastically)
 - Pile axial forces;

The following are some examples of force-controlled actions that could be deemed noncritical actions:

- Any component where the failure would not result in either collapse or substantive loss of the seismic resistance of the structure.

C16.4.2.2 Deformation-Controlled Actions

ASCE 7-16 included requirements to evaluate the adequacy of deformation-controlled actions against limiting deformation levels at which laboratory testing suggested either that element behavior was unpredictable or that loss of vertical load-carrying ability would occur. While substantive data exist to indicate the capacity of force-controlled actions, there are relatively few laboratory data to indicate the deformation at which a deformation-controlled element action reaches a level where loss of vertical load-carrying capacity occurs. There are a number of reasons for this, including the following: (1) the deformation at which such loss occurs can be very large and beyond the practical testing capability of typical laboratory equipment; (2) many researchers have tested such components with the aim of quantifying useful capacity for elements of a seismic force-resisting system and have terminated testing after substantial degradation in strength has occurred, even though actual failure has not yet been experienced; and (3) testing of gravity-load-bearing elements to failure can be dangerous and destructive of test equipment. In ASCE 7-22, this requirement was dropped in favor of an approach in which the demand imposed on deformation-controlled actions is evaluated against the valid range of modeling for that element in each analysis. The valid range of modeling is defined as that limiting deformation at which the model used in the analysis can be benchmarked against suitable laboratory test data. It is permitted to extend the valid range of modeling beyond this limiting deformation if the component's strength and stiffness are degraded to a negligible value once these maximum deformations are reached and ability of the component to carry gravity loads is verified, or the loss of gravity support is included in the computer simulation. This approach has been successfully used in the design of tall buildings, using the procedures specified in the Pacific Earthquake Engineering Research Center, Guidelines for

Performance-based Seismic Design of Tall Buildings, and was found to produce acceptable designs compared with prior practice. In addition, this approach will be facilitated with the adoption of data indicating the valid range of nonlinear modeling for deformation-controlled elements by both ACI and AISC in their standards.

C16.4.2.3 Elements of the Gravity Force-Resisting System The basic deformation-compatibility requirement of ASCE 7-16, Section 12.12.4, is imposed for gravity-system components, which are not part of the established seismic force-resisting system, using the deformation demands predicted from response history analysis under MCE_R -level ground motions, as opposed to evaluation under linear analysis.

If an analyst wanted to further investigate the performance of the gravity system (which is not required), the most direct and complete approach (but also the most time-consuming) would be to directly model the gravity system components as part of the structural model and then impose the same acceptance criteria used for the components of the seismic force-resisting system. An alternative approach (which is more common) would be to model the gravity system in a simplified manner and verify that the earthquake-imposed force demands do not control over the other load combinations and/or to verify that the mean gravity system deformations do not exceed the deformation limits for deformation-controlled components.

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COMMENTARY TO CHAPTER 17, SEISMIC DESIGN REQUIREMENTS FOR SEISMICALLY ISOLATED STRUCTURES

Revise Section C17.1 by adding the following to the end of the existing commentary:

C17.1 GENERAL

During the 2026 cycle, a group of experts in seismic isolation was convened via an issue team to deliberate and further study the topic of the probability of collapse of seismically isolated structures designed in accordance with these *Provisions*. The committee consensus is that, given MCE_R shaking, seismically isolated structures satisfying the requirements of this chapter achieve the collapse targets of Section 1.1.1 (i.e., 10% probability of collapse).

The reliability target for a seismic isolation system for a Risk Category II structure is a 10% probability of *failure* given MCE_R shaking, where failure is *conservatively* defined as the loss of function of the isolation system but not necessarily attached to building collapse, which would be associated with greater displacements and the inability of the isolators to support gravity loads.

Insufficient clearance to adjacent construction, resulting in a moderate-to-high probability of impact given MCE_R shaking, is one possible source of failure. Although impact on adjacent construction will limit the horizontal displacement of the isolation system and protect against loss of gravity-load support, it may result in high frequency, high amplitude accelerations in the isolated superstructure (Yu et al. 2024), which could be damaging for a non-ductile superstructure and non-structural components and systems.

Seismic isolation systems for buildings are generally composed of tens to hundreds of gravity-load-supporting isolators. Damping devices may also form part of an isolation system but typically do not support gravity loads. The current design requirements for seismic isolation systems include:

- a. Analysis for maximum direction ground shaking consistent with the MCE_R ,
- b. Consideration of lower-bound force-displacement hysteresis,
- c. Consideration of upper-bound force-displacement hysteresis, and
- d. Consideration of accidental torsion.

The horizontal displacement demand associated with (a), (b) and (d) is denoted D_{TM} , noting that the use of lower-bound properties will result in greater values of displacement than either best-estimate or upper-bound properties. The clearance between an isolated superstructure and adjacent construction shall not be less than D_{TM} unless impact-related calculations are performed, but no margin beyond displacement D_{TM} is specified.

Displacement D_{TM} exceeds the displacement at the center of rigidity of the isolation system, denoted D_M . The displacement D_M is calculated considering lower- and upper-bound force-displacement hysteresis but does not account for the effects of accidental torsion. An approximate ratio between D_{TM} and D_M based on the equivalent lateral force procedure in accordance with this chapter equals 1.15 for a building that is square in plan and approximately 1.25 for a building that is rectangular in plan (with one side dimension twice that of the other). Accordingly, $D_{TM} \geq 1.15 D_M$, noting that displacement D_M will be maximized using lower-bound force-displacement hysteresis.

Reliability calculations, consistent with those prepared for conventional, non-isolated structures in these *Provisions*, utilize planar framing systems and do not explicitly vary structural/material properties. Accordingly, the displacement demand for reliability calculations are computed at the center of rigidity without accidental torsion and using a *best-estimate* isolator force-displacement hysteresis. The resulting displacement will therefore be less than both D_{TM} and D_M .

The assembled issue team collected data from approximately 15 completed seismically isolated projects to characterize ranges of both D_{TM}/D_M and the ratio of D_{TM} to the displacement at the center of rigidity of the isolation system calculated using best-estimate properties, herein denoted D_M^* . Values for D_{TM}/D_M ranged between 1.1 and 1.55. Less data were available for D_{TM}/D_M^* , with a range between 1.33 and 2.1.

Prototype isolators are tested for displacement D_{TM} . Two units are tested of each type and size, including tests for maximum and minimum vertical load at displacement D_{TM} . Time coincident vertical force and horizontal displacement demands are not used for testing, with maximum and minimum axial loads for testing likely coming from analysis using upper bound properties and maximum displacements for testing coming from analysis using lower bound properties. Further, for testing of only two isolators of a given type and size, perhaps representing tens or hundreds of isolators of that type in the entire isolation system, the maximum and minimum vertical loads and the maximum displacement demands may be attached to different isolators in the isolation system. This means that a prototype isolator is tested, and its performance is assessed, for demands that are more onerous than those that might reasonably be expected during MCE_R shaking. Importantly, prototype isolators are not tested to actual failure to protect against damage to expensive, bespoke test equipment. As such, isolation system displacement capacity, as characterized using prototype tests on individual isolators to displacement D_{TM} under uncorrelated axial loads, will be underestimated, and perhaps by a wide margin.

For collapse calculations, a fragility function is routinely characterized by a lognormal distribution with a log standard deviation, β . For seismically isolated structures, which rely on seismic isolators

and/or dampers that have low uncertainty, β will be small, in the range of 0.10. Fractiles can then be computed such as median (50th), herein denoted D_{50} , and 90th, herein denoted D_{90} .

Consider an isolation system for which $D_M^* = 500$ mm and $D_{TM} = 667$ mm, calculated using a low estimate on the range for $D_{TM}/D_M^* = 1.33$. Setting $D_{50} = 500$ mm and conservatively assigning $\beta = 0.10$ results in $D_{90} = 570$ mm ($D_{90} / D_{50} = 1.14$ for lognormal distribution with $\beta = 0.10$). This D_{90} , which could represent a 10% probability of failure of the isolation system, is substantially smaller than $D_{TM} = 667$ mm, which would be the basis for prototype testing of isolators and setting the moat gap. Thus, even ignoring other conservatisms, seismically isolated structures satisfying the requirements of this chapter are expected to achieve a 10% probability of collapse.

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COMMENTARY TO CHAPTER 19, SOIL–STRUCTURE INTERACTION FOR SEISMIC DESIGN

Revise Sections C19.2 and C19.4 as follows:

C19.2 SSI ADJUSTED STRUCTURAL DEMANDS

When the equivalent lateral force procedure is used, the equivalent lateral force is computed using the period of the flexible base structure and is modified for the SSI system damping. For the modal analysis procedure, a response spectrum, which has been modified for the SSI system damping and then divided by (R/I_e) , is input into the mathematical model. The lower bound limit on the design base shear based on the equivalent lateral force procedures per Section 12.9.1.4 still applies, but the equivalent lateral force base shear modified to account for SSI effects replaces the base shear for the fixed-base case.

For both the equivalent lateral force and response spectrum procedures, the total reduction caused by SSI effects is limited to a percentage of the base shear, determined in accordance with Section 12.8.1, which varies based on the R factor. This limitation on potential reductions caused by SSI reflects the limited understanding of how the effects of SSI interact with the R factor. All of the SSI effects presented herein are based on theoretical linear elastic models of the structure and geologic media. That is why reductions of 30% are permitted for $R = 3$ or less. It is believed that those systems exhibit limited inelastic response and therefore a larger reduction in the design force caused by SSI should be permitted. For higher R factor systems, where significant damping caused by structural yielding is expected, the contribution of foundation damping is assumed to have little effect on the reduction of the response. Some reduction is permitted because of (1) an assumed period lengthening resulting from the incorporation of base flexibility, (2) potential reduction in mass participation in the fundamental mode because two additional degrees of freedom are present caused by translation and rotation of the base, and (3) limited foundation damping interacting with the structural damping.

Reductions to the response spectrum caused by the SSI system damping and kinematic SSI effects are for the elastic 5% damped response spectrum typically provided to characterize free-field motion. In addition, studies have indicated that there is a fair amount of uncertainty in the amount of kinematic SSI when measured reductions between the free-field motion and the foundation input motion are compared with the theoretical models (Stewart 2000).

Reductions for kinematic SSI effects are not permitted for the equivalent lateral force and modal response spectrum procedures. The equations for predicting the kinematic SSI effects are based on

modifications to the linear elastic response spectrum. Studies have not been performed to verify if they are similarly valid for inelastic response spectra, on which the R factor procedures are based. In addition, the amount of the reduction for kinematic SSI effects is dependent on the period of the structure, with the greatest modifications occurring in the short period range. Because the fundamental periods of most structures lengthen as they yield, what would potentially be a significant reduction at the initial elastic period may become a smaller reduction as the structure yields. Without an understanding of how the period may lengthen in the equivalent lateral force or modal response spectrum procedures, there is a potential for a user to overestimate the reduction in the response parameters caused by kinematic SSI effects. Thus, their use is not permitted.

All types of SSI effects are permitted to be considered in a response history analysis per Chapter 16. If SSI effects are considered, the site-specific response spectrum should be used as the target to which the acceleration histories are scaled. The requirement to use a site-specific response spectrum was placed in the provisions because of the belief that it provided a more realistic definition of the earthquake shaking than is provided by the design response spectrum and MCE_R response spectrum in accordance with Sections 11.4.6 and 11.4.7. A more realistic spectrum was required for proper consideration of SSI effects, particularly kinematic SSI effects. The design response spectrum and MCE_R response spectrum, in accordance with Sections 11.4.6 and 11.4.7, use predetermined factors to modify the probabilistic or deterministic response spectrum for the soil conditions. These factors are sufficient for most design situations. However, if SSI effects are to be considered and the response spectrum modified accordingly, then more accurate representations of how the underlying geologic media alter the spectral ordinates should be included before the spectrum is modified because of the SSI effects.

A site-specific response spectrum that includes the effects of SSI can be developed with explicit consideration of SSI effects by modifying the spectrum developed for free-field motions through the use of the provisions in Sections 19.3 and 19.4. If the foundation damping is not specifically modeled in the analytical model of the structure, the input response spectrum can include the effects of foundation damping. Typically, the base slab averaging effect is not explicitly modeled in the development of a site-specific response spectrum and the provisions in Section 19.4.1 are used to modify the free-field, site-specific response spectrum to obtain the foundation input spectrum. Embedment effects can be modeled directly by developing the site-specific spectrum at the foundation base level, as opposed to the ground surface. Alternatively, the site-specific spectrum for the free field can be developed at the ground level and the provisions of Section 19.4.2 can be used to adjust it to the depth corresponding to the base of foundation.

The limitations on the reductions from the site-specific, free-field spectrum to the foundation input spectrum are based on several factors. The first is the scatter between measured ratios of foundation input motion to free-field motion versus the ratios from theoretical models (Stewart 2000). The second is the inherent variability of the properties of the underlying geologic media over the footprint of the structure. Whereas there is a requirement to bound the flexibility of the soil and foundation springs, there are no corresponding bounding requirements applied to the geologic media parameters used to compute the foundation damping and kinematic SSI. The last factor is the

aforementioned lack of research into the interaction between SSI effects and yielding structures. Some studies have shown that there are reductions for most cases of SSI when coupled with an R factor-based approach (Jarernprasert et al. 2013).

Reduction of the MCER response spectrum developed using Section 11.4.6 is permitted due to both site-specific hazard studies in Section 21.1 and kinematic effects as discussed in this section. The reductions from each effect are multiplicative but must be quantified separately. There is a concern that combining SSI effects with site-specific ground motions could significantly reduce the seismic demand from that provided in Section 11.4.6. However, it was recognized that these modifications are real and reductions from each effect could be combined if there were (1) adequate peer review of the site-specific seismic hazard analysis and the methods used to determine the reductions attributable to SSI effects and (2) approval of the Authority Having Jurisdiction.

If 3D, nonlinear dynamic analysis including the stiffness of the below-grade structure is performed per Section 19.2.3, then the mesh for the geologic media should extend to an appropriate depth and horizontal distance away from the foundation to absorb outgoing seismic waves generated by the foundation. The pairs of horizontal-component input motions to be applied at the base of the kinematic SSI model should be consistent with the pairs of horizontal-component motions at the same elevation from the site response analyses used to develop the unadjusted response spectrum. Vertical-component input motions are permitted to be included in the kinematic SSI analyses so long as they are included in both the analyses with and without the stiffness of the below-grade structure modeled. In general, kinematic SSI models should not include inertial effects from the structure and therefore should not include the mass of the above- or below-grade structures. 2D plane-strain analysis is not permitted by the provisions in Section 19.2.3.

The peer review should include, but not be limited to, the following:

1. Confirmation of the process used to develop the site-specific response spectrum;
2. Confirmation that the determination of foundation stiffness and damping, including the properties of the underlying subsurface media used in the determination is appropriate; and
3. Confirmation that the base slab and first slab above the base are sufficiently rigid to allow base slab averaging to occur, including verification that the base slab is detailed to act as a diaphragm; and
4. Validating the assumptions used in the development of the soil and radiation damping ratios.

The SSI effects can be used in a response history analysis, per Chapter 16. Two options for the modeling of SSI are as follows:

1. Create a nonlinear finite-element (FE) model of the structure, foundation, and geologic media. The mesh for the geologic media should extend to an appropriate depth and horizontal distance away from the foundation with transmitting boundaries along the sides to absorb outgoing seismic waves generated by the foundation. The motion should be input at

the base of the FE model and should propagate upward as shear waves. The free-field response spectrum can be reduced for kinematic SSI only per the provisions in Section 19.4, but embedment effects would not be allowed in the reduction because the waves propagating up from the depth of the foundation to the surface would automatically include kinematic effects of embedment.

2. Create a nonlinear finite element model of the structure and foundation, with springs and dashpots attached to the perimeter walls and base of the foundation to account for the soil – foundation interaction. Guidance on the development of dashpots can be found in NIST GCR 12-917-21 (NIST 2012). The free-field response spectrum can be reduced for kinematic SSI per Section 19.4, but embedment effects may or may not be allowed in the reduction depending on whether or not (1) the motion is allowed to vary with depth along the embedded portion of the foundation, and (2) the free-field motion used as input motion is defined at the ground surface or at the bottom of the basement. The dashpots would account for the radiation and hysteretic damping of the geologic media, either per Section 19.3 or more detailed formulations

C19.4 BASE SLAB AVERAGING AND EMBEDMENT (KINEMATIC) SSI EFFECTS

Kinematic SSI effects are broadly defined as the difference between the ground motion measured in a free-field condition and the motion which would be measured at the structure's foundation, assuming that it and the structure were massless (i.e., inertial SSI was absent). The differences between free-field and foundation input motions are caused by the characteristics of the structure foundation, exclusive of the soil and radiation damping effects in the preceding section. There are two main types of kinematic interaction effects: base slab averaging and embedment. The provisions provide simplified methods for capturing these effects. The basis for the provisions and additional background material can be found in NIST GCR 12-917-21 (NIST 2012) and in Chapter 8 of Kramer and Stewart (2025).

NIST (2012) and Kramer and Stewart (2025) recommend against applying the empirical equations for kinematic effects to very soft soil sites such as E and F. These provisions allow the use of empirical equations for kinematic effects for structures on sites with $2100 \text{ ft/s} > \bar{V}_s > 700 \text{ ft/s}$, but not for softer sites or sites that are Site Class E on the basis of the soft clay layer thickness criteria given in Section 20.2.2. That is not to say that kinematic interaction effects are not present at prohibited sites, but that the empirical equations for kinematic effects should not be used; rather, more detailed site-specific assessments are permitted to be used to determine the possible modifications at those sites. Note that while there are restrictions on the applicability of the empirical equations for kinematic effects in Section 19.4 for some sites, nonlinear kinematic response history analysis including the soil and below-grade structure is permitted for all site classes per Section 19.2.3.

In addition to the prescriptive methods contained in the standard, there are also provisions that allow for direct computation of the transfer function of the free-field motion to a foundation input motion caused by base slab averaging or embedment. Guidance on how to develop these transfer functions can be found in NIST GCR 12-917-21 (NIST 2012), Kramer and Stewart (2025), and the references contained therein.

C19.4.1 Base Slab Averaging

Base slab averaging refers to the filtering of high-frequency portions of the ground shaking caused by the incongruence of motion over the base. For this filtering to occur, the base of the structure must be rigid or semirigid with respect to the vertical lateral force-resisting elements and the underlying soil. If the motions are out of phase from one end of the foundation to the other and the foundation is sufficiently rigid, then the motion on the foundation would be different from the ground motion at either end. The ground motions at any point under the structure are not in phase with ground motions at other points along the base of the structure. That incongruence leads to interference over the base of the structure, which translates into the motions imparted to the foundation, which are different from the ground motions. Typically, this phenomenon results in a filtering out of short-period motions, which is why the reduction effect is much more pronounced in structures with short fundamental periods, as shown in Figure C19.4-1.

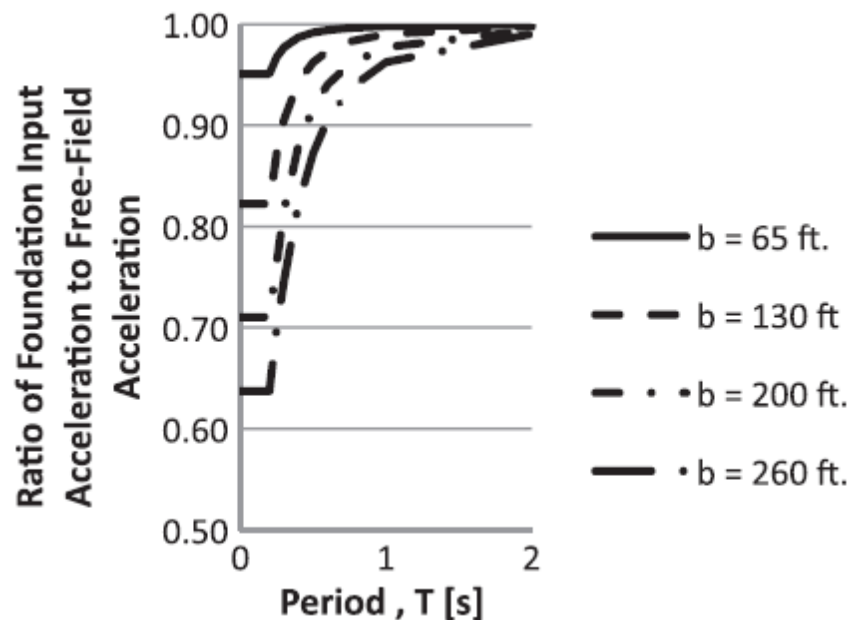


Figure C19.4-1. Example of base slab averaging response spectra ratios.

Figure C19.4-1 illustrates the increase in reduction as the base area parameter, b_e , increases. This parameter is computed as the square root of the foundation area. Therefore, for larger foundations, base slab averaging effects are more significant.

For base slab averaging effects to occur, foundation components must be interconnected with grade beams or a concrete slab that is sufficiently stiff to permit the base to move as a unit and allow this filtering effect to occur. That is why requirements are placed on the rigidity of the foundation diaphragm relative to the vertical lateral force-resisting elements at the first story. In addition, requirements are placed on the floor diaphragm or roof diaphragm, in the case of a one-story structure needing to be stiff in order for this filtering of ground motion to occur. FEMA 440 (FEMA 2005) indicates that there is a lack of data regarding this effect when either the base slab is not interconnected or the floor diaphragms are flexible. It is postulated that reductions between the ground motion and the foundation input motion may still occur. Because cases like this have not been studied explicitly, the requirements for foundation connectivity and stiff or rigid diaphragms above the foundation have been incorporated into the provisions.

The underlying models have only been studied up to an effective base size of 260 ft (79.2 m), which is why that limitation has been placed on Equation (19.4-4). NIST (2012) postulates that this effect is likely to still occur for larger base areas, but there has not been sufficient study to compare the underlying equations to data at larger effective base sizes.

Also, because the reduction can become quite significant and because studies of these phenomena have indicated variability between the theoretically predicted modifications and actual measured modifications (Kim and Stewart, 2003), a 0.75 factor is applied to the equations that are found in NIST GCR 12-917-21 (NIST 2012) and Kramer and Stewart (2025) to provide an upper bound estimate of the reduction factors with respect to the theoretical models. This is why the equations differ from those found in the references.

Lastly, the method has been developed in part from structures that include pile foundations (Kim and Stewart 2003), hence it is considered reasonable to extend the application to pile-supported structures in which the pile caps are in contact with the soil and are laterally connected to one another.

C19.4.2 Embedment

The kinematic interaction effects caused by embedment occur because the seismic motions vary with depth below the ground surface. It is common for these effects to be directly considered in a site-specific response spectrum by generating response spectra and acceleration histories at the embedded base of the structure instead of the ground surface. If that is not done, then these effects can be accounted for using the provisions in this section. However, these provisions should not be used if the response spectrum has already been developed at the embedded base of the structure. The embedment effect model was largely based on studies of structures with basements. The provisions can also be applied to structures with embedded foundations without basements where the foundation is laterally connected at the plane taken as the embedment depth. However, the provisions are not applicable to embedded individual spread footings.

As with base slab averaging, the reduction can become quite significant, and studies of these phenomena have indicated reasonable agreement between theoretically predicted modifications

and actual measured modifications (Stewart and Tileylioglu 2007). Again, a 0.75 factor is applied to the equations found in NIST GCR 12-917-21 (NIST 2012) and Stewart and Kramer (2025) to provide a slightly conservative estimate of the reductions with respect to the theoretical models. This is why the equations differ from those found in the references.

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COMMENTARY TO CHAPTER 21, SITE-SPECIFIC GROUND MOTION PROCEDURES FOR SEISMIC DESIGN

Replace Section C21.1 with the following:

C21.1 SITE RESPONSE ANALYSIS

C21.1.0 Introduction

As described in Section 11.4.3, site-specific procedures for analysis of seismic site response can be used with site-specific probabilistic seismic hazard analyses (PSHA) performed in accordance with Section 21.2 or with PSHA results drawn from the USGS Seismic Design Geodatabase (<https://doi.org/10.5066/F7NK3C76>), which is a product of the National Seismic Hazard Model (NSHM) (Petersen et al. 2023).

Site response analyses performed using the procedures in Section 21.1 are intended to capture the effects of site geotechnical conditions on ground-motion amplification and resulting surface ground-motion spectra. They require a much greater degree of site characterization effort and engineering analysis than the use of relatively simple site amplification models that are integrated into ground-motion models (GMMs), which in turn provide the ground-motion characterization components of the NSHM.

The site amplification models used with GMMs are *ergodic models* in the sense that they are derived from large global or regional data sets and capture average responses conditioned on site parameters such as V_{S30} and depths to shear wave isosurfaces ($z_{1.0}$, $z_{2.5}$). Ergodic models' main advantage is the ease of use – the site parameters are simple and can generally be obtained with relatively inexpensive investigations, and the computation of site amplification only requires using those parameters with the model equations. The disadvantage is large uncertainty in the predicted site responses, which is quantified with standard deviation terms. The practical impact of that uncertainty is that actual site amplification for a particular site can differ substantially from the mean estimate of an ergodic model. Procedures that aim to quantify site-specific effects are referred to as *non-ergodic site response* (Stewart et al. 2017; Kramer and Stewart 2025, Chapter 7). In the context of PSHA, such procedures are also referred to in some cases as *partially non-ergodic* when source and path components of GMMs are not customized for a specific application.

Section 21.1 refers to aleatory variability and epistemic uncertainty, which are considered in the formulation of site amplification models and in their application for hazard calculations. *Aleatory variability* refers to random, intrinsic variability, whereas epistemic uncertainty refers to uncertainty related to limited knowledge (e.g., McGuire 2004). In PSHA, aleatory variability is quantified using

standard deviations that account for event-to-event variability (random variations of ground motions between earthquakes, denoted τ_{lnIM}), site-to-site variability (variations of site response when an ergodic model is used, ϕ_{S2S}) and single-station variability (within-event variations in ground motion for a single station, ϕ_{SS}). When non-ergodic site response is performed, the site-to-site component of aleatory variability is removed because of the additional knowledge contributed by the site investigations and analyses. However, consideration of epistemic uncertainty is essential because the information on the site properties and site response is inevitably limited. The amount of epistemic uncertainty varies widely depending on the amount and type of information that is available for the site. Epistemic uncertainty is generally incorporated into PSHA through logic trees (McGuire 2004, Bommer and Scherbaum 2008, Bommer 2012).

Section 21.1 describes two general approaches for evaluating site amplification effects on surface ground motions: (1) traditional approach, i.e., one similar to that Sect. 21.1 in ASCE 7-22 and prior editions, but modified to address its limitations and renamed the *hybrid ground motions* approach, and (2) *site-specific site response* approach. The modifications to the traditional approach are as follows:

1. Section 21.1.1 in ASCE 7-22 indicates the site response is to be conducted for the MCE_R ground motion. In the hybrid approach, the site response also is to be conducted for 2/3 of this motion. Section 21.1.1 in ASCE 7-22 also indicates that at least 5 motions representative of the MCE_R shall be selected as the input motions to the site response analyses. In the hybrid ground motions approach the number of records for each of the two levels is at least 7.
2. Section 21.1.2 in ASCE 7-22 lists the parameters that need to be considered in the modeling of the soil column but does not state the minimum number of realizations of the soil properties that must be considered to characterize the effects of their uncertainties. In the hybrid ground motions approach, the minimum number of realizations for each ground motion is 5.
3. Section 21.1.3 in ASCE 7-22 says the recommended surface MCE_R response spectrum shall not be lower than the MCE_R response spectrum of the base motion multiplied by the average surface-to-base response spectral ratios from the site response analyses. In the hybrid ground motions approach this average surface response spectrum is to be computed for both the MCE_R ground motions and 2/3 of these motions, and the envelope of these two average response spectra shall be taken.
4. Section 21.1 of ASCE 7-22 does not require that the fundamental period of the site V_s profile used in the analysis (T_{col}) be provided in the project report, nor does it contain guidance for derivation of the surface spectrum for $T > T_{col}$. In the hybrid ground motions approach, T_{col} is to be provided in the project report, and for applications in which spectral ordinates for $T > T_{col}$ are relevant to the project, the potential for site responses involving deeper portions of the site stratigraphy is to be considered in the development of the median site amplification model.

Site-specific site response, as used in Section 21.1, is a synonym for non-ergodic site response. The procedures used to derive hybrid ground motions share many elements with those used for site-specific site response (similar site characterization and analytical modeling). However, the hybrid ground-motion approach is more approximate in the way that site amplification is used to modify reference site spectral ordinates, such that the hazard or risk level associated with the reference site hazard is not preserved. For this reason, the aleatory variability reduction (by removing site-to-site variability) is not permitted and enveloping of site amplification results derived for different intensities of input motions is required. These features of the procedure do not render the hybrid ground motions as hazard- or risk-consistent with the reference site motions, but they avoid unconservatism in most cases.

Two general approaches for site-specific site response are permitted in Section 21.1:

- Approach I: Site response is evaluated for linear conditions using ground-motion recordings at the site. If nonlinearity in site response is anticipated given the site condition and the strength of shaking, nonlinear components of the site response model are derived from ground response analyses and the results from both analyses are combined to derive the mean amplification model.
- Approach II: Site response for linear and nonlinear conditions are derived from ground response analyses.

The remainder of this commentary elaborates on the language in the Provisions, with references to resource documents where feasible for procedural details. Example applications of the procedures described in this Commentary are provided in Stewart et al. (2019).

The following definitions of terms and symbols apply to the commentary of Section 21.1 in addition to the definitions presented in the Provisions:

Epistemic Uncertainty: Uncertainty in ground-motion predictions related to limited knowledge, such as limitations in available ground-motion data, site properties data, or the capabilities of ground response analyses. More information is provided in Section C21.1.0.

Ergodic Model: Site amplification model used to estimate natural log mean (equivalent to median) site amplification for a particular ground-motion *IM* based on a site parameter (typically V_{S30}). Ergodic models provide mean amplification across a diverse set of sites conditioned in the site parameters and therefore do not capture site-specific effects, which produces large site-to-site variability. More information is provided in C21.1.0.

Ground-Motion Models: Models used to predict the statistical distribution of ground motion intensity measures (like peak acceleration or response spectral acceleration at a given percentage of critical damping and for a given oscillator period) conditional on independent variables such as magnitude, site-to-source distance, and site parameters.

Isosurface Depths: Depths from the ground surface to the first occurrence of a particular shear-wave velocity value. When that velocity is 1.0 km/s and 2.5 km/s, the depths are denoted $z_{1.0}$ and $z_{2.5}$, respectively.

Non-Ergodic Site Response: Synonym for site-specific site response.

One-Step Site Adjustment Approach: Application of ground response analyses using both a reference site profile and the target site profile in which computed surface ground motion *IMs* from both profiles are differenced (in natural log units) to derive site adjustment factors. Further information is provided in Sections C21.1.1.1 and C21.1.3.1.

Site-Response Modeling Uncertainty: Epistemic uncertainty in predicted site amplification for ground-motion *IMs* related to limitations of ground response analysis model. More information is provided in Section C21.1.4.1c.

Soil Column Period: The fundamental period of the site V_s profile considered in the analysis (denoted T_{col}), typically identified from the transfer function derived from ground response analysis. T_{col} is less than the site period when the soil column does not extend to a reference rock condition.

Soil-Over-Reference Site Approach: Application of ground response analyses using a single profile in which amplification is computed as the ratio of the surface ground motion *IM* to the input *IM*. Further information is provided in Sections C21.1.1.1 and C21.1.3.1.

Within-Event Variability: Aleatory variability of ground-motion *IMs* within earthquakes, expressed as standard deviation ϕ_{lnIM} . Generally obtained from ground-motion models.

C21.1.1 Reference Site Motion

Section 21.1 defines a *reference site* as the site condition associated with baseline ground motions against which ground motions for other site conditions are compared to evaluate site amplification. As a result, site amplification is by definition 1.0 at the reference site condition (or null in natural log units).

C21.1.1.1 Application to Site-Specific Site Response Modeling Approach

The reference site condition can be defined in three ways as given in Section 21.1.1.1. The manner by which the specific reference site condition should be defined depends on how the site response is to be computed, as follows:

- **V_{s30} -based reference site:** This is used when an ergodic model is to be applied or when site response is derived using site-specific site response Approach I (Section C21.1.0). In both cases, the reference site condition is defined based on the GMM's reference condition. Typically reference site V_{s30} values are in the range of 760-1100 m/s.

- **V_s profile-based reference site:** This is used in a particular method of applying site-specific site response Approach II that is referred to as the *one-step site adjustment approach* (Rodriguez-Marek et al. 2021a, Williams and Abrahamson 2021). As described in Section C21.1.3.1, the one-step approach considers the V_s profile for the reference site (profile implicit within the GMM used in the hazard analysis) and the V_s profile for the site under consideration (referred to as the *target site*).
- **Firm horizon within site profile:** The reference site is defined as a specific horizon within the site at which either bedrock or firm soil conditions are encountered (equivalent to Class C or stiffer); firm soil is used only when bedrock is sufficiently deep that the fundamental-mode site period exceeds the period range of interest for the project. For example, if the elastic building period is 0.5 sec and the elongated period from structural nonlinearity is about 1 sec, a firm soil reference horizon could be used if the site period is > 1 sec. This is likely to often be the case in major sedimentary basins such as the California Central Valley, Los Angeles Basin, and the Mississippi Embayment. Microtremor horizontal-to-vertical spectral ratio testing provides a convenient means by which to estimate site periods (e.g., Wang et al. 2023), especially for deep basin sites for which V_s profiles that reach rock may be impractical to develop through site testing.

If the firm horizon site condition is different from the reference condition in a GMM, the differences between the reference conditions need to be considered in the derivation of the site-specific amplification model. The firm horizon approach is a method of applying site-specific site response Approach II in which only the target site profile is considered in ground response analyses (referred to as the *soil-over-reference site approach*) (Sections C21.1.3.1 and C21.1.3.2). The V_{s30} for the firm horizon ($V_{s30,ref}$) should be defined using the procedure described in Section 21.1.1.2.

Different representations of reference site ground motions should be used depending on how the site response is to be computed, as follows:

- **GMM-derived:** Reference site ground motions are provided by GMMs at their reference site condition when an ergodic model is to be applied or when site-specific site response analysis is applied with Approach I.
- **Spectra:** Random Vibration Theory (RVT)-based methods of ground response analysis (site-specific site response Approach II) take the input motion as a Fourier amplitude spectrum, which can be derived from a response spectrum using an RVT inversion technique (e.g., Rathje et al. 2005). The reference site response spectrum, in turn, is computed from hazard analyses as described in Section 21.2. As indicated in Section 21.1.1.1, in a site-specific site response application, the reference site spectrum should, at a minimum, be developed for a 2% probability of exceedance in 50-year rate. More frequent (higher) rates can also be used to support the development of median site amplification functions over a wide range of input ground motions.

- Time series: Ground-motion time series compatible with reference site response spectra are needed for use in non-RVT-based ground response analyses, including equivalent-linear and nonlinear methods.

The spectra representations of reference site motions are permitted to consist of a response spectrum or Fourier amplitude spectrum, whereas the time series representation of reference site motions uses a response spectrum as the target for ground-motion selection and scaling. As indicated in Section 21.1.1.1, target response spectra can be derived using either Method 1 or Method 2 as specified in Sections 16.2.1.1 and 16.2.1.2, respectively. For site response applications, the Method 1 target spectrum shall be the MCE_R spectrum and is permitted to be supplemented by additional uniform hazard spectra at one or more return periods. Method 2 spectra represent scenario spectra in which matches to the MCE_R or uniform hazard spectra are applied at two or more conditioning periods.

For a given target response spectrum, time series selection and modification procedures provided in Sections 16.2.2 and 16.2.3 are permitted with two exceptions (1) 7 single-component (horizontal) input time series are permitted for ground response analyses in lieu of 11 multi-component time series for structural applications and (2) the period range for scaling or matching, as provided in Section 16.2.3.1 shall be based on the period of the soil column. Additional guidance on these issues is provided in NIST (2011), Stewart et al. (2014; Section 2.5) and Kramer and Stewart (2025; Section 4.5).

C21.1.1.2 Application with Hybrid Ground-Motion Approach

For the application of the hybrid ground-motion approach, a horizon should be identified within the site profile, below which is the reference site condition. The reference site condition should correspond to firm-ground conditions such that the effects of site response nonlinearity below the horizon will be negligible. The depth to the reference horizon is $Z_{h,ref}$ and the reference site condition time-averaged velocity ($V_{S30,ref}$) is defined over the depth interval $Z_{h,ref}$ to $Z_{h,ref} + 100$ ft (30 m).

The reference site spectrum should be developed using $V_{S30,ref}$ in conformance with the requirements for MCE_R spectra in Sections 11.4.3 or 21.2.

The additional target spectrum specified in Section 21.1.1.2 (reduced amplitude spectrum corresponding to $\frac{2}{3}$ of the MCE_R spectrum) will often produce larger site amplification for profiles that experience nonlinear site responses. Larger amplifications under $\frac{2}{3}$ of the MCE_R spectrum will occur when the site response is affected by soil nonlinearity, which increases the effects of material damping – when input ground motion amplitudes are reduced, material damping is reduced, which increases site amplification. These larger amplifications, combined with the enveloping provisions in Section 21.1.5.2, are intended to overcome unconservatism in hybrid procedures for combining site response with reference site hazard analysis (e.g., Goulet and Stewart 2009). The reason MCE_R spectra, and not uniform hazard spectra, are required is that the hybrid ground-motion approach as applied in Section 21.1.5.2 (and C21.1.5.2) directly estimates MCE_R ordinates at the ground surface without the intermediate step of evaluating ground surface hazard curves.

Some methods of ground response analysis require ground-motion time series whereas others do not (RVT-based methods). For those that require time series, 7 single-component ground motions are required for both the MCE_R spectrum and the reduced spectrum. Similar to the site-specific site response approach, no specific requirements for ground-motion selection and scaling are specified for site response applications, but NIST (2011), Stewart et al. (2014; Section 2.5) and Kramer and Stewart (2025; Section 4.5) provide guidance.

C21.1.2 Site Properties Model

A site properties model should include information on shear wave velocities, modulus reduction and damping vs shear strain (MRD) relationships, and unit weights. Shear strength may also be required in some cases.

Shear-wave velocity (V_s) should be measured using reliable techniques, which include surface wave methods, suspension logging, downhole testing, and cross-hole testing. Kramer and Stewart (2025, Section 6.5.2) describe these methods. Passive surface wave methods (i.e., without a controlled vibration source) in which a single-direction linear array is used (e.g., ReMi-based techniques; Louie 2001) should be avoided due to the potential for biased V_s values, particularly at depth (Cox and Beekman 2011). In general, site-specific site response as derived for use with Section 21.1 should not use estimated V_s profiles (e.g., from procedures described in Section C.20.3). In circumstances where V_s of individual layers cannot be measured (e.g., engineered fill placed after design is complete), V_s and its associated uncertainty should be estimated.

Variability in V_s profiles can be conceptualized, for a given depth, as the variability of V_s values (σ_{lnV}) measured at that depth if profiles were measured at multiple locations across a site footprint. Such variability is a consequence of natural variability in geotechnical deposits, even if the site layering is nominally similar between locations. If multiple V_s profiles are measured within a site, this variability can be measured. More typically, it is estimated from available models derived using data from sites with multiple measurements, as shown in Figure C21.1 and summarized by Kramer and Stewart (2025; Section 6.6.2.1).

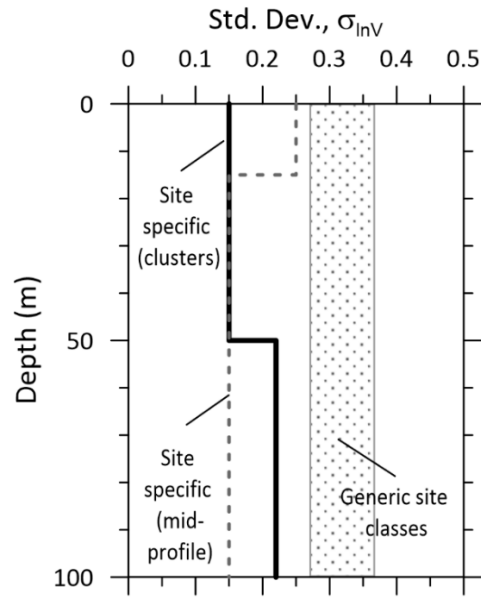


Figure C21.1. Variation of the standard deviation of V_s with depth for generic and site-specific conditions. Adapted from Toro (1995), Stewart et al. (2014), and Toro (2022). Figure from Kramer and Stewart (2025).

When surface wave testing is performed, the measured quantities are phase velocities vs. frequency or wavelength (Foti et al. 2018), which should be provided in the project report as part of the site investigation documentation (Figure C21.2a). V_s profiles are inverted from the measured phase velocities (Figure C21.2c and d), which presents challenges due to non-uniqueness (Hosseini and Pezeshk 2015; Cox and Teague 2016, Vantassel and Cox 2021). Application of site-specific constraints (e.g., depth to an impedance contrast, horizontal-to-vertical spectral ratios) during the inversion process will generally improve the quality of V_s profiles (Teague et al. 2018). For the site tested in Figure C21.2, measured horizontal-to-vertical spectral ratios were used to screen out inverted profiles that produced resonances at incorrect frequencies (Figure C21.2c).

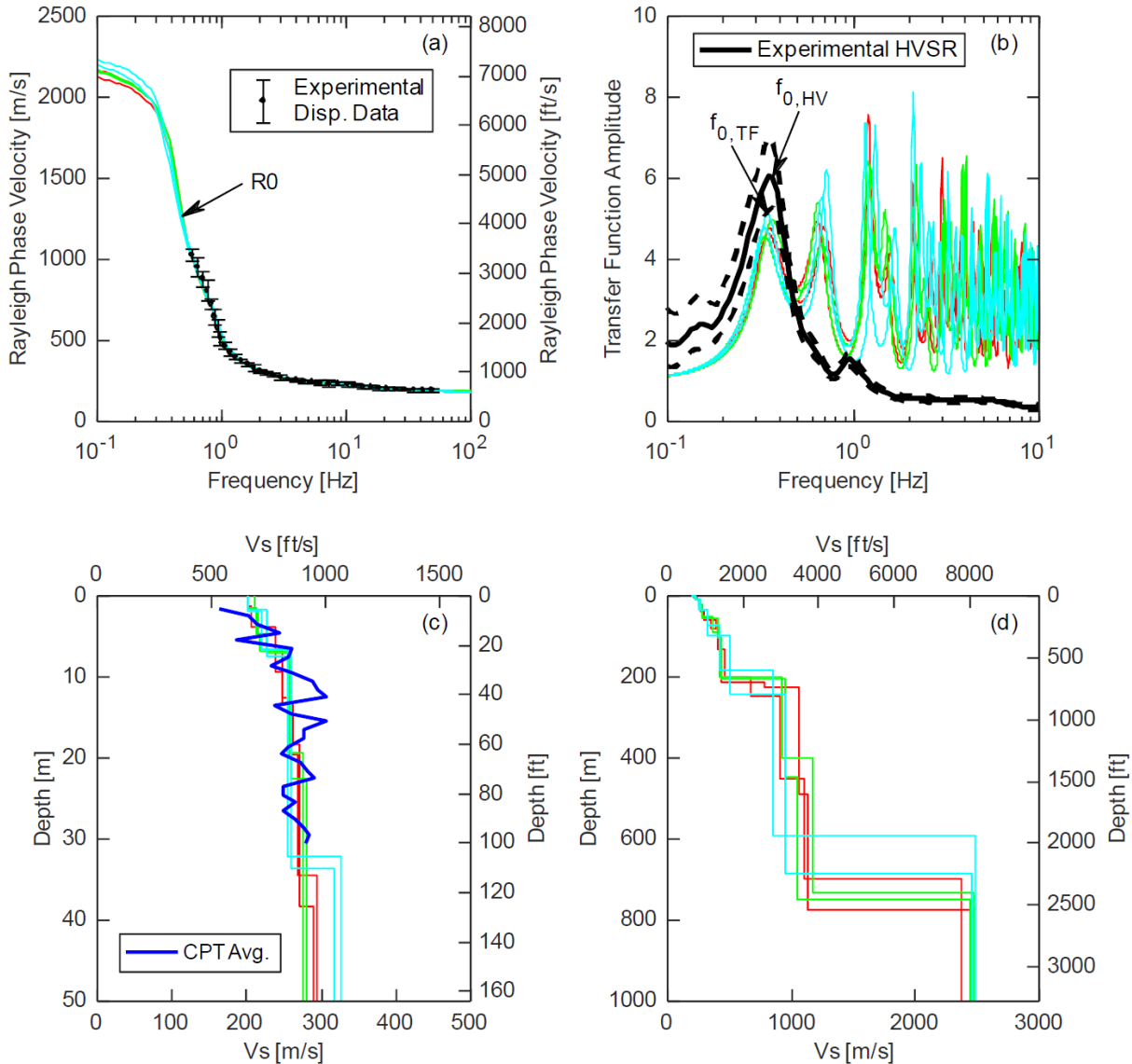


Figure C21.2. Example results of site characterization using surface wave testing, (a) Rayleigh wave phase velocities vs. frequency (data shown with black symbols; dispersion of inverted profiles in color), (b) horizontal-to-vertical spectral ratios measured (black) and theoretical transfer functions of Vs profiles from inversion (colors), (c) inverted Vs profiles with downhole data overlain for comparison, and (d) Vs profiles shown over large depth interval. Ref ENGEO (2018)

Modulus reduction curves are plots of the ratio of secant shear modulus (G) to maximum shear modulus (G_{max}) vs shear strain (Figure C21.3). These can be measured in the laboratory for soil specimens or estimated using relations in literature. Such tests also provide information on material damping (D) vs. shear strain. Stewart et al. (2014; Sections 2.2.2 and 2.3) summarize a series of models derived from such tests that can be used when index properties (e.g., PI, gradation) and stress conditions are known for a soil layer; results for one of those models (Darendeli 2001) are shown in Figure C21.3. Such models are generally calibrated for shear strain levels of about 0.8%

and less. For layers that may experience larger strains, it is important to provide constraint on the modulus reduction values so that shear stresses at large strains do not have a large misfit from the layer's shear strength. Stewart et al. (2014, Section 2.2.3) and Yee et al. (2013) provide guidelines for adjusting modulus reduction curves at large strains to align with strength values. Groholski et al. (2016) and Yniesta et al. (2017) also provide models for this behavior.

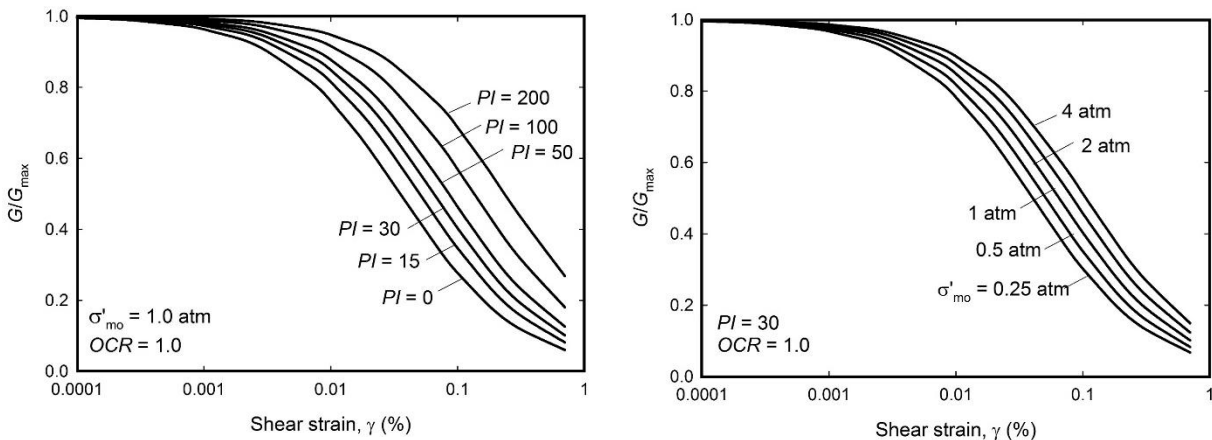


Figure C21.3. Modulus reduction curves produced by Darendeli (2001) model: (a) effects of PI at low vertical effective stress, and (b) effects of vertical effective stress for $PI=30$ soil. Figure from Kramer and Stewart (2025).

When modeling the site response at sites underlain by potentially liquefiable soils, the effects of excess porewater pressure (PWP) generation should be accounted for in the site properties model through the use of effective stress constitutive models. Guidelines and information on the current state of practice regarding effective stress site response models can be found in Matasovic et al. (2024; Section 3.4.2). Two general classes of effective stress models are often used: (1) *semi-empirical models*, where the generation of excess PWP and its effects (such as the degradation of soil stiffness and strength) are computed separately from the total stress-strain model that is initially used to model the soil; and (2) *advanced plasticity-based models*, which model cyclic effective stress behavior directly and couple volumetric and shear strains, and the development of excess PWP is computed from the resulting cyclic response.

Semi-empirical models are convenient for use owing to their implementation in commonly-used software (such as DEEPSOIL, Hashash et al. 2024), but cannot capture within-cycle phase transformation. Such behavior occurs at the point of minimum effective stress, where the shearing behavior of the liquefied soil transitions from contractive to dilative, resulting in a temporary stiffening of the soil. Phase transformation is an important phenomenon to capture in site response modeling of liquefiable soils, as it tends to limit maximum cyclic strains and affects high-frequency ground motions (Figure C21.4). Because semi-empirical models cannot capture this mechanism, their use in modeling liquefiable soils under strong ground shaking can often result in excessive and unrealistically large shear strains and overdamping in the dynamic response of the soil profile, resulting in un-conservatively low computed surface ground motions. For this reason, site response models of liquefiable soils should include the effects of phase transformation (see Section 3.4.2.3 of

the Matasovic et al. 2024 for more information about such models). If phase transformation is not included in the site response constitutive model, an explanation should be provided as to why this effect is not applicable to a given problem. If semi-empirical models are to be used instead of more advanced plasticity models, maximum shear strains should be checked and documented in order to assess the potential levels of overdamping in the model and the reasonableness of the results.

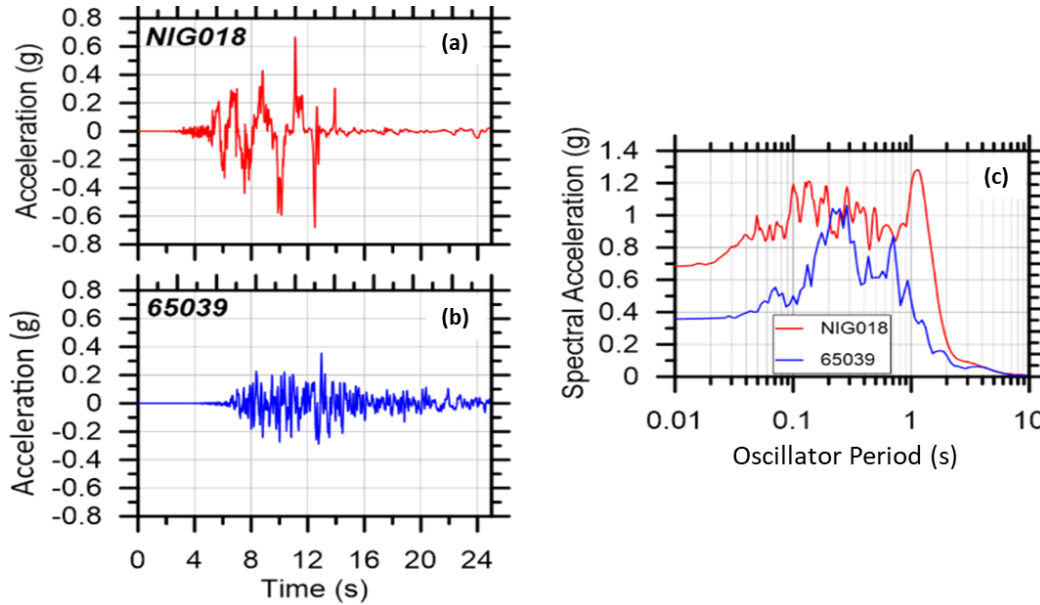


Figure C21.4. Recorded ground motions from nearby sites with (a) no liquefaction at station 65039 and (b) liquefaction at station NIG018 during the 2007 Niigata Ken Chuetsu Oki earthquake, and (c) response spectra for the two ground motions. The spikes in the NIG018 record demonstrate the occurrence of phase transformation behavior, which produces elevated oscillator responses (part c). Figure courtesy of Scott Brandenburg (with modifications)

C21.1.3 Site Response Modeling

As indicated in Section C21.1.0, Section 21.1 provides options for site response modeling using either site-specific site response procedures in Section 21.1.3.1 or hybrid ground-motion procedures in Section 21.1.3.2.

C21.1.3.1 Site-Specific Site Response Approach

The analysis of site-specific site response requires the development of a mean model for site amplification and models for the aleatory variability of site amplification. This section addresses the mean amplification; variability and uncertainty considerations are discussed in Section C21.1.4.

The mean site response in natural log units (equivalent to median site response) for site-specific analysis can be expressed as:

$$\mu_{\ln Y} = f_1 + f_2 \ln \left(\frac{x_{IMT} + f_3}{f_3} \right) \quad (\text{C21.1})$$

where μ_{lnY} is the mean site response expressed in natural log units, $f_1 - f_3$ are coefficients to be established using Approaches I or II, and x_{IMr} represents the intensity of ground shaking (in engineering units, such as fraction of g) for the reference site. In Eq. C21.1, the f_1 coefficient represents the linear site amplification (i.e., the level of site amplification that would apply for very small values of x_{IMr} and the second term on the right side of the equation represents the effects of nonlinearity. As shown in Figure C21.5, the f_2 coefficient represents the slope of the log amplification with the log of x_{IMr} and f_3 (in the same units as x_{IMr}) represents a transitional level of x_{IMr} between linear and strongly nonlinear response are depicted in Figure C21.5.

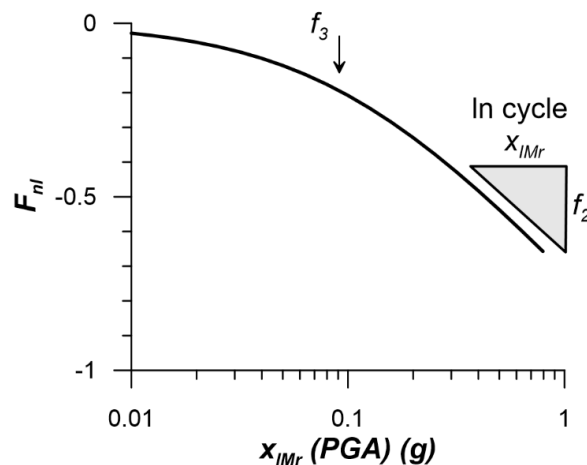


Figure C21.5. Nonlinear (second) term in site amplification function (Eq. C21.1) showing the physical meaning of the f_2 and f_3 coefficients

The coefficients in the mean model can be established using Approaches I or II.

In Approach I, ground-motion recordings from a station (i.e., accelerometer or seismometer) at the site or close to it are used to infer site response, which is derived relative to the reference site condition in a GMM (V_{S30} -based reference site, Section C21.1.1.1). Approach I requires analysis of ground motions recorded at the site and other stations that recorded the same earthquake events. Procedures for deriving site response using Approach I are provided in Stewart et al. (2017). Because the ground motions available for such applications are generally low in amplitude, these procedures are mainly applicable to the derivation of the linear component of site response. The results of those calculations are site terms (η_{sj} ; index j denotes the site) that represents the mean misfits of the data from the site relative to the ergodic site response model. The sum of η_{sj} and the ergodic model prediction for the site provides the site-specific f_1 . The nonlinear coefficients are generally established from simulations, as in Approach II.

If the ground-motion records applied in Approach I are from one or more stations that are not located at the site of interest, the applicability of these ground motions to the site of interest must be assessed. As a first-order screen, only motions on the same geological unit as that of the site should

be considered. More specific insights can be gained by performing V_s testing or microtremor horizontal-to-vertical spectral ratio (mHVSr) testing at both the target site and station site; distinct characteristics of the V_s profiles or of mHVSr vs. frequency plots should eliminate a station from consideration. Even if the above checks are performed and satisfied, due to loss of correlation with separation distance, stations further than about 1 km from the site should generally not be used unless the loss of correlation and associated uncertainties are directly considered in the evaluation of epistemic uncertainties (Section C21.1.3.1c).

In Approach II, the site properties model is used in ground response analyses to estimate site amplification relative to the reference site condition. If the reference site is defined as a firm horizon within the site profile, a soil-over-reference site approach should be used to derive the site amplification model up to the period of the modeled soil column (T_{col}). Period T_{col} is not necessarily the same as the traditional site period, which is the period of layers above a significant impedance contrast. The two periods would match if the base of the profile is also the location of a large impedance contrast. However, a traditional site period may not be defined if the profile lacks a strong impedance contrast. By specifying T_{col} as the period of the soil column that is being used in ground response analysis, it is always defined. Where an appreciable impedance contrast exists at the base of the profile, this period can be taken from the results of a linear ground response analysis as the inverse of the frequency of the first-mode response (i.e. the lowest frequency peak in the site transfer function). For cases where no such impedance contrast exists at the base of the profile, a linear ground response analysis can be performed using a stiffer reference condition at the base of the profile for the sole purpose of identifying T_{col} .

If the reference site is defined as a V_s profile, the one-step site adjustment approach can be used to derive site adjustment factors (Rodriguez-Marek et al. 2021a; Williams and Abrahamson 2021). In this approach, ground response analyses are performed both for a reference site profile and the target site profile (Figure C21.6). Computed motions at the ground surface from both profiles are differenced (in natural log units) to derive site adjustment factors. Further details on the procedures for deriving site adjustment factors using the one-step approach are provided in Rodriguez-Marek et al. (2021a) and Williams and Abrahamson (2021).

Approach II is ideally conducted for a wide range of input motions having different amplitudes (x_{IMr}), which allows the potential effects of nonlinearity in the computed amplification to be evaluated so as to constrain the coefficients in Eq. C21.1. Figure C21.7 shows an example fit of Eq. C21.1 to data points derived from simulations. For projects in which amplification is only derived for a small number of input motion levels, guidelines and tools are available to fit Eq. C21.1 (Stewart et al. 2014, Sections 2.6 and 4.2).

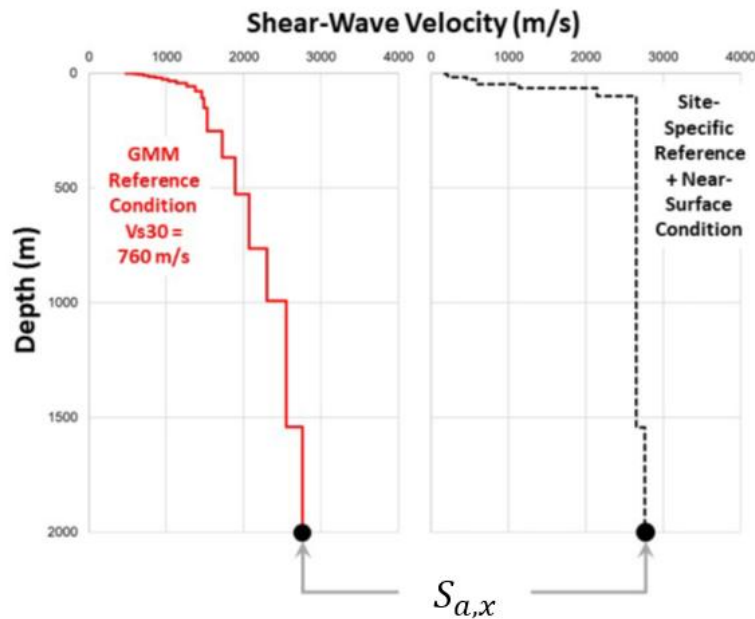


Figure C21.6. Illustration of the two profiles used in the one-step adjustment approach. The profile on the left is for a reference site condition (in this case 760 m/s) and is associated with the ergodic site response predicted for that condition. The profile on the right is a site-specific profile. The same input motion ($S_{a,x}$) is applied to both profiles, and the difference in the computed surface ground motions ($F_s^{t/h} = \ln S_{a,z}^{tar} - \ln S_{a,z}^{host}$) is used to derive site adjustment factors, which can be parameterized for range of input motion levels using Eq. C21.1. Modified from Rodriguez-Marek et al. (2021a).

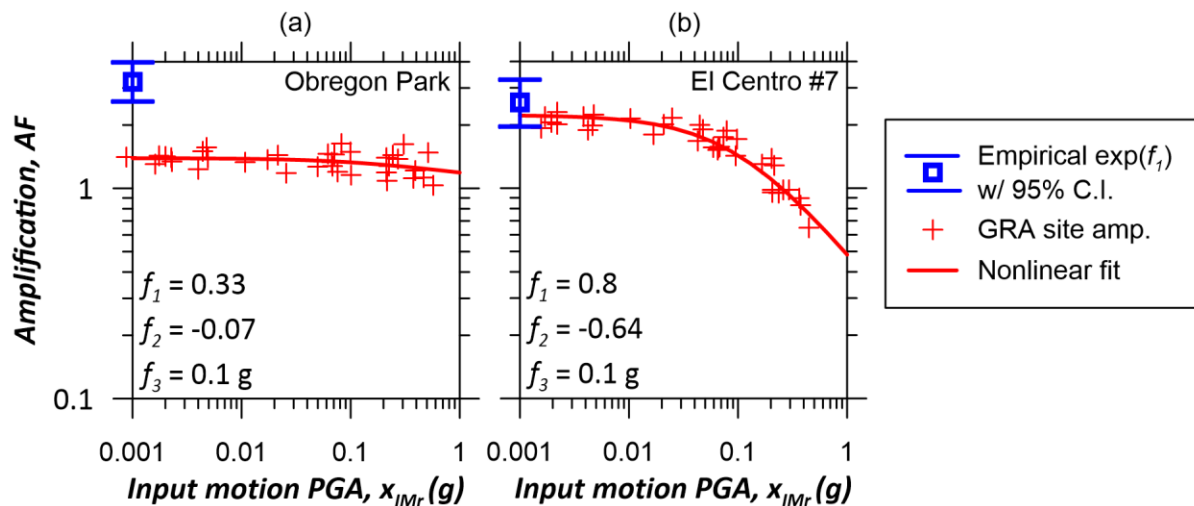


Figure C21.7. Example fits of PGA amplification (AF) to input motion amplitude (x_{IMr}) for two sites with varying levels of nonlinearity. The blue symbols indicate levels of linear site amplification derived from recordings (Approach I). Figure from Stewart et al. (2017).

Site response estimates from Approach II are only effective for $T < T_{col}$ (i.e., periods shorter than the period of the modeled soil column). For longer periods, wave propagation analyses produce very little amplification, as the wavelength is much greater than the profile thickness (the amplification approaches unity). There are two general approaches for addressing this problem:

1. Derive amplifications using Approach II up to a maximum period for which the coefficients are valid and then transition the model coefficients to those from an ergodic model (e.g., Figure C21.8). Details of this approach are provided by Stewart et al. (2014; Section 2.6).
2. Extend V_s profiles, beyond the maximum depth of direct V_s measurement, to a sufficient depth that T_{col} exceeds the maximum period of interest for the project. The profile extension should consider typical gradients (i.e., increases in V_s with depth) for the geotechnical conditions encountered near the base of the profile. There are no established best practices for such extensions, but two possibilities are: (i) to connect the measured profile to a reference profile derived for the site's V_{s30} (some such profiles are provided by Al Atik and Abrahamson 2021; Figure C21.9a) or (ii) if the depth to a firm horizon is known, apply an average profile for that condition (i.e., derived from measurements at other sites) at the depth where that horizon is encountered (Williams and Abrahamson 2021) (Figure C21.9b). There can be large uncertainties in extending V_s profiles in this manner, especially beyond period T_{col} , which need to be considered (Section C21.1.4).

Uncertainties in site amplifications derived from ground response analyses should be evaluated by using a suite of input motions (if represented by acceleration time series) and alternate realizations of soil properties represented in the site properties model. These uncertainties contribute to the spread of the data points shown in Figure C21.7, influencing the model fit using Eq. C21.1.

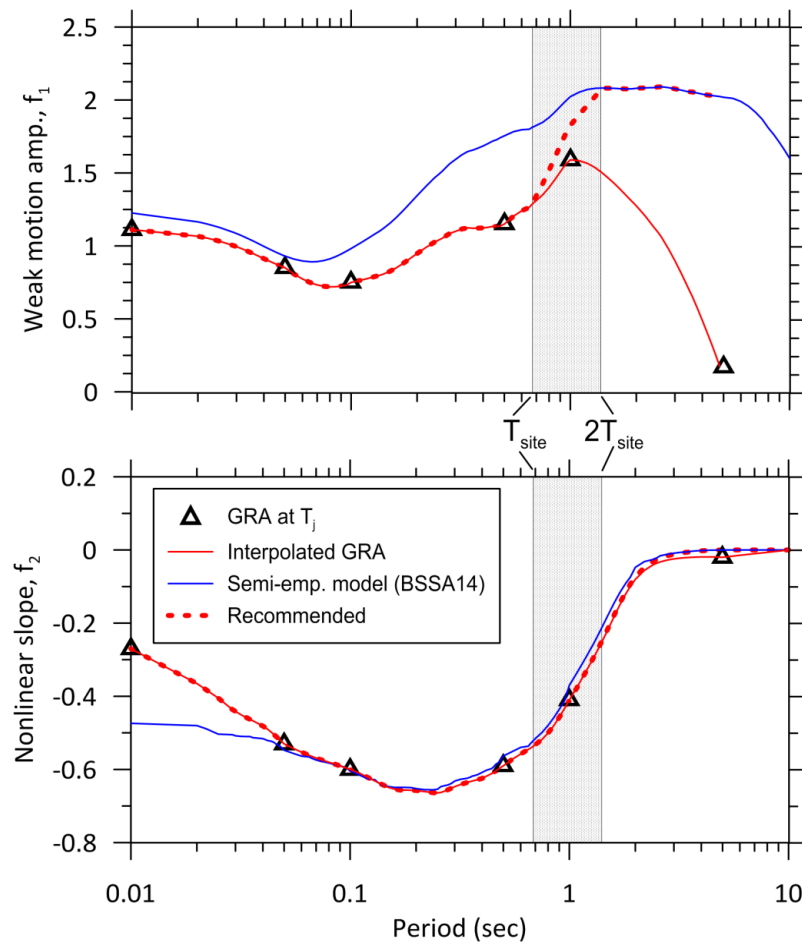


Figure C21.8. Period-dependence of coefficients f_1 and f_2 as derived from ground response analyses (GRA) and ergodic model. The recommended coefficients transition from simulation results at short periods ($T < T_{col}$) to the ergodic model at long periods ($T > 2T_{col}$).

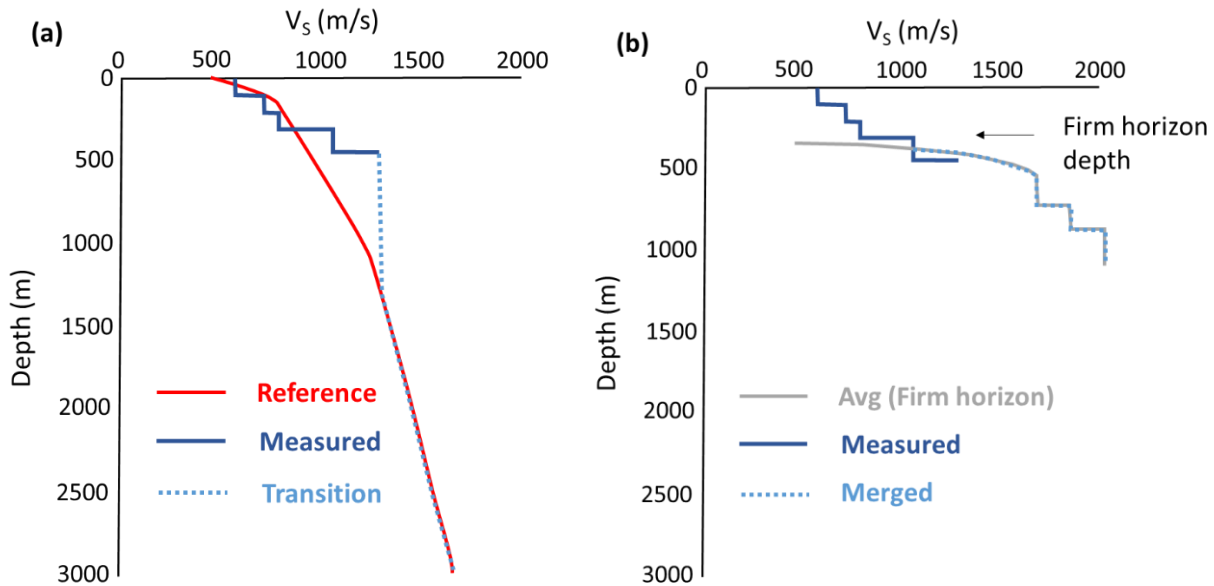


Figure C21.9. Schematic illustration of profile extension approaches. (a) Transitioning of measured profile to the reference profile for the site V_{s30} value, (b) Merging the measured profile with the average profile for firm horizon (from Williams and Abrahamson 2021) applied at the horizon depth.

C21.1.3.2 Hybrid Ground-Motion Approach

In the hybrid ground-motion approach, the site properties model is used in ground response analyses to compute site amplification relative to the reference site condition, which is defined as a firm horizon within the site profile as defined in Section 21.1.1.2. A soil-over-reference site approach can be used to derive the site amplification model up to the period of the modeled soil column (T_{col}).

Site amplification uncertainties should be quantified by performing simulations for a suite of at least 7 input motions (if represented by acceleration time series) compatible with the MCE_R spectrum (Section 21.2) and a reduced spectrum with ordinates that are $\frac{2}{3}$ of the MCE_R spectrum. For each input motion, at least 5 alternate values of soil properties shall be considered to account for the effects of their statistical uncertainties on site amplification. Those uncertainties are defined from the site properties model (Section 21.1.2). In general, it is recommended that the 5 variations of site properties include one analysis using the median V_s profile and median nonlinear and site damping properties, two analyses that vary the V_s profile while maintaining median nonlinear and site damping properties, and two analyses that vary nonlinear or site damping properties with the median V_s profile.

C21.1.4 Ground-Motion Variability and Uncertainty

Definitions of aleatory variability and epistemic uncertainty within the context of Section 21.1 are provided in Section C21.1.0. This section provides commentary on the selection of appropriate values for these variabilities and uncertainties.

C21.1.4.1 Application with Site-Specific Site Response Approach

(a) General Aleatory Variability Framework and Models

The total aleatory variability of ground motion, σ_{lnIM} , has between- and within-event components (τ_{lnIM} and ϕ_{lnIM} , respectively), which combine as follows:

$$\sigma_{lnIM} = \sqrt{\tau_{lnIM}^2 + \phi_{lnIM}^2} \quad (C21.2)$$

Source of variability for the within-event component include variable path effects and site effects, with the site contribution arising from site-to-site variability ϕ_{S2S} (i.e., systematic differences between actual site response and the ergodic site response model) and within-site variability. Variability related to path and within-site effects combine to form the single-station within-event variability, ϕ_{SS} . The total within-event variability ϕ_{lnIM} is a combination of site-to-site and single-station variability,

$$\phi_{lnIM} = \sqrt{\phi_{SS}^2 + \phi_{S2S}^2} \quad (C21.3)$$

When a site-specific site response model is used, the site-to-site variability can be removed. Accordingly, the within-event variability becomes ϕ_{SS} and the combined, single-station overall variability σ_{SS} is

$$\sigma_{SS} = \sqrt{\tau_{lnIM}^2 + \phi_{SS}^2} \quad (C21.4)$$

An exception to the removal of ϕ_{S2S} is needed for $T > T_{col}$. Due to large uncertainty in the methods used to estimate site amplification over this period range (Section C21.1.3.1), ϕ_{S2S} should be retained over this period range with a transition zone applied between the short- and long-period amplification models (i.e., $T < T_{col}$ and $T > T_{col}$, respectively).

GMMs from the NGA-East (Goulet et al. 2021) and NGA-Sub (Bozorgnia et al. 2022) projects, which are used in the NSHM, include models for τ_{lnIM} and ϕ_{SS} . The GMMs for active tectonic regions from the NGA-West2 project (Bozorgnia et al. 2014) did not provide models for ϕ_{SS} , but such models have been produced after that project (Al Atik 2015, Geopentech 2015), some results of which are shown in Figure C21.10. Note the substantially reduced aleatory variability when ϕ_{SS} is used in place of ϕ_{lnIM} .

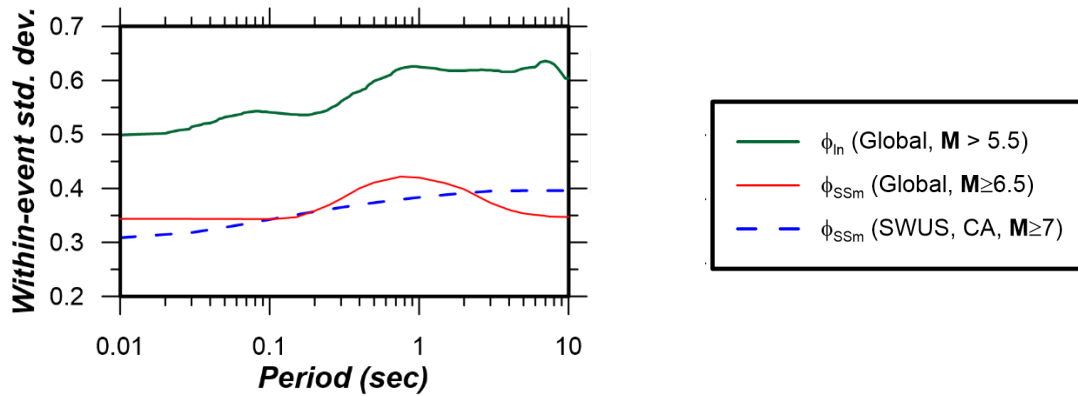


Figure C21.10. Single-station standard deviation models (ϕ_{SSm}) developed for South-Western US (SWUS) project (GeoPentech, 2015) and for global applications (Al Atik 2015). Boore et al. (2014) (BSSA14) ϕ_{lnIM} model shown for comparison (plotted results applicable for $M > 5.5$, $R_{JB} < 79$ km, and $V_{S30} > 300$ m/s).

(b) Site Response Variability Estimation and Application

Site response variability, denoted ϕ_{lnY} , quantifies the effects of different site responses between events. It is one of the contributors to within-site variability, which is a component of ϕ_{SS} . This variability has been quantified in several previous investigations using ground-motion data (Lin et al. 2011, Rodriguez-Marek et al. 2011, Kaklamanos et al. 2013). A synthesis by Stewart et al. (2017) shows that the results are consistent between studies and across periods, with ϕ_{lnY} being in the range of 0.2-0.3 in natural log units.

As part of the site-specific site response model development, this source of variability can be evaluated from variations of individual site response results relative to the mean model (e.g., the spread of the data points in Figure C21.7). Due to variability in nonlinear soil properties (modulus reduction, damping levels beyond the minimum damping, shear strength), ϕ_{lnY} tends to increase as the intensity of shaking increases (Figure C21.11). This increase, denoted $\Delta\phi_{lnY}$, is important to consider in the formulation of the ground-motion within-event variability at the ground surface.

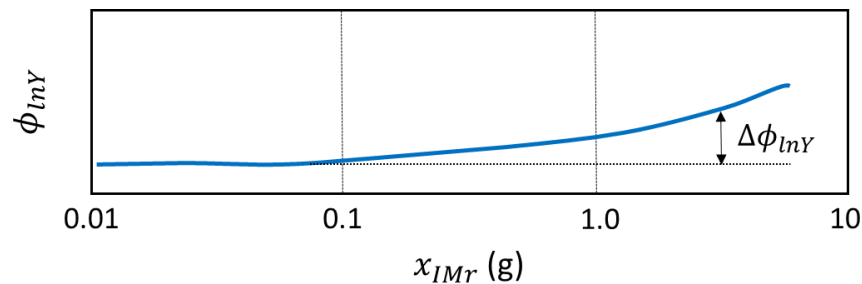


Figure C21.11. Change of site response between-event standard deviation with ground-motion intensity due to additional sources of variability related to nonlinear components of the site response model. Adapted from Rodriguez-Marek et al. (2021a).

Nonlinear site response is known to affect the within-event variability of surface ground motions, based on theoretical considerations and empirical observations (e.g., Bazzurro and Cornell 2004, Boore et al. 2014). When the site amplification is represented using the nonlinear function in Eq. C21.1, the surface ground-motion within-event variability is given by (adapted from Stewart et al. 2017),

$$\phi_{lnZ} = \sqrt{\left(\frac{f_2 x_{IMr}}{x_{IMr} + f_3} + 1\right)^2 \phi_{SS}^2 + \Delta\phi_{lnY}^2} \quad (C21.5)$$

The first squared term inside the radical represents the modification of the input motion standard deviation from nonlinearity – because f_2 is generally negative, this term will be less than unity, hence the impact is uncertainty reduction. The $\Delta\phi_{lnY}$ term represents the added uncertainty from ground-motion variability contributed by site response nonlinearity. The full value of ϕ_{lnY} need not be used because the linear component is already included in ϕ_{SS} .

As described in Section C21.1.5, depending on the method of hazard implementation, either $\Delta\phi_{lnY}$ or the within event variability that includes its impact (ϕ_{lnZ}) is applied.

(c) Site Response Epistemic Uncertainty

It is essential to consider epistemic uncertainty when a site-specific site response model is used. Epistemic uncertainties are often expressed using standard deviation terms (σ_{ep}), but these terms should not be confused with those for aleatory variability. Once an epistemic standard deviation has been derived, the distribution can be sampled using procedures from Miller and Rice (1983) to define logic tree branches (Rodriguez-Marek et al. 2021b). There are two main sources of epistemic uncertainty in the site response model that should be considered:

1. Uncertain soil properties as used in ground response analyses. Rodriguez-Marek et al. (2021b), Stewart and Afshari (2021), and Ulmer et al. (2021) describe the distinction between epistemic uncertainty and aleatory variability in site properties. Examples of epistemic uncertainties include variations among V_s profiles measured using alternate reliable techniques at nominally the same location and shear strength estimates derived from correlations (the epistemic uncertainty would be the standard deviation of the correlation model in this case).
2. Model error, which represents the uncertainty introduced by assuming that site response can be modeled with ground response analysis when actual responses are more complex (inclined waves, surface waves, laterally heterogeneous soil conditions). A model for model error was developed by Stewart and Afshari (2021), which is shown in Figure C21.12 along with results from several additional studies that confirm the model estimates. Values are generally in the range of 0.25-0.3 over a wide range of oscillator periods. The reduction in the uncertainty in the model error model near 1.0 sec can in general be applied at T_{col} . This

reduction accounts for the reduced site response, and site response uncertainty, as derived from ground response analyses for periods exceeding T_{col} .

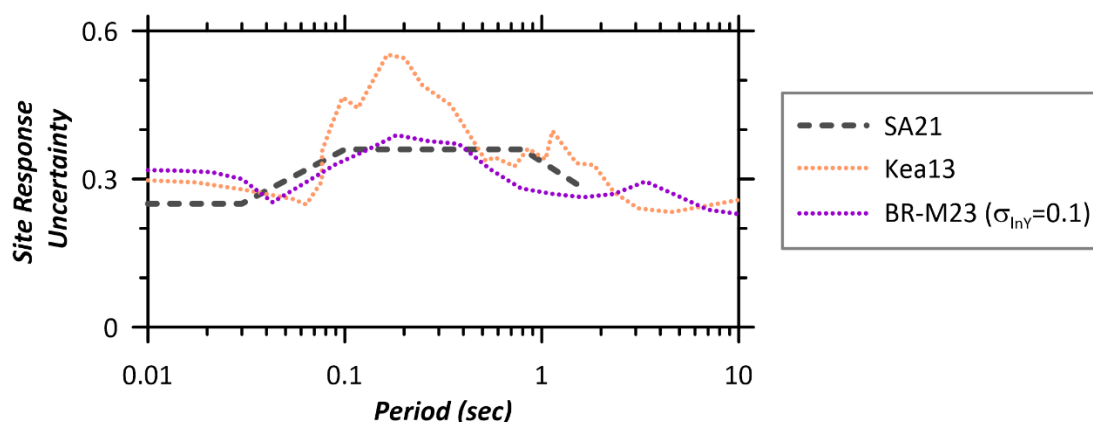


Figure C21.12. Model for model error from Stewart and Afshari (2021), derived using data from California, as compared to estimates derived using Japanese data by Kaklamanos et al. (2013) and Bahrapouri and Rodriguez-Marek (2023) (the latter result was derived from the results in their study using $\sigma_{lnY}=0.1$).

C21.1.4.2 Application with Hybrid Ground-Motion Approach

As indicated in Section C21.1.0, the hybrid ground-motion approach makes several approximations in the implementation of site response with the reference site hazard. These approximations align this method of analysis reasonably closely with the Provisions of Section 21.1 prior to the 2026 update and are easier to apply. However, they also cause the hazard and risk levels of the resulting ground motions to become mismatched from those of the reference site. Consequently, it was decided that the aleatory reduction associated with removing ϕ_{S2S} is not justified.

For this reason, when the hybrid ground-motion approach is applied, the full aleatory variability (Eq. C21.2) should be used in hazard calculations for the reference site. Both the between-event and within-event standard deviations can be taken from the GMMs used in the hazard calculation. The site-response uncertainty described in Section C21.1.4.1b is not directly quantified as in site-specific site response, but it is indirectly considered by using at least 11 single-component input motions for each target spectrum and alternate realizations of soil properties from the soil properties model (Section C21.1.3.2).

C21.1.5 Analysis of Ground-Motion Hazard Including Site Response

C21.1.5.1 Application with Site-Specific Site Response Approach

There are two general methods for combining a site amplification model with reference site ground motions to produce hazard curves for the ground surface site condition that preserve target hazard and risk levels – convolution and modification of the GMM in the hazard integral.

The convolution approach (Bazzurro and Cornell 2004) combines the hazard curve for the reference site with the statistical distribution of site response (using the mean model from Eq. C21.1 and $\Delta\phi_{lnY}$ from Section C21.1.4.1). Spreadsheet tools for implementation are available (Stewart et al. 2014; Sections 1.2.5 and 4.2.3).

The reference site hazard curve should be derived using the single-station standard deviation (Eq. C21.4). The 2023 NSHM does not include hazard results in which σ_{SS} is used, but these results may be included in future versions, which would obviate the need to conduct reference site hazard calculations as part of implementing site-specific site response. Epistemic uncertainties that should be considered are those related to alternate GMMs, the seismic source characterization model, and site response (Section C21.1.4c).

Modification of the GMM in hazard calculations involves retaining the source and path components of the model, removing the site response model (which is ergodic), and replacing it with the site-specific mean model (Eq. C21.1). The standard deviation that should be used with the GMM is a combination of the GMM's τ_{lnIM} model without modification and a modified within-event standard deviation as given by Eq. C21.5. ϕ_{SS} can be taken from available models (e.g., Figure C21.10). PSHA is then performed using the modified GMM. The uncertainties that should be considered in this process are the same as with convolution.

For both approaches, outcomes of the calculations are hazard curves for the reference site that account for site response. The hazard curves can be sampled to produce uniform hazard spectra or convolved with structural fragility curves to produce MCE_R spectra (Section 21.2).

C21.1.5.2 Application with Hybrid Ground-Motion Approach

To illustrate the application of the hybrid ground-motion approach in Section 21.1.5.2, Figure C21.13a shows a uniform hazard spectrum (475-year return period) for a reference site condition ($V_{S30} = 760$ m/s) with a gray line. The x_{IMr} metric is PGA and the PGA_r value for the 475-year return period is 0.43 g. The site has a V_{S30} of 183 m/s. Computed median site amplifications for 0.43 g and $\frac{2}{3}$ of that value are presented in Figure C21.13b, along with an envelope of the two results, which is required for application (Section 21.1.5.2). For $T < 1.3$ sec, the weaker input motion produces the stronger site amplification, and controls the enveloped response. For periods larger than the elastic site period ($T_{col} = 1.0$ sec) the stronger input motion controls the enveloped site amplifications for a range of periods from 1.3 to 2.0 sec.

The black dashed line in Figure C21.13a shows the adjusted spectrum to account for site response using the enveloped amplification, as required by the hybrid ground-motion approach. Also shown for comparison is the UHS for the soil site condition in blue that is obtained from convolution. The results do not match perfectly, but the comparison is much better than if only the stronger input (0.43 g) was used for the site response calculations. Comparisons for many sites like the one shown in Figure C21.13 led to the selection of the $\frac{2}{3}$ factor and the use of enveloping.

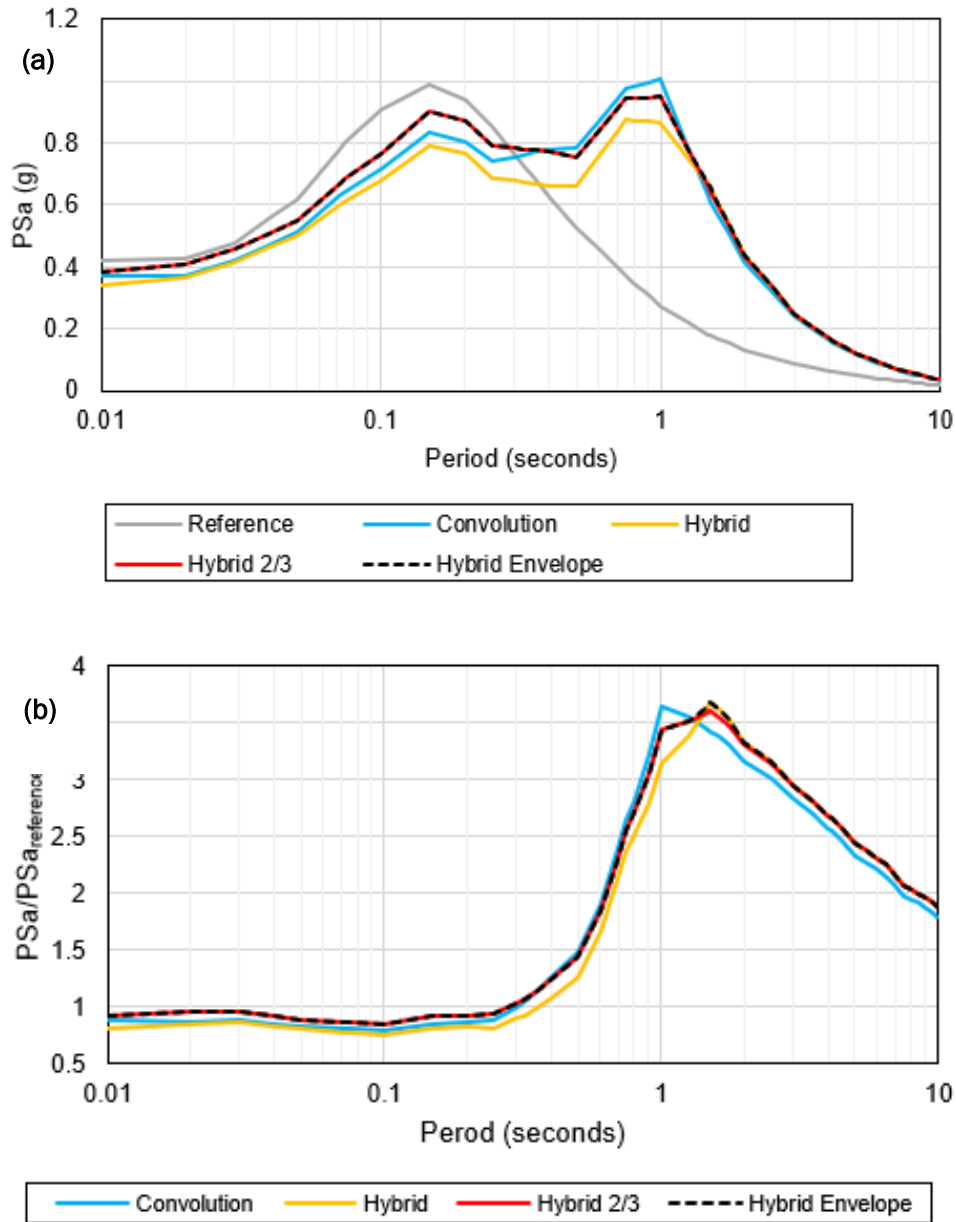


Figure C21.13. Illustration of hybrid ground-motion approach. (a) reference rock uniform hazard spectrum for 475 year return period, spectrum adjusted for site response using the hybrid procedure, and hazard-consistent uniform hazard spectrum including site response for comparison purposes; (b) period-dependent site amplification as derived using x_{IMr} (PGA) for a 475-year return period (0.43 g) and $2/3$ of 0.43 g, along with enveloped site amplification.

Revise Section C21.2 as follows:

C21.2 RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION HAZARD ANALYSIS

Site-specific risk-targeted maximum considered earthquake (MCER) ground motions are based on separate calculations of site-specific probabilistic and site-specific deterministic ground motions.

Both the probabilistic and deterministic ground motions are defined in terms of 5% damped spectral response in the maximum direction of horizontal response. The maximum direction in the horizontal plane is considered the appropriate ground motion intensity parameter for seismic design, using the equivalent lateral force (ELF) procedure of Section 12.8, whose primary intent is avoiding collapse of the structural system.

Most ground motion relations are defined in terms of average (geometric mean) horizontal response. Maximum response in the horizontal plane is greater than average response by an amount that varies with period. Maximum response may be reasonably estimated by factoring average response by period-dependent factors of 1.2 at periods less than or equal to 0.2 s, by 1.25 for a period of 1.0 s, and by 1.3 for periods greater than or equal to 10 s (Resource Paper 4, NEHRP Provisions 2015, which is based on Shahi and Baker 2013). The maximum direction was adopted as the ground motion intensity parameter for use in seismic design, in lieu of explicit consideration of directional effects.

When performing a ground motion hazard analysis, it is important to consider the regional tectonic setting, geology, and seismology. For the expected recurrence rates on known faults, this may include accounting for the elapsed time since the most recent large earthquake (e.g., Working Group on Utah Earthquake Probabilities, 2016) and/or fault ruptures that occur synchronously (e.g., Wong & Thomas, 2024 for the Wasatch and West Valley faults in Utah). An important geologic consideration is topography, which has been shown to contribute significantly to the effects local geology has on ground motions by increasing the amplitude of ground motions relative to flat ground conditions and causing damage to structures near the crests of slopes (e.g., Assimaki et al. 2005). Because most existing ground motion models do not account for the effects of site topography (Rai et al. 2017), the ground motion hazard analysis should consider topographic effects where appropriate. Topographic amplification is typically considered significant where all of the following are present: (1) slope angles greater than about 17 degrees (Bouckovalas and Papadimitriou 2005), (2) the horizontal or vertical dimensions of the topographic feature are comparable to the seismic wavelengths of interest (e.g., the shear wave velocity of the feature divided by the frequencies that impact structural response), and (3) the location of interest is within a horizontal distance approximately equal to one wavelength from the slope crest. Topographic amplification can be estimated by semi-analytical solutions (e.g., Bouckovalas and Papadimitriou 2005), numerical modeling (e.g., Jeong et al. 2019), laboratory experiments (e.g., Jeong et al. 2019), field experiments (e.g., Wood and Cox 2016), and ground motion models (e.g., Rai et al. 2017). Engineers should be

cognizant of the large degree of spatial variability in the effects of surface topography on ground motions (Wood and Cox 2016) and the limitations, applicability, and uncertainty of each method of analysis.

Revise Section C21.2.3 as follows:

C21.2.3 Site-Specific MCE_R Response Spectrum

The MCE_R response spectrum may be taken as equal to the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase for the applicable site class without performing site-specific calculations. If the site-specific MCE_R spectral response acceleration at any period is derived using hybrid ground motion procedures as defined in Section 21.1 or site-specific ground motion hazard analyses per Section 21.2, it shall not be taken as less than 80% of the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase for the applicable site class. If the site-specific site response procedures are applied as described in Section 21.1, the MCE_R response spectral accelerations cannot be taken as less than 70% of the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase for the applicable site class.

These requirements recognize the earthquake hazard and ground motion expertise reflected in the National Seismic Hazard Model and rely on the MCE_R ground motions available in the USGS Seismic Design Geodatabase (1) to provide an acceptable alternative to calculating site-specific ground motions (which can be a burden for smaller projects), and (2) to establish a “safety net” on site-specific ground motions.

Eighty percent of the MCE_R response spectrum obtained from the USGS Seismic Design Geodatabase was established as the lower limit to prevent the possibility of approximate site response implementations that produce hybrid ground motion (as defined in Section 21.1) generating unreasonably low ground motions from potential misapplication of site-response procedures or misinterpretation or mistakes in the quantification of the basic inputs to these procedures. When site-specific site response procedures, as defined in Section 21.1, are applied to derive mean site response, its uncertainty, and the associated aleatory variability, which are then implemented in a manner that approximately preserves probability and risk levels, the allowable lower limit is reduced to 70%. The larger allowable reduction was selected in consideration of the greater level of site-specific knowledge of ground motions, the greater precision of their implementation in hazard calculations, and the requirement for peer review when such procedures are used.

In the case of Site Class F, an 80% lower limit is considered reasonable because of the greater uncertainty associated with modeling site response in soils that experience severe strength loss and phase transformation behavior, such as liquefiable soils. An exception has been introduced at the end of this section to permit a site class other than E to be used in establishing this limit when a site is classified as F. This revision eliminates the possibility of an overly conservative design spectrum on sites that would normally be classified as Site Class C or D.

For reasons discussed in Section C11.4.3, a requirement is added that in the Central and Eastern United States, the lower limit for Site Classes DE and E sites should be based on 80% of the MCE_R response spectrum obtained from the USGS web service for Site Class D, as opposed to the values for Site Classes DE or E.

Revise Section C21.4 as follows:

C21.4 Design Acceleration Parameters

The criteria of Section 21.4 are based on the premise that the value of the parameter a_g should be taken as 90% of peak value of site-specific response spectral acceleration, regardless of the period (greater than or equal to 0.2 s) at which the peak value of response spectral acceleration occurs. Consideration of periods beyond 0.2 s recognizes that site-specific studies (e.g., softer site conditions) can produce response spectra with ordinates at periods greater than 0.2 s that are significantly greater than those at 0.2 s. Periods less than 0.2 s are excluded for consistency with the 0.2 s period definition of the short-period ground motion parameter, and recognizing that certain sites, such as Central and Eastern United States (CEUS) sites, could have peak response at very short periods that would be inappropriate for defining the value of the parameter a_g for the reasons discussed in Commentary Section C11.4.5. The upper bound limit of 5 s precludes unnecessary checking of response at periods that cannot govern the peak value of site-specific response spectral acceleration. Ninety percent (rather than 100%) of the peak value of site-specific response spectral acceleration is considered appropriate for defining the parameter a_g (and the domain of constant acceleration) because most short-period structures have a design period that is not at, or near, the period of peak response spectral acceleration. Away from the period of peak response, response spectral accelerations are less, and the domain of constant acceleration is adequately described by 90% of the peak value. For those short-period structures with a design period at, or near, the period of peak response spectral acceleration, anticipated yielding of the structure during ground motions effectively lengthens the period and shifts dynamic response to longer periods at which spectral demand is always less than that at the peak of the spectrum.

Revise Section C21.5 as follows:

C21.5 MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN PEAK GROUND ACCELERATIONS

Site-specific requirements for determination of MCE_G peak ground acceleration (PGA_M) are provided in Section 21.5, which is parallel to the procedures for developing site-specific response spectra ($MCE_R S_{aM}$) in Section 21.2. Like the Section 21.2 procedures, the Section 21.5 requirements changed from ASCE 7-22 to the 2026 NEHRP Provisions. Deterministic capping of PGA_M was replaced with an increase in the specified uniform-hazard probability of exceedance, from 2%-in-50yrs to 3%-in-50yrs. Candidate increases in the specified exceedance probability ranged from factors of 1 to 2, but the selected 1.5 increase impacted the PGA_M values by the most acceptable amount, as demonstrated below. This increase in the specified probability of exceedance of PGA_M

parallels the Section 21.2 increase in the target risk of collapse for S_{aM} , from 1%-in-50yrs to 1.5%-in-50yrs. As deterministic capping of PGA_M resulted in exceedance probabilities above 2%-in-50yrs (e.g., 13%-in-50yrs at the San Jose location in Table C21.5-1), the newly specified 3%-in-50yrs represents an increased exceedance probability for some locations and site classes but a decrease for others.

The impacts of replacing the ASCE 7-22 deterministic capping of $MCE_G PGA_M$ with the increased 3%-in-50yrs probability of exceedance are demonstrated in Figure C21.5-1 and Table C21.5-1, using the same 36 city locations and default site conditions considered in the Chapter 22 Commentary. In the top and bottom panels of the figure, respectively, the impacts are illustrated without and with the update from the 2018 to 2023 USGS National Seismic Hazard Model (NSHM), which is explained in the Chapter 22 Commentary. From the two figure panels and corresponding table, note that replacing the ASCE 7-22 deterministic capping with the 3%-in-50yrs exceedance probability ...

- does not change the $MCE_G PGA_M$ values by more than 20% at 28 of the 36 city locations.
- decreases the $MCE_G PGA_M$ values by more than 20% at the Las Vegas, Charleston, Chicago, New York, and Washington DC locations. However, when combined with the update to the 2023 USGS NSHM, only the values at Las Vegas and Charleston decrease by more than 20%. At Las Vegas, the PGA_M decreases by 26%, from 0.34g to 0.25g, but if deterministic capping had been retained, the 2023 NSHM update alone would have decreased the PGA_M by 9%. At Charleston, the PGA_M decreases by 29%, from 0.97g to 0.69g, but the 2023 NSHM update alone would have decreased the PGA_M by 8%.
- increases the $MCE_G PGA_M$ values by more than 20% at the Riverside, San Francisco, and San Jose locations, where the ASCE 7-22 deterministic capping reduced the corresponding probabilistic (2%-in-50yrs) PGA_M values by between 27% and 43%, resulting in exceedance probabilities between 5.5%-in-50yrs and 13%-in-50yrs. When combined with the update to the 2023 USGS NSHM, the increases in $MCE_G PGA_M$ at these three city locations remain >20% (i.e., 26%, 30%, and 59%, respectively). If deterministic capping had been retained, the 2023 NSHM update alone would have increased the $MCE_G PGA_M$ values at Riverside, San Francisco, and San Jose by 3%, 16%, and 77%, respectively.

At additional site locations and for Site Classes C through E (obtainable from the USGS Seismic Design Geodatabase), the impacts of replacing the ASCE 7-22 deterministic capping of $MCE_G PGA_M$ with the increased 3%-in-50yrs exceedance probability are similar to those bulleted above. That is, most of the PGA_M values change by less than 20%, and the larger (>20%) changes are mostly decreases. When combined with the update to the 2023 USGS NSHM, there are even fewer locations with >20% changes, and those are more evenly split between decreases and increases. For Site Classes A through BC, the combination of the 3%-in-50yrs exceedance probability and the 2023 USGS NSHM still mostly results in PGA_M changes of less than 20%, but the larger changes are mostly decreases. These PGA_M impacts across site classes are similar to the parallel S_{MS} impacts of replacing MCE_R deterministic capping with an increased 1.5%-in-50yrs target risk of collapse. Thus,

see the S_{MS} discussions in Section C21.2.1 for justifications of the PGA_M impacts of replacing MCE_G deterministic capping with the increased 3%-in-50yrs exceedance probability.

Section 21.5 also states that the site-specific MCE_G peak ground acceleration cannot be less than 70% or 80% of the value of PGA_M obtained from the USGS Seismic Design Geodatabase, depending on the method of site-specific seismic hazard analysis. The rationale for these limits is the same as given in Section C21.2.3.

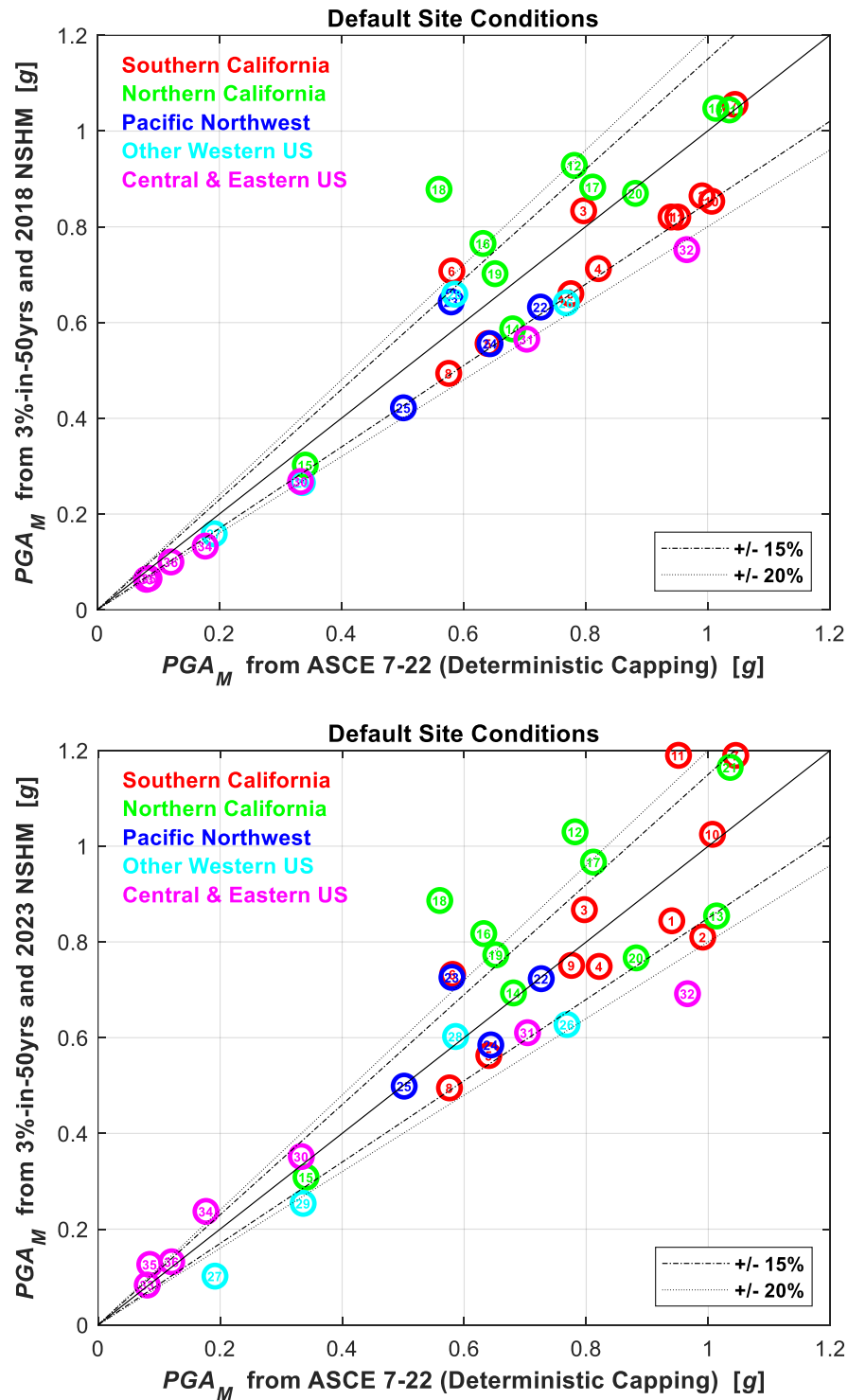


Figure C21.5-1. Comparison of MCE_G PGA_M values, for default site conditions, from the increased 3%-in-50yrs exceedance probability (without deterministic capping, using the 2018 and 2023 USGS NSHMs in the upper and lower panels, respectively) vs. those from ASCE 7-22 (with deterministic

capping, using the 2018 USGS NSHM). The numbers in the plotted circles correspond to the city locations tabulated in Table C21.5-1.

Table C21.5-1. MCE_G PGA_M values of Figure C21.5-1 and, for reference only, corresponding probabilities of exceedance (PoE) of the ASCE 7-22 MCE_G PGA_M values alongside values of the ASCE 7-22 probabilistic (uniform-hazard, 2%-in-50yrs) PGA_M .

#	Latitude	Longitude	City	ASCE 7-22 Probabilistic PGA_M (g)	ASCE 7-22 MCE_G PGA_M (g)	ASCE 7-22 MCE_G PoE (in 50yrs)	3%-in-50yrs 2018 NSHM PGA_M (g)	3%-in-50yrs 2023 NSHM PGA_M (g)
1	34.05	-118.25	Los Angeles	0.94	0.94	2.0%	0.82	0.84
2	34.05	-118.40	Century City	1.00	0.99	2.0%	0.86	0.81
3	34.20	-118.55	Northridge	0.94	0.80	3.5%	0.83	0.87
4	33.80	-118.20	Long Beach	0.82	0.82	2.0%	0.71	0.75
5	33.65	-117.80	Irvine	0.64	0.64	2.0%	0.56	0.56
6	33.95	-117.40	Riverside	0.79	0.58	5.6%	0.71	0.73
7	34.10	-117.30	San Bernardino	1.18	1.04	3.1%	1.06	1.19
8	35.30	-120.65	San Luis Obispo	0.58	0.58	2.0%	0.49	0.50
9	32.70	-117.15	San Diego	0.78	0.78	2.0%	0.66	0.75
10	34.45	-119.70	Santa Barbara	1.01	1.01	2.0%	0.85	1.02
11	34.30	-119.30	Ventura	0.95	0.95	2.0%	0.82	1.19
12	37.80	-122.25	Oakland	1.05	0.78	5.1%	0.93	1.03
13	37.95	-122.00	Concord	1.19	1.01	3.3%	1.05	0.85
14	36.60	-121.90	Monterey	0.68	0.68	2.0%	0.59	0.69
15	38.60	-121.50	Sacramento	0.34	0.34	2.0%	0.30	0.31
16	37.75	-122.40	San Francisco	0.86	0.63	5.5%	0.77	0.82
17	37.55	-122.30	San Mateo	1.02	0.81	3.8%	0.88	0.97
18	37.35	-121.90	San Jose	0.98	0.56	13.0%	0.88	0.89
19	36.95	-122.05	Santa Cruz	0.78	0.65	3.8%	0.70	0.77
20	38.10	-122.25	Vallejo	0.99	0.88	2.9%	0.87	0.77
21	38.45	-122.70	Santa Rosa	1.19	1.04	3.1%	1.04	1.16
22	47.60	-122.30	Seattle	0.73	0.73	2.0%	0.63	0.72
23	47.25	-122.45	Tacoma	0.73	0.58	4.0%	0.64	0.73
24	48.00	-122.20	Everett	0.64	0.64	2.0%	0.56	0.58
25	45.50	-122.65	Portland	0.50	0.50	2.0%	0.42	0.50
26	40.75	-111.90	Salt Lake City	0.77	0.77	2.0%	0.64	0.63
27	43.60	-116.20	Boise	0.19	0.19	2.0%	0.16	0.10
28	39.55	-119.80	Reno	0.74	0.59	4.2%	0.66	0.60
29	36.20	-115.15	Las Vegas	0.34	0.34	2.0%	0.27	0.25
30	38.60	-90.20	St. Louis	0.33	0.33	2.0%	0.27	0.35
31	35.15	-90.05	Memphis	0.70	0.70	2.0%	0.57	0.61
32	32.80	-79.95	Charleston	0.97	0.97	2.0%	0.75	0.69
33	41.85	-87.65	Chicago	0.081	0.081	2.0%	0.064	0.083
34	40.75	-74.00	New York	0.18	0.18	2.0%	0.13	0.24
35	38.90	-77.05	Washington DC	0.085	0.085	2.0%	0.066	0.13
36	33.75	-84.40	Atlanta	0.12	0.12	2.0%	0.10	0.13

Liquefaction-Targeted MCE_L Peak Ground Acceleration

The liquefaction-targeted PGA_L is defined through the integration of ground motion hazard curves with probabilistic liquefaction triggering models that describe the probability of liquefaction or probability of non-exceedance of FS_L as a function of soil properties and earthquake loading conditions, using a method known as probabilistic liquefaction hazard analysis (PLHA). Extensive background information on PLHA methods can be found in Kramer & Mayfield (2007) and Makdisi & Kramer (2024a). PLHA calculations are incorporated into the development of PGA parameters for liquefaction analysis in order to address multiple limitations inherent to the use of the MCE_G PGA_M . The liquefaction-targeted PGA_L addresses these limitations by (1) accounting for differences in hazard curve shapes across different regions, rather than using a single return period of ground shaking; (2) accounting for the effects of duration on liquefaction hazard from all plausible magnitudes, rather than selecting a single representative value; and (3) accounting for uncertainty in the estimation of liquefaction triggering through the use of probabilistic triggering models.

In the context of these provisions, PLHA is used to calculate hazard curves for liquefaction triggering that are based on the annualized rate of non-exceedance of liquefaction factor of safety, Λ_{FS_L} , which is computed as:

$$\Lambda_{FS_L}(fs_L) = \sum_{j=1}^{N_m} \sum_{i=1}^{N_{pga}} P[FS_L < fs_L | pga_i, M_{w,j}] \Delta\lambda_{pga_i, m_{w,j}} \quad \text{Eq C21.5.1}$$

where $P[FS_L < fs_L | pga_i, M_{w,j}]$ is the conditional probability of non-exceedance of a given FS_L value (i.e., fs_L) for a given combination of PGA and M_w , which can be estimated using probabilistic liquefaction triggering models (e.g., Cetin et al., 2004; Moss et al., 2006; Boulanger & Idriss, 2012; Boulanger & Idriss, 2016); and $\Delta\lambda_{pga_i, m_{w,j}}$ is the joint incremental annualized rate of exceedance of PGA for a given M_w bin, obtained from the PSHA-based hazard curve and disaggregation. This calculation is typically repeated for a range of fs_L values to generate hazard curves for FS_L , and performed for all susceptible soil layers in a given profile. A set of FS_L hazard curves is shown for a hypothetical soil profile, using PGA hazard data from a Seattle, Washington, in Figure C21.5.1. The arrows in Figure C21.5.1 illustrate how the FS_L hazard curves can be used to obtain estimates of the return period of liquefaction triggering, $T_{R,L}$. Alternatively, the FS_L hazard curves can be used to compute the FS_L value with a particular return period of non-exceedance.

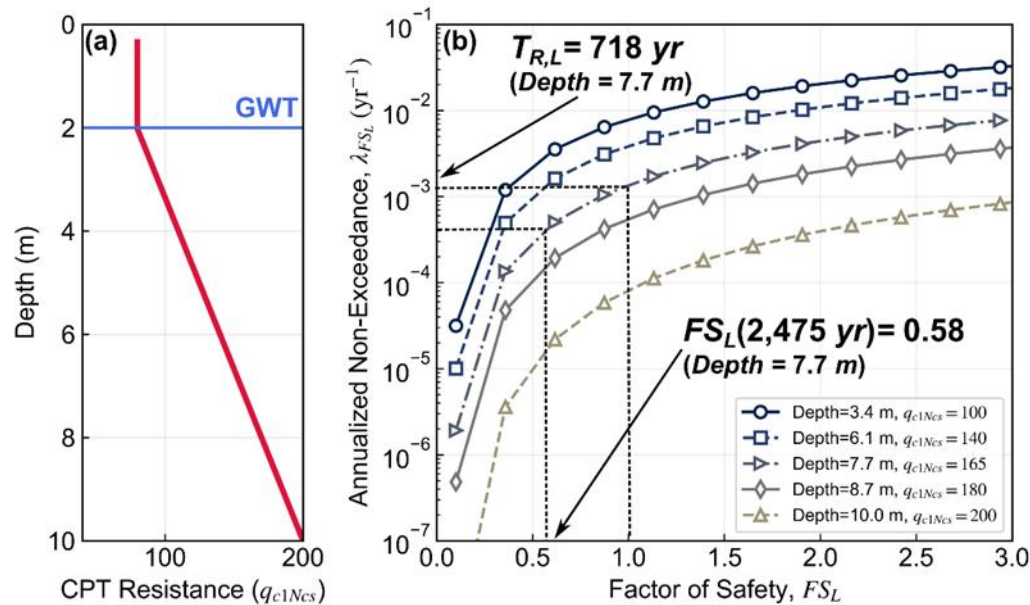
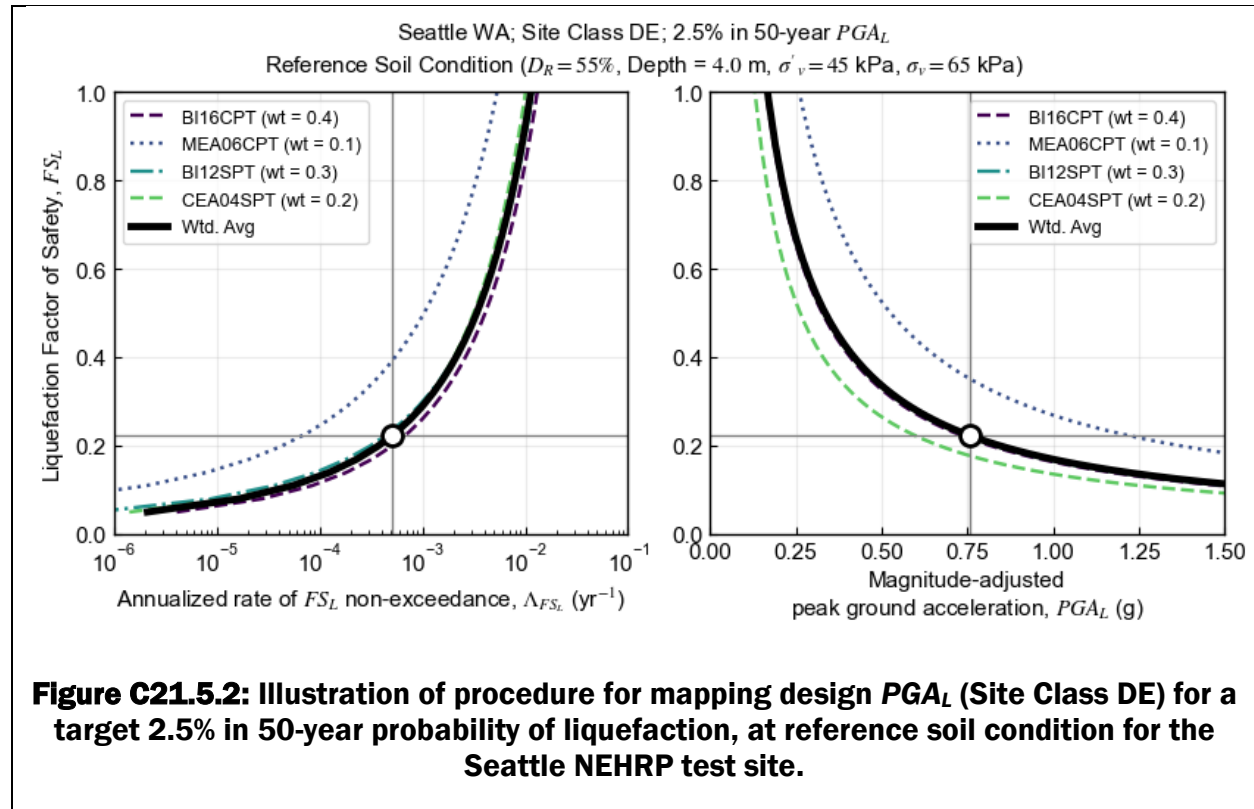


Figure C21.5.1: (a) example CPT profile and (b) liquefaction triggering hazard curves describing the annualized rate of non-exceedance of factor of safety, for a five depth increments, using ground motion data at a site in Seattle, WA. (Makdisi & Kramer,

The goal of the $MCE_L PGA_L$ is to approximate probabilistic liquefaction hazard estimates in conventional deterministic liquefaction hazard calculations for a targeted liquefaction hazard level of interest. Such liquefaction hazard-consistent estimates could otherwise be calculated directly using the PLHA procedures outline above. In these provisions, PGA_L is considered to be a more appropriate loading parameter for liquefaction hazard analysis than the MCE_G peak ground acceleration, as it is informed by PLHA-based calculations and a 2.5% probability of liquefaction design criteria. The PLHA-based calculations used to develop the PGA_L parameter ensure that the PGA hazard levels and magnitude contributions most relevant to liquefaction hazard are represented in the loading parameter. The $MCE_L PGA_L$ can be used to estimate FS_L values with a 2.5% probability of non-exceedance over a time period of 50 years. These liquefaction-targeted design parameters effectively allow conventional liquefaction triggering procedures to be used to assess whether liquefaction hazard exceeds the acceptable, 2.5% in 50-year probability; if the FS_L values computed using PGA_L are less than 1.0, this is an indication that the probability of liquefaction is greater than the acceptable design level of 2.5% in 50 years, or a liquefaction return period of less than 1,975 years. The PGA_L values can also be used as inputs to other liquefaction hazard models, such as the estimation of surface manifestation and liquefaction severity, free-field settlements, and others, in the same way that the $MCE_G PGA_M$ value has been used in previous editions of these provisions.

The procedure for determining the $MCE_L PGA_L$ is demonstrated in **Figure C21.5.2**, using the framework described in greater detail in Makdisi & Kramer (2024b), for a reference soil condition. The left-hand side of **Figure C21.5.2** shows the PLHA-based FS_L hazard curve, which is computed as the weighted average of four probabilistic liquefaction triggering models in order to capture epistemic uncertainties in the liquefaction fragility. The hazard curve is interpolated to obtain the

reference FS_L value that corresponds to the target liquefaction hazard level. The right-hand side of **Figure C21.5.2** shows the corresponding deterministic curves that relate PGA_L with FS_L for each of the triggering models. The curves on the right are obtained through application of conventional deterministic liquefaction triggering models, where FS_L is calculated for a range of PGA_L values at the reference soil condition, assuming magnitude scaling or duration weighting factors equal 1.0 (i.e., that PGA_L is already magnitude-adjusted). The overall PGA_L - FS_L curve is similarly taken as a weighted average of the four component models, and is then interpolated at the 2.5% in 50-year FS_L values obtained on the right side of **Figure C21.5.2** to determine PGA_L .



The PGA_L value is provided for a given set of coordinates and site class in the USGS Seismic Design Geodatabases, along with the liquefaction coefficient C_L for site-specific procedures. The PGA_L can be used in the same way that PGA_M has been used for a wide variety of liquefaction hazard analyses.

The utility of the single, reference soil condition for mapping PGA_L has been justified in Makdisi & Kramer (2024b), which show that the design PGA_L determined in this framework is relatively insensitive to soil conditions – that is, that changes in cyclic resistance, depth, or stress conditions tend to cause the PLHA-based FS_L hazard curves (**Figure C21.5.2a**) and the deterministic PGA_L - FS_L curves (**Figure C21.5.2b**) to shift vertically by similar amounts. The reference soil condition is generically defined here using the following properties that are used as inputs to typical liquefaction triggering models: a clean sand with relative density D_R of 55%, a depth of 4.0 meters (13.1 ft), a total stress of 65 kPa (0.64 atm), and an effective stress of 45 kPa (0.44 atm). For each triggering model used to define the liquefaction fragility, the equivalent cyclic resistance parameters for the

reference condition are summarized, along with the model weighting factor used to determine the overall PGA_L parameter, as follows:

- Boulanger & Idriss (2016) CPT: model weight = 0.40, equivalent, clean sand-corrected cone penetrometer test (CPT) resistance $q_{c1Ncs} = 100$
- Moss et al. (2006) CPT: model weight = 0.10, normalized CPT cone tip resistance $q_{c1} = 11$ MPa, CPT friction ratio $R_f = 0.30\%$, tip normalization exponent $c = 0.60$, shear wave velocity in the upper 12 m of site $V_{s12} = 150$ m/s
- Boulanger & Idriss (2012) SPT: model weight = 0.30, clean sand-corrected standard penetration test (SPT) resistance $(N_1)_{60,cs} = 13$, fines content $FC = 0$
- Cetin et al. (2004) SPT: model weight = 0.20, standard penetration test (SPT) resistance $(N_1)_{60} = 13$, $FC = 0$, $V_{s12} = 150$ m/s

Section 21.5.4 describes two methods for using the results of a site-specific PSHA to compute the liquefaction-targeted PGA_L .

1. The simpler, but more approximate method of computing the liquefaction-targeted PGA_L is to first interpolate the site-specific PGA hazard curve at the mean annual frequency corresponding to a 3% probability of exceedance in 50 years. This “uniform-hazard” PGA value is then multiplied by the “liquefaction coefficient”, C_L , as defined in Section 21.5.4, which can be obtained as part of the PGA_L data provided in the USGS Seismic Design Geodatabases. This first method can be considered analogous to the “Risk Coefficients” that were provided in the ASCE 7-10 and 7-16 guidelines for converting uniform hazard spectral accelerations (in the case of the MCE_R risk coefficients, corresponding to a 2% probability of exceedance in 50 years for consistency with the previous MCE design basis). Using this method, the limitation that the site-specific PGA_L not be less than 80% of the PGA_L provided in the USGS Seismic Design Geodatabase is applied for consistency with other site-specific MCE_R procedures outlined in Chapter 21.
2. The second method essentially outlines the calculation of the liquefaction-targeted PGA_L at soil conditions specific to the site of interest, using the procedures detailed in this commentary section. This method allows for deviations from the reference condition and more accurate estimates of the 2.5% in 50-year FS_L , and allows for more flexible specification of the models and weights used to define the liquefaction fragility. For this reason, site-specific procedures are permitted to be as low as 70% of the Geodatabase PGA_L values.

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COMMENTARY TO CHAPTER 22, SEISMIC GROUND MOTION AND LONG-PERIOD TRANSITION MAPS

Revise Chapter C22 as follows:

This standard contains risk-targeted maximum considered earthquake (MCE_R) spectral response acceleration maps (Figures 22-1 through 22-8) and maximum considered earthquake geometric mean (MCE_G) peak ground acceleration maps (Figures 22-9 through 22-13), both introduced in the 2009 NEHRP provisions and ASCE 7-10. Long-period transition period (T_L) maps are also provided (Figures 22-14 through 22-17), as introduced in ASCE 7-05. The MCE_R and MCE_G maps are updated based on the revised site-specific ground motion procedures and definitions of Chapter 21, using the 2023 US Geological Survey (USGS) National Seismic Hazard Model (NSHM) for the conterminous United States (CONUS), Alaska and Hawaii (Petersen et al. 2024; Powers et al. 2024, Petersen et al. 2022), and the 2025 USGS NSHM for Puerto Rico and the US Virgin Islands (Shumway et al. 2025). The updates to the MCE_R and MCE_G maps are summarized next, including (i) a summary of hazard model development details, (ii) examples of changes at 36 CONUS sites, four Alaska and Hawaii sites, and two Puerto Rico and the US Virgin Islands sites, and (iii) a description of USGS resources for retrieving Chapter 22 values.

Modifications to MCE_R and MCE_G ground motions. Whereas the MCE_R and MCE_G maps of ASCE 7-22 for CONUS were derived from the 2018 USGS NSHM (Petersen et al. 2019), the MCE_R and MCE_G ground motions of this provision are derived from the 2023 USGS NSHM (Petersen et al. 2024). For locations outside of the conterminous United States (OCONUS), the updates to seismic hazard models vary and are described later in this section. In addition to the seismic hazard model, changes are also partly due to the revised site-specific ground motion procedures and definitions of Chapter 21 and replacing the deterministically capped risk-targeted and uniform-hazard ground motions of ASCE 7-22 with different risk-targeted and uniform-hazard ground motions. The major modifications in the 2023 NSHM for CONUS are summarized here. Details are presented in the overview documentation of the 2023 update (Petersen et al. 2024) with additional references to various components of the model. Examples of the resulting changes to the values of MCE_R and MCE_G ground motions are given in the ensuing subsections.

The 2023 NSHM update for CONUS used the best available science related to earthquake seismicity, fault rupture characterization, ground-motion distribution, and hazard estimation techniques to produce a standard of practice for public policy applications. Examples of such advancements in science are outlined in 1) Field et al. (2024), which summarizes the earthquake rupture forecast (ERF) model, 2) Moschetti et al. (2024), which summarizes the improvements in ground motion model (GMM) characterizations including basin amplifications, and 3) Rezaeian et al. (2024), which summarizes advancements in subduction GMMs including 3D simulation-based amplifications for

the Seattle basin. This new probabilistic seismic hazard model provides important new data, models, and methods that improve the hazard characterization, but generally results in larger estimates of ground motions compared to previous models because so many new data (e.g., added faults), models and methods (i.e., increased epistemic uncertainty) are incorporated for both ERF and GMM inputs. In improving the input component models, the USGS ensured that the new modeling changes are consistent with the best available science; that input component models are defensible; and results are robust, tested as well as possible, and reproducible. Several review panels provided a consensus affirmation that these new models represent an improvement in forecasting earthquake sources and ground motions. The 2023 NSHM attempted to better categorize aleatory variability and epistemic uncertainty and took into consideration feedback from external scientists and engineers including the BSSC PUC. The major changes in the 2023 NSHM for CONUS are as follows:

1. Using a new updated earthquake catalog, as well as new declustering methods and spatial smoothing methods to produce a new background seismicity model. For this new model, full-catalog rates were considered rather than declustered earthquake rates applied previously in NSHMs. These changes could result in increases in the estimated hazard by more than 10% in some locations.
2. Using an updated suite of deformation models that included two additional geodetic deformation models (expanded from three previous models) that are all somewhat consistent with the available data and are weighted differently in the 2023 ERF, generally increasing the epistemic uncertainty in the model.
3. Using new earthquake rupture rate models including new broad inversion-based fault-system solutions of ERF for the entire western US (WUS) rather than just California in the previous models.
4. Improving the subduction GMMs, which were known to have limitations in the previous models, and now include the recently published NGA-Subduction GMMs. These models are amplified or constrained using simulation-derived basin amplifications in the Seattle basin.
5. Modification of GMMs in central and eastern US (CEUS) to reduce data residuals and better account for non-linearity and a bias recognized in some GMMs.
6. Improving estimates of long-period ground motions in sedimentary basins by accounting for new sedimentary basin-depth models in San Francisco, Los Angeles, and Seattle and for Coastal Plain amplification models that generally increase the long-period ground motions and reduce short-period motions. Some de-amplifications are allowed in the shallower San Francisco basin based on user feedback. Also, 3D ground-motion simulations are used to improve the basin-amplification models in Los Angeles and Seattle basins.

Finally, other changes were considered in the 2023 NSHM for CONUS, such as including a new directivity model for active faults in California and a new time-dependent model for Cascadia but were ultimately not implemented. As summarized in Petersen et al. (2024), even though these changes were considered reasonable from a scientific standpoint, they were deemed to be too immature for application in current engineering practices.

For Alaska, Hawaii, and territories outside of the conterminous United States (OCONUS), the MCE_R and MCE_G ground motion values and maps in ASCE 7-22 were derived based on a procedure documented in FEMA P-2078 (2020b). USGS hazard models for these regions were not available at the time for developing multi-period response spectra adopted in ASCE 7-22 and therefore the FEMA P-2078 (2020b) procedure was developed to approximate site class effects and spectral values at all required periods as an interim solution pending forthcoming USGS updates for these states and territories. USGS updated the hazard models of Hawaii in 2021 (Petersen et al. 2022), Alaska in 2023 (Powers et al. 2024), and Puerto Rico and the US Virgin Islands in 2025 (Shumway et al. 2025). These updated hazard models are used to calculate the values of MCE_R and MCE_G ground motions for Hawaii, Alaska, and Puerto Rico and the US Virgin Islands in this provision instead of their approximated values in ASCE 7-22 that were based on the 2001 Hawaii, 2007 Alaska, and 2003 Puerto Rico and the Virgin Islands hazard models (see details and comparisons of new to previous hazard models in Petersen et al. 2022, Powers et al. 2024, and Shumway et al. 2025) combined with the procedures of FEMA P-2078 (2020b). Four example sites that were previously used in FEMA P-2078 (2020b) are added to the tables of this chapter for Alaska and Hawaii: Anchorage and Fairbanks in Alaska, and Hilo and Honolulu in Hawaii. Two example sites are added for Puerto Rico and the US Virgin Islands: San Juan, PR, and Christiansted, VI. The MCE_R and MCE_G ground motions for other United States territories do not change in this provision from those in ASCE 7-22 and will continue to be approximated based on FEMA P-2078 (2020b), pending forthcoming USGS updates for those regions.

The new 2023 Alaska hazard model (Powers et al. 2024) includes a wide range of updates that reflect considerable advances in earthquake hazard science and data in this region since the previous version of the model in 2007. The model was reviewed by a six-member review panel team, as well as a supplemental eight-member GMM panel to advise the USGS on issues related to the implementation of NGA-Subduction GMMs in Alaska. Feedback from both panels was incorporated into the final model update, with the panels concluding that the updated Alaska hazard model represents the best available science and is well-documented. The major improvements are as follows:

1. Use of a significantly updated earthquake catalog, declustering and spatial smoothing methods (consistent with the 2023 NSHM CONUS update) to produce a new background seismicity model for off-fault crustal and subduction (intraslab and smaller-magnitude interface) earthquakes.
2. The expansion of modeled crustal faults from 9 in 2007 to 105 in 2023, with improved geologic and geodetic slip rate estimates, as well as the addition of a number of deformation zone sources including the Cook Inlet, Kodiak Shelf, Northern Alaska Range, Pamplona, and Coast Shear zones.
3. Advancements to the hazard characterization along the Alaska-Aleutian subduction zone, including better constraints on the geometry of the subduction interface and inclusion of newer magnitude-scaling relationships, ultimately resulting in an improved earthquake

segmentation and recurrence model that incorporates the latest paleoseismic and paleotsunami observations and geodetic data.

4. Implementation of the NGA-West2 GMMs for active crustal earthquakes, which include improvements such as site and basin depth scaling, hanging wall effects, improved aleatory variability and epistemic uncertainty representation, and support ground motion estimates for a broader range of periods and site conditions consistent with the multi-period response spectra requirements of the NEHRP Provisions.
5. Implementation of the NGA-Subduction suite of ground motion models, which are more appropriate for estimating intraslab and interface ground motions than the 2007 hazard model and include better representations of aleatory variability and epistemic uncertainties

The new 2021 Hawaii hazard model (Petersen et al. 2022) also represents a considerable improvement in seismic hazard assessment for the State of Hawaii relative to the previous version of the model published in 2001 (Klein et al. 2001). Similar to the 2023 NSHM CONUS and Alaska updates, the 2021 Hawaii update was subjected to thorough external review and feedback. The major improvements are as follows:

1. A significantly expanded and updated earthquake catalog to model background seismicity for off-fault earthquakes, as well as new declustering and spatial smoothing models. These earthquake catalogs were used to assess three rate models for 50-year periods and found to differ significantly with the faster rates occurring prior to 1960. Additionally, new earthquake source information on the 1871 M 6.8-7.5 Lanai earthquake were considered to inform the maximum magnitudes.
2. Improvements to hazard assessments near the crater on Kilauea, using a new model for earthquakes produced by caldera collapses.
3. Improvements to Quaternary fault mapping and estimation of fault slip rates through the collection of geodetic data over the past two decades.
4. Implementation of the NGA-West2 GMMs for active crustal earthquakes, and two new Hawaii-specific GMMs that include ground motion recordings from the region (including recordings from two large earthquake sequences in 2006 and 2018) over the past two decades. The suite of GMMs used for the 2021 Hawaii model allows for improved estimation of ground motions from shallow crustal events on the Hawaiian islands, as well as deep earthquakes beneath the islands.

The new 2025 Puerto Rico and US Virgin (PRVI) Islands hazard model (Shumway et al. 2025) represents considerable improvements in seismic hazard assessment compared to the previous version of the model published in 2003 (Mueller et al. 2003), based on over 20 years of updated science, data, and models that includes knowledge from the 2020 M 6.4 earthquake sequence. The

2025 NSHM has been reviewed by panels of local and topical experts on earthquake rupture forecasting and ground motion characterization. The major improvements are as follows:

1. Development and implementation of a new historical earthquake catalog to model background seismicity from observed crustal, interface, and intraslab earthquakes, based on improved data from the Puerto Rico Seismic Network.
2. Refinements to earthquake catalog separation and de-clustering methods, smoothing algorithms, and background seismicity rate models.
3. Use of new magnitude-area scaling relationships for both crustal and subduction sources, to convert modeled ruptures to magnitudes.
4. Expansion of the geologic crustal fault source characterization from three faults and two fault-zone polygons in the 2003 model to 35 fault sections and 6 fault-zone polygons in the 2025 model.
5. Implementation of new methods for deriving geologic-based fault deformation rates (consistent with the 2023 NSHM methodology use for WUS faults), as well as improved uncertainty representation of those estimated rates.
6. Improved modeling of subduction interface sources, including updates from the 2003 model to the geometry and deformation estimates of the Puerto Rico Trench (which lies north of the PRVI region, dipping south towards the region), and a newly implemented model of the Muertos Trough (which lies south of the study area, dipping north toward the study region).
7. Implementation of new earthquake rupture rate models using the inversion-based fault-system solution, and a more complete representation of epistemic uncertainties (consistent with the methodology used in the 2023 NSHM to derive fault rupture rates in the WUS)
8. Implementation of new crustal and subduction GMMs that include greatly expanded ground-motion datasets, use new functional forms, and incorporate important effects such as hanging-wall and nonlinearity. The new GMMs are a substantial improvement compared to the pre-NGA GMMs that were used in the 2003 model, and are more aligned with current GMMs used in CONUS and Alaska. The new model also gives partial weight to GMMs that are adjusted to data from the PRVI region to better capture region-specific attenuation behavior.
9. Improved site-response amplifications through recalibration of global GMMs to ground motions recorded in the PRVI region. Regionalized site-response models receive partial weight and make use of ground-motion data from the PRVI region.

10. Improved GMM epistemic and aleatory variability representation, using models developed specifically for the PRVI region. Such improvements to the framework for ground-motion characterization allow for more transparent and accurate weighting of logic-tree branches.

Examples of changes in MCE_R and MCE_G values. The combined impacts of the USGS NSHM modifications and the revised site-specific ground motion procedures and definitions of Chapter 21 on MCE_R and MCE_G values are demonstrated in Figures C22-1 through C22-3 for the 36 CONUS example locations listed in Table C22-1, and in Tables C22-2 through C22-4 for all CONUS and OCONUS locations listed in Table C22-1. The CONUS locations are the 34 locations considered in ASCE 7-22, which were first introduced in the 2009 NEHRP Provisions (NEHRP 2009), with the addition of two new locations in Washington DC and Atlanta, GA. The six OCONUS locations added to the tables are new additions in this provision. In the MCE_R and MCE_G figures and tables, ground motions of this standard are compared with those from ASCE 7-22, ASCE 7-16, and ASCE 7-10. The S_{MS} , S_{M1} , and PGA_M ground motions are for default site conditions, so that the comparisons capture the most common site class effects. Corresponding seismic design categories (SDCs) are compared in Table C22-5. For the locations where the MCE_R and/or MCE_G ground motions for default site conditions have changed by greater than 15% and where the SDC values have changed from ASCE 7-22 to this provision, an explanation of the predominant causes follows. Changes less than 15%, while potentially impactful, tend to be combinations of several modifications, and hence do not lend themselves to clear explanations.

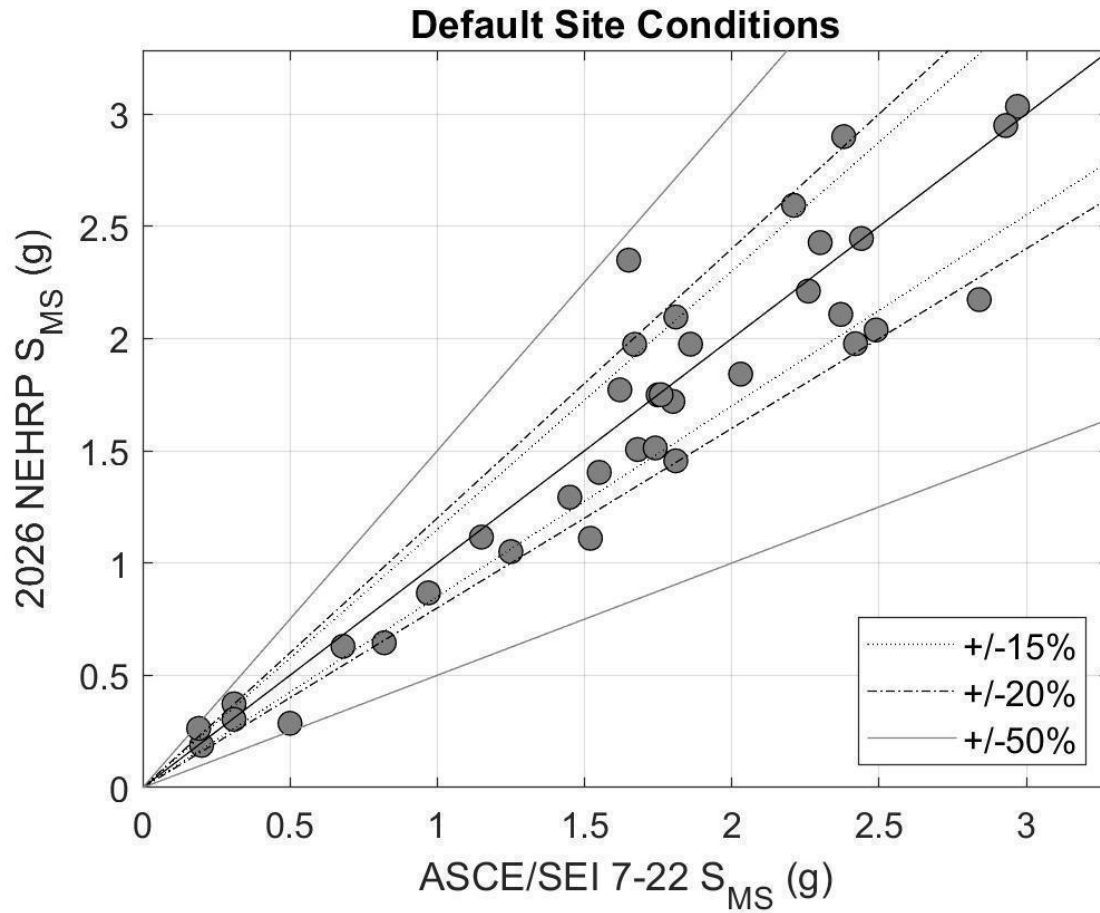


Figure C22-1. Comparison of short-period MCE_R spectral response acceleration (S_{MS}) values from this standard versus ASCE 7-22 for default site conditions and 36 CONUS example locations.

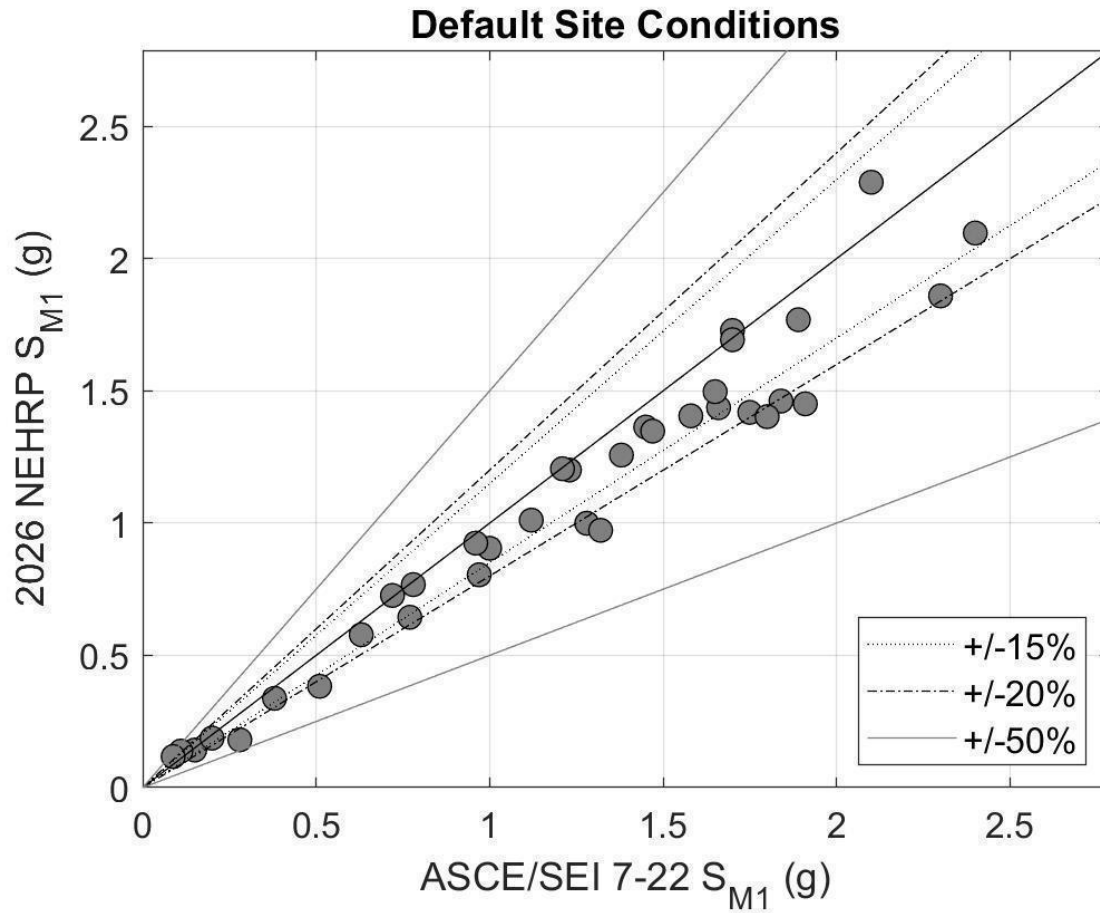


Figure C22-2. Comparison of 1.0 s MCE_R spectral response acceleration (S_{M1}) values from this standard versus ASCE 7-22 for default site conditions and 36 CONUS example locations.

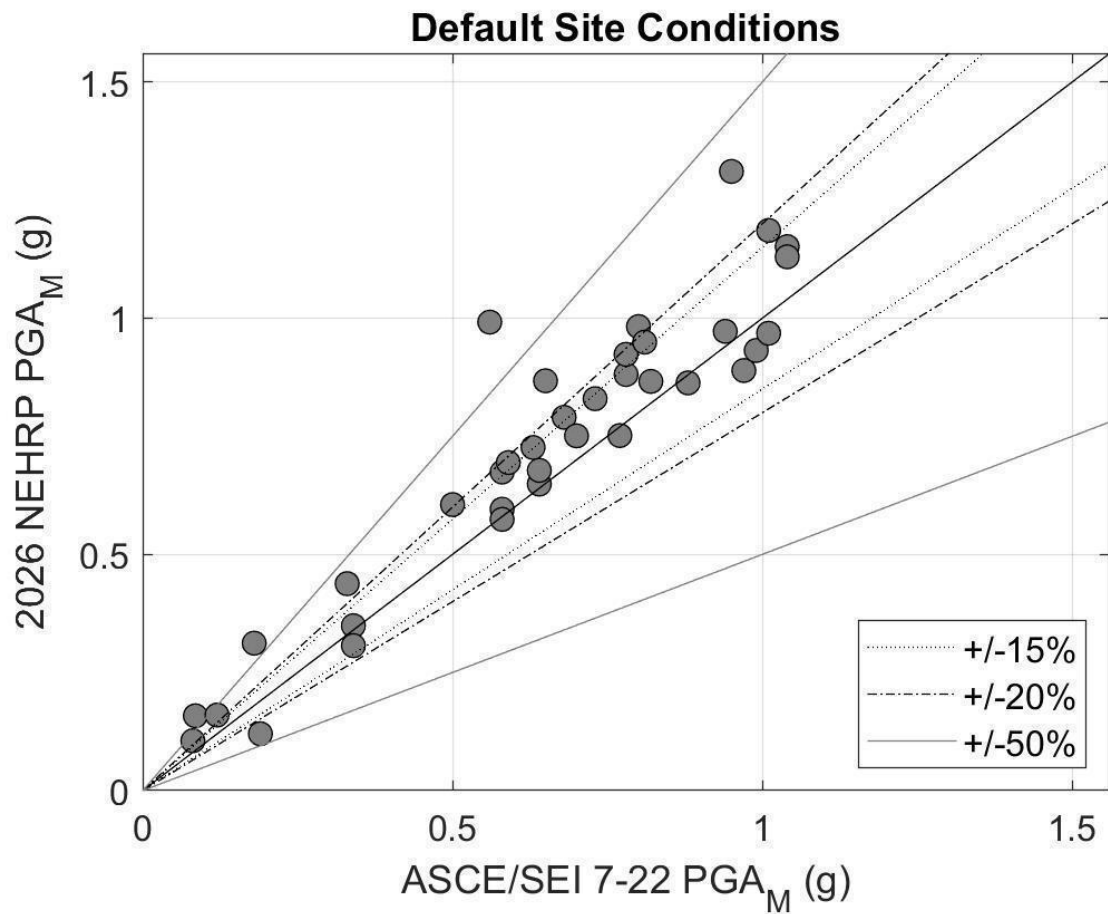


Figure C22-3. Comparison of MCE_G peak ground acceleration (PGA_M) values from this standard versus ASCE 7-22 for default site conditions and 36 CONUS example locations.

Table C22-1. Latitudes and longitudes at which MCE_R and MCE_G ground motions and corresponding seismic design categories from this provision are compared.

	City and Location of Site		
Region	Name	Latitude	Longitude
Southern California	Los Angeles	34.05	-118.25
	Century City	34.05	-118.40
	Northridge	34.20	-118.55
	Long Beach	33.80	-118.20
	Irvine	33.65	-117.80
	Riverside	33.95	-117.40

	San Bernardino	34.10	-117.30
	San Luis Obispo	35.30	-120.65
	San Diego	32.70	-117.15
	Santa Barbara	34.45	-119.70
	Ventura	34.30	-119.30
Northern California	Oakland	37.80	-122.25
	Concord	37.95	-122.00
	Monterey	36.60	-121.90
	Sacramento	38.60	-121.50
	San Francisco	37.75	-122.40
	San Mateo	37.55	-122.30
	San Jose	37.35	-121.90
	Santa Cruz	36.95	-122.05
	Vallejo	38.10	-122.25
	Santa Rosa	38.45	-122.70
Pacific Northwest	Seattle	47.60	-122.30
	Tacoma	47.25	-122.45
	Everett	48.00	-122.20
	Portland	45.50	-122.65
Other WUS	Salt Lake City	40.75	-111.90
	Boise	43.60	-116.20
	Reno	39.55	-119.80
	Las Vegas	36.20	-115.15

CEUS	St. Louis	38.60	-90.20
	Memphis	35.15	-90.05
	Charleston	32.80	-79.95
	Chicago	41.85	-87.65
	New York	40.75	-74.00
	Washington DC	38.90	-77.05
	Atlanta	33.75	-84.40
Alaska	Anchorage	61.20	-149.90
	Fairbanks	64.85	-147.70
Hawaii	Hilo	19.70	-155.10
	Honolulu	21.30	-157.85
Puerto Rico	San Juan	18.45	-66.10
Virgin Islands	Christiansted	17.75	-64.70

Notes: ASCE 7-22, ASCE 7-16, and ASCE 7-10 are compared in Tables C22-2 through C22-5.

As previously mentioned, in addition to the 36 CONUS locations, six new OCONUS locations in Alaska, Hawaii, Puerto Rico and the US Virgin Islands are also added in this provision. It is important to note that these locations are each just one of many in the named cities, and their ground motions may be significantly different than those at other locations in the cities.

As seen in Table C22-2 and Figure C22-1, which are for the short-period MCE_R spectral response accelerations, the S_{MS} values change from ASCE 7-22 to this provision by less than 20% at all but nine of the 36 CONUS locations, with four locations at Ventura, San Jose, New York City, and Washington DC increasing (by 22%, 42%, 20%, and 39%, respectively) and five locations at Concord, Salt Lake City, Boise, Las Vegas, and Charleston decreasing (by 23%, 20%, 43%, 21%, and 27%, respectively). At three locations in Riverside, Oakland, and San Francisco increases more than 15% but less than 20% are observed (18%, 17%, and 16%, respectively); and at three locations in Century City, Vallejo, and Memphis decreases more than 15% but less than 20% are observed (18%, 18%, and 16%, respectively). These changes are either due to a significant update in the science of the 2023 USGS NSHM such as the new fault models introduced in the ERF or the update of subduction GMMs or basin amplifications, or due to the revised site-specific ground motion procedures and definitions of Section 21.2.1, which removed the deterministic caps and replaced the 1% with 1.5% risk target. Major changes are explained for each location as follows.

- The estimate of seismic hazard increases at the Ventura and Santa Barbara locations by 43% and 19%, respectively, mainly due to improved fault modeling and slip rate characterization on the Pitas Point fault (e.g., Johnson et al. 2017), as well as a general increase in the rate of moderate ruptures due to a reduction in larger, multi-fault rupture rates in the ERF model (Field et al. 2024). On the other hand, the revised site-specific ground motion procedures and definitions of Section 21.2.1 lower the ground motion at these locations (relative to existing ASCE 7-22 procedures) resulting in a final increase of 22% at Ventura (from 2.38g to 2.90g), and almost no change at Santa Barbara (from 2.44g to 2.45g).
- The large increase of 42% at the San Jose location is mainly due to the ERF changes of the 2023 NSHM and modifications to the close-distance Silver Creek Fault and the farther Hayward Fault. While MCE_R spectral accelerations at shorter periods in San Jose were largely governed by the deterministic lower limit in ASCE 7-22, the increase in the modeled slip rate for Silver Creek Fault causes design spectral accelerations to be generally governed by either probabilistic or 84th percentile deterministic ground motions if methods of ASCE 7-22 were to be used, resulting in an increase from 1.65g to 2.69g. However, the revised site-specific ground motion procedures and definitions of Section 21.2.1 lower the final ground motion (relative to existing ASCE 7-22 procedures) to be 2.35g.
- The large increases at the New York and Washington DC locations (20% from 0.31g to 0.37g and 39% from 0.19g to 0.26g, respectively) are mainly due to modifications to the new updated background seismicity model with additions and revisions to the seismicity catalog (Field et al. 2024) and improvements to other CEUS sources such as the addition of the Central Virginia fault zone model. Also, note that these percent changes seem more extreme due to the much smaller values of ground motion parameters in these locations, whereas the absolute value changes (0.06g and 0.07g) may not be as substantial.
- The moderate increases at the locations in Riverside, Oakland, and San Francisco are a combination of the 2023 NSHM increasing the ground motions by less than 15% and the revised site-specific ground motion procedures and definitions of Section 21.2.1 also increasing the ground motions by less than 15%, with combination of both changes resulting in the final moderate increases at these locations.
- At some locations such as Santa Cruz, Seattle and Portland, the 2023 NSHM modifications result in an increase in the ground motion values, but the revised site-specific ground motion procedures and definitions of Section 21.2.1 lower the ground motions and therefore the final changes are very small. At the Santa Cruz location, the increase in the estimate of hazard is mostly due to an increase in the rate of full-fault ruptures on the controlling Reliz Fault scenario. While a resulting increase in the mean magnitude contribution resulted in only a modest increase in the probabilistic ground motions, a much larger increase is observed in deterministic ground motions if the definitions of ASCE 7-22 were to be used. Under the revised site-specific procedures of Section 21.2.1, ground motion at this location changes from 1.86g to 1.97g, only a 6% increase. At the Seattle and Portland locations, the

estimate of hazard increases due to a combination of factors in both the ERF and GMM components of the 2023 NSHM update, each causing moderate increases in probabilistic ground motions. ERF-changes include the significant improvements to the characterization of fault sources in Portland (and less substantive modifications to faults in Seattle), as well as the incorporation of the entire WUS into the inversion framework for estimating fault slip rates, which allows for the inclusion of multi-fault ruptures among other improvements. GMM-related changes can be attributed solely to the inclusion of the NGA-Subduction GMMs that in general increased short-period ground motions. Finally, the revised site-specific ground motion procedures and definitions of Section 21.2.1 lower the final ground motion with the final values decreasing by 1% at the Seattle location (from 1.76g to 1.75g) and by 3% at the Portland location (from 1.15g to 1.12g).

- The large decrease of 43% at the Boise locations (from 0.50g to 0.29g) can be attributed to a number of factors related to the updated background gridded seismicity model, which contributes roughly 85% to the overall seismic hazard in southwest Idaho. These factors include: (1) the decrease in the observed seismicity rate due to no recorded $M > 4$ events in the southwest Idaho region since the 2018 NSHM update; and (2) the removal of minimum ‘floor’ seismicity rates estimated from the fixed smoothing algorithm in WUS, as these rates no longer represent the best available science for estimating background seismicity in the WUS.
- The large decreases at the Concord, Salt Lake City, Las Vegas, Charleston, Century City, and Vallejo locations are a combination of the 2023 NSHM lowering the estimate of the seismic hazard by 15% or less and the revised site-specific ground motion procedures and definitions of Section 21.2.1 also lowering the ground motion by 15% or less by accepting a higher risk of collapse, resulting in decreases more than 15% in comparison to ASCE 7-22. At the Concord location, ground motion decreases from 2.84g to 2.17g (it would have been 2.51g if 2023 NSHM was used with the site-specific procedures of ASCE 7-22). At the Salt Lake City location, ground motion decreases from 1.81g to 1.46g (it would have been 1.78g if 2023 NSHM was used with the site-specific procedures of ASCE 7-22). At the Las Vegas location, ground motion decreases from 0.82g to 0.65g (it would have been 0.77g if 2023 NSHM was used with the site-specific procedures of ASCE 7-22). At the Charleston location, ground motion decreases from 1.52g to 1.11g (it would have been 1.43g if 2023 NSHM was used with the site-specific procedures of ASCE 7-22). At the Century City location, ground motion decreases from 2.49g to 2.04g (it would have been 2.38g if 2023 NSHM was used with the site-specific procedures of ASCE 7-22). At the Vallejo location, ground motion decreases from 2.42g to 1.98g (it would have been 2.28g if 2023 NSHM was used with the site-specific procedures of ASCE 7-22).

As also seen in Table C22-2 for Alaska locations, the S_{MS} values increase from ASCE 7-22 to this provision by 98% and 16%, respectively in Anchorage and Fairbanks in Alaska. Major changes are explained for these locations as follows.

- The 98% increase in S_{MS} in Anchorage (from 1.65g to 3.27g) is attributed to substantial improvements to both earthquake sources and GMM components in updating the 2007 Alaska hazard model to the 2023 edition. The updates most impactful to hazard at the Anchorage site are (1) a significant increase in the rate of M6-8 intraslab earthquakes and M7-9 interface earthquakes within 200 km of Anchorage; (2) improvements to the characterization of the subduction interface geometry as well as the spatial distribution of subduction interface events, resulting in interface ruptures modeled closer to Anchorage than in the 2007 model; (3) the adoption of the NGA-Subduction GMMs and their impacts on the increased intraslab and interface hazard contributions near Anchorage, which result in higher ground motions at shorter spectral periods in the near field; and (4) the addition of the Cook Inlet source zone, which contributes to roughly 5 to 15% of the MCE_R ground motion level at short periods.
- The 19% increase in S_{MS} in Fairbanks (1.23 to 1.42 g) is largely attributed to increased hazard from improvements to the crustal and intraslab gridded seismicity models, and the associated updates to the crustal GMMs (NGA-West2) and subduction GMMs (NGA-Sub) described earlier in this section.

For Hawaii locations, the S_{MS} value increases from ASCE 7-22 to this provision by 72% in Hilo, but does not change much in Honolulu. The increase in Hilo is partly due to the new definition of the site-specific ground motion based on Chapter 21 because Hilo was previously deterministically capped and now is fully probabilistic, and partly due to the increased hazard estimates (primarily as a result of the new GMMs implemented) at this location compared to the models used about 20 years ago in the 2001 Hawaii hazard model (more details are provided in Petersen et al., 2022).

For Puerto Rico and the US Virgin Islands, the S_{MS} value decreases by 8% in San Juan, PR, but increases by 12% in Christiansted, VI, from ASCE 7-22 to this provision. These changes are due to a combination of 1) the new definition of site-specific ground motions in Chapter 21, 2) changes in seismic hazard, and 3) the ASCE 7-22 approximation of site class amplifications. In this region, ground motions of ASCE 7-22 were fully probabilistic for most periods and site classes and are now decreased due to a change in the target risk of collapse in 50 years from 1% to 1.5%. On the other hand, seismic hazard estimates have increased significantly at short periods based on the updated 2025 hazard model (by about 35% and 80% for San Juan and Christiansted, respectively, for the 2% probability of exceedance in 50 years uniform hazard). For both locations, significant hazard contributions are seen from the improved gridded seismicity models for active crustal, deep subduction intraslab, and subduction interface earthquake sources. The highest seismic hazard in this region is on the western side of the island of Puerto Rico (San Juan is located on the northern coast of Puerto Rico), while the lowest hazard is in the northern central part of the US Virgin Island of St. Croix (Christiansted is located on the northern coast of St. Croix). More details on seismic hazard changes are provided in Shumway et al. (2025). When comparing the 2025 to the 2003 model, hazard estimates are generally higher at shorter periods but similar at longer periods. The reasons for the short-period increases are mainly due to introduction of new seismic sources such as the Muertos Trough, new crustal and subduction GMMs, and improved methodologies in the ground-motion characterization. The new PRVI GMMs for crustal and subduction earthquakes are aligned

with what is currently used in the WUS of CONUS and in Alaska. However, adjustments to the PRVI GMMs are made based on a regional ground-motion dataset and given partial weight that slightly reduces the forecasted ground motions. Finally, whereas the ASCE 7-22 approximation of site class amplifications were based on NGA-West2 models in a region similar to those in California with $T_L=12$ sec, the new 2025 hazard model incorporates regionalized site-amplification models with partial weight that show weaker scaling of ground motions with V_{S30} at periods greater than 0.5 sec compared to WUS crustal GMMs. When comparing the ASCE 7-22 estimates of ground motions for soft site classes in this region, short periods increase up to 20% and decrease down to 15% depending on the location, while long periods decrease about 35%. Although the long-period changes may seem dramatic, the resulting values are closer to those in ASCE 7-10 (note that ASCE 7-16 required site-specific analysis in these situations, due to the shortcomings of site coefficients at long periods in California, which led to the adoption of multi-period response spectra and elimination of site coefficients in ASCE 7-22).

Table C22-2. Comparison of Short-Period MCE_R Spectral Response Acceleration Values from This Standard, ASCE 7-22, ASCE 7-16, and ASCE 7-10.

	ASCE/SEI 7-10		ASCE/SEI 7-16		ASCE/SEI 7-22		2026 NEHRP	
Location Name	S_s (g)	S_{MS} (g)	S_s (g)	S_{MS} (g)	S_s (g)	S_{MS} (g)	S_s (g)	S_{MS} (g)
Los Angeles, CA	2.40	2.40	1.97	2.36	2.25	2.37	1.97	2.11
Century City, CA	2.17	2.17	2.11	2.53	2.37	2.49	1.87	2.04
Northridge, CA	1.69	1.69	1.74	2.08	2.09	2.26	2.03	2.21
Long Beach, CA	1.64	1.64	1.68	2.02	1.90	2.03	1.69	1.84
Irvine, CA	1.55	1.55	1.25	1.50	1.43	1.68	1.23	1.51
Riverside, CA	1.50	1.50	1.50	1.80	1.50	1.67	1.67	1.97
San Bernardino, CA	2.37	2.37	2.33	2.79	2.78	2.97	2.93	3.03
San Luis Obispo, CA	1.12	1.18	1.09	1.31	1.23	1.45	1.03	1.29
San Diego, CA	1.25	1.25	1.58	1.89	1.74	1.80	1.68	1.72
Santa Barbara, CA	2.83	2.83	2.12	2.54	2.37	2.44	2.41	2.45
Ventura, CA	2.38	2.38	2.02	2.42	2.25	2.38	2.86	2.90
Oakland, CA	1.86	1.86	1.88	2.26	2.07	2.21	2.46	2.59
Concord, CA	2.08	2.08	2.22	2.67	2.76	2.84	2.01	2.17
Monterey, CA	1.53	1.53	1.33	1.59	1.50	1.75	1.53	1.75
Sacramento, CA	0.67	0.85	0.57	0.76	0.66	0.97	0.57	0.87

San Francisco, CA	1.50	1.50	1.50	1.80	1.57	1.81	1.91	2.10
San Mateo, CA	1.85	1.85	1.80	2.16	2.10	2.30	2.26	2.43
San Jose, CA	1.50	1.50	1.50	1.80	1.50	1.65	2.11	2.35
Santa Cruz, CA	1.52	1.52	1.59	1.91	1.65	1.86	1.74	1.97
Vallejo, CA	1.50	1.50	1.50	1.80	2.33	2.42	1.76	1.98
Santa Rosa, CA	2.51	2.51	2.41	2.90	2.81	2.93	2.83	2.95
Seattle, WA	1.37	1.37	1.40	1.68	1.56	1.76	1.64	1.75
Tacoma, WA	1.30	1.30	1.36	1.63	1.50	1.62	1.63	1.77
Everett, WA	1.27	1.27	1.20	1.44	1.36	1.55	1.26	1.40
Portland, OR	0.98	1.09	0.89	1.07	0.94	1.15	1.01	1.12
Salt Lake City, UT	1.54	1.54	1.55	1.85	1.76	1.81	1.39	1.46
Boise, ID	0.31	0.48	0.31	0.48	0.33	0.50	0.16	0.29
Reno, NV	1.50	1.50	1.47	1.76	1.54	1.74	1.33	1.51
Las Vegas, NV	0.50	0.70	0.65	0.83	0.70	0.82	0.47	0.65
St Louis, MO	0.44	0.64	0.46	0.66	0.63	0.68	0.59	0.63
Memphis, TN	1.01	1.11	1.03	1.23	1.32	1.25	1.10	1.05
Charleston, SC	1.15	1.20	1.42	1.70	1.66	1.52	1.21	1.11
Chicago, IL	0.14	0.22	0.12	0.19	0.15	0.20	0.15	0.19
New York City, NY	0.28	0.44	0.29	0.46	0.28	0.31	0.34	0.37
Washington DC					0.15	0.19	0.22	0.26
Atlanta, GA					0.24	0.31	0.25	0.30
Anchorage, AK					1.50	1.65	3.27	3.27
Fairbanks, AK					1.02	1.23	1.15	1.42
Hilo, HI					1.50	1.65	2.17	2.83
Honolulu, HI					0.53	0.74	0.47	0.71
San Juan, PR					1.00	1.42	0.95	1.31
Christiansted, VI					0.84	1.19	1.05	1.33

Notes: The S_{MS} values are for default site conditions.

As seen in Table C22-3 and Figure C22-2, which are for the 1.0 s MCE_R spectral response accelerations, the S_{M1} values changed from ASCE 7-22 to this standard by more than 20% at nine of the 36 CONUS locations, with two locations at New York City and Washington DC increasing (by 26% and 33%, respectively) and seven locations at Century City, Concord, Vallejo, Salt Lake City, Boise, Reno, and Las Vegas decreasing (by 21%, 24%, 22%, 22%, 35%, 26%, and 25%, respectively). Only at the New York City, Washington DC, and Boise locations, the changes are due to a significant update in the science of USGS NSHM and are explained as follows. At all other locations, the changes are mainly due to the revised site-specific ground motion procedures and definitions of Section 21.2.1. Although the final ground motion values do not change substantially at the Memphis, San Diego, Monterey, and Chicago locations, significant changes to the hazard estimates occurred that are also explained as follows.

- The changes at the locations in Boise, New York City, and Washington DC (from 0.28g to 0.18g, from 0.11g to 0.14g, and from 0.09g to 0.12g, respectively) are for the same reasons as explained for S_{MS} above.
- The estimate of hazard increases at the Memphis location and can be generally attributed to improved geologic characterization and paleoseismicity constraints in the New Madrid fault zone region, particularly an extension of the Axial fault to the southwest (Shumway et al. 2024), and partly due to the bias corrections to the site amplifications in the CEUS. However, in combination with the revised site-specific ground motion procedures and definitions of Section 21.2.1, the final ground motion increases only 1% (from 0.72g to 0.73g).
- The increases in the estimates of hazard at the San Diego location (mainly due to improved paleoseismic event-rate constraints on the Rose Canyon fault), the Monterey location (mainly due to increases in geodetic-based slip rate estimates and improved epistemic uncertainty representation of those estimates on the Monterey Bay–Tularcitos and Reliz faults), and at the Chicago location (largely attributed to improvements in how gridded seismicity sources are modeled in the CEUS). However, in combination with the revised site-specific ground motion procedures and definitions of Section 21.2.1, the final changes for these ground motions are less than 15%.

As seen in Table C22-3, the S_{M1} value increases by only 11% for Anchorage, AK, and does not change for Fairbanks, AK, compared to ASCE 7-22. This is mainly due to the use of the new hazard model that slightly increases long-period ground motion estimates due to a combination of increased earthquake rates and using region-appropriate site amplifications (more details are provided in Powers et al., 2024) compared to the approximate methods of developing MPRS in ASCE 7-22 in combination with decreases that are due to the revised site-specific ground motion procedures and definitions in Chapter 21. The S_{M1} value from ASCE 7-22 to this provision does not change much for Hilo, HI, but decreases by 20% for Honolulu, HI, mainly due to the new definition of site-specific ground motion in Chapter 21. The S_{M1} values decrease significantly for San Juan, PR, and Christiansted, VI, by 42% and 25%, respectively. A significant portion of these decreases at the default site conditions are due to using region-appropriate site amplifications in the 2025 hazard model update compared to the approximate methods of developing MPRS in ASCE 7-22 that were

based on California models and an overestimation for this region at long periods. By comparison, for Site Class BC the decreases are 29% and 5%, respectively. The remaining decreases are due to the new definition of the site-specific ground motion in Chapter 21 that increases the target risk of collapse in 50 years from 1% to 1.5%. The seismic hazard improvements of the 2025 model have decreased the long-period ground motions in San Juan slightly, by about 5%, but increased them in Christiansted by about 40%.

Table C22-3. Comparison of 1.0 s MCE_R Spectral Response Acceleration Values.

	ASCE/SEI 7-10		ASCE/SEI 7-16		ASCE/SEI 7-22		2026 NEHRP	
Location Name	S_1 (g)	S_{M1} (g)	S_1 (g)	S_{M1} (g)	S_1 (g)	S_{M1} (g)	S_1 (g)	S_{M1} (g)
Los Angeles, CA	0.84	1.26	0.70	1.79	0.72	1.66	0.67	1.44
Century City, CA	0.80	1.21	0.75	1.92	0.92	1.84	0.72	1.46
Northridge, CA	0.60	0.90	0.60	1.53	0.70	1.65	0.68	1.50
Long Beach, CA	0.62	0.93	0.61	1.55	0.75	1.58	0.66	1.41
Irvine, CA	0.57	0.86	0.45	1.24	0.45	1.12	0.40	1.01
Riverside, CA	0.60	0.90	0.58	1.50	0.58	1.45	0.54	1.36
San Bernardino, CA	1.08	1.63	0.93	2.37	1.07	2.83	1.04	2.54
San Luis Obispo, CA	0.43	0.67	0.40	1.14	0.40	1.00	0.36	0.91
San Diego, CA	0.48	0.73	0.53	1.40	0.53	1.23	0.51	1.20
Santa Barbara, CA	0.99	1.49	0.78	1.98	0.87	1.89	0.78	1.77
Ventura, CA	0.90	1.35	0.76	1.95	1.01	2.10	1.15	2.29
Oakland, CA	0.75	1.12	0.72	1.83	0.72	1.70	0.81	1.73
Concord, CA	0.74	1.10	0.67	1.72	0.91	1.91	0.67	1.45
Monterey, CA	0.56	0.84	0.50	1.34	0.49	1.21	0.49	1.21
Sacramento, CA	0.29	0.53	0.25	0.79	0.24	0.63	0.26	0.58
San Francisco, CA	0.64	0.96	0.60	1.53	0.60	1.75	0.64	1.42
San Mateo, CA	0.86	1.29	0.74	1.88	0.81	2.30	0.85	1.86
San Jose, CA	0.60	0.90	0.60	1.53	0.61	1.70	0.81	1.69
Santa Cruz, CA	0.60	0.90	0.60	1.54	0.59	1.47	0.60	1.35
Vallejo, CA	0.60	0.90	0.60	1.53	0.89	1.80	0.67	1.40

Santa Rosa, CA	1.04	1.55	0.94	2.39	1.15	2.40	1.05	2.10
Seattle, WA	0.53	0.79	0.49	1.32	0.66	1.38	0.66	1.26
Tacoma, WA	0.51	0.76	0.47	1.29	0.46	0.96	0.49	0.92
Everett, WA	0.48	0.73	0.43	1.20	0.47	0.97	0.42	0.80
Portland, OR	0.42	0.66	0.39	1.13	0.36	0.78	0.34	0.77
Salt Lake City, UT	0.56	0.84	0.56	1.45	0.62	1.28	0.48	1.00
Boise, ID	0.11	0.25	0.11	0.26	0.10	0.28	0.06	0.18
Reno, NV	0.52	0.78	0.52	1.38	0.54	1.32	0.39	0.97
Las Vegas, NV	0.17	0.36	0.21	0.68	0.20	0.51	0.14	0.38
St Louis, MO	0.17	0.36	0.16	0.39	0.18	0.38	0.16	0.34
Memphis, TN	0.35	0.60	0.35	1.02	0.35	0.72	0.34	0.73
Charleston, SC	0.37	0.61	0.41	1.17	0.38	0.77	0.33	0.64
Chicago, IL	0.062	0.15	0.063	0.15	0.070	0.15	0.06	0.14
New York City, NY	0.072	0.17	0.060	0.14	0.051	0.11	0.07	0.14
Washington DC					0.04	0.09	0.06	0.12
Atlanta, GA					0.09	0.20	0.09	0.19
Anchorage, AK					0.65	1.85	0.95	2.05
Fairbanks, AK					0.34	0.85	0.33	0.85
Hilo, HI					0.60	1.70	0.45	1.64
Honolulu, HI					0.15	0.42	0.09	0.34
San Juan, PR					0.35	0.94	0.25	0.54
Christiansted, VI					0.28	0.75	0.27	0.57

Source: From ASCE 7-22, ASCE 7-16, and ASCE 7-10. Notes: The S_{M1} values are for default site conditions. The ASCE 7-16 S_{M1} values include the 1.5 multiplier of the applicable section 11.4.8 exception.

As seen in Table C22-4 and Figure C22-3, which are for the MCE_G peak ground accelerations, the PGA_M values changed from ASCE 7-22 to this standard by increasing more than 20% at 10 of the 36 CONUS locations with values increasing at Northridge, Ventura, San Jose, Santa Cruz, Portland, St Louis, Chicago, New York City, Washington DC, and Atlanta (by 23%, 38%, 77%, 33%, 21%, 33%, 30%, 73%, 86%, and 33%, respectively) and decreasing by more than 20% at one location at Boise (by 37%). Additionally, at seven of the 36 locations, values increased more than 15% but less than

20% at Santa Barbara, Oakland, Monterey, San Francisco, San Mateo, Tacoma, and Reno (by 17%, 18%, 16%, 15%, 17%, 16%, and 18%, respectively). Specific explanation of these changes was provided above for locations at Ventura, San Jose, Santa Cruz, Portland, Chicago, New York, Boise, Santa Barbara, and Monterey. For all other locations, the general explanation is that the source model updates and better modeling of epistemic uncertainties in the 2023 NSHM in general increases the estimates of ground motions as new information about previously unknown sources and added data improve the model; however, unlike S_{MS} and S_{M1} parameters, these increased PGA_M values are not countered with decreases due to GMM models such as lower subduction values for longer periods or de-amplifications being allowed for shallow basin in San Francisco that would only affect longer periods. Furthermore, these values do not get countered with the revised site-specific ground motion procedures and definitions of Section 21.2.1. In Alaska, Anchorage observes a significant increase from 0.56g to 1.29g due to the same reasons as the increase in short period ground motion explained above, but Fairbanks observes a relatively small increase of 10% from 0.49g to 0.54g. In Hawaii, Hilo observes a significant increase due to the same reasons as for short period ground motions, partly due to the new definition of the site-specific ground motion based on Chapter 21 because Hilo was previously deterministically capped, and partly due to the increased hazard estimates at this location compared to the models used about 20 years ago in the 2001 Hawaii hazard model. In Puerto Rico and the US Virgin Islands, San Juan and Christiansted observe a decrease and an increase of 8% and 9%, respectively, due to the same reasons as the changes in short-period ground motions explained above.

Table C22-4. Comparison of MCE_G Peak Ground Acceleration values from this provision, ASCE 7-22, ASCE 7-16, and ASCE 7-10.

	ASCE/SEI 7-10		ASCE/SEI 7-16		ASCE/SEI 7-22		2026 NEHRP	
Location Name	PGA (g)	PGA_M (g)	PGA (g)	PGA_M (g)	"PGA" (g)	PGA_M (g)	"PGA" (g)	PGA_M (g)
Los Angeles, CA	0.91	0.91	0.84	1.01	0.85	0.94	0.87	0.97
Century City, CA	0.81	0.81	0.91	1.09	0.91	0.99	0.83	0.93
Northridge, CA	0.62	0.62	0.71	0.86	0.72	0.80	0.85	0.98
Long Beach, CA	0.64	0.64	0.74	0.89	0.74	0.82	0.77	0.87
Irvine, CA	0.60	0.60	0.53	0.63	0.54	0.64	0.55	0.65
Riverside, CA	0.50	0.50	0.50	0.60	0.50	0.58	0.50	0.60
San Bernardino, CA	0.91	0.91	0.98	1.18	0.95	1.04	1.06	1.15
San Luis Obispo, CA	0.44	0.47	0.48	0.58	0.49	0.58	0.48	0.57
San Diego, CA	0.57	0.57	0.72	0.86	0.71	0.78	0.81	0.88
Santa Barbara, CA	1.09	1.09	0.93	1.12	0.93	1.01	1.12	1.19

Ventura, CA	0.91	0.91	0.89	1.06	0.87	0.95	1.26	1.31
Oakland, CA	0.72	0.72	0.79	0.95	0.71	0.78	0.83	0.92
Concord, CA	0.79	0.79	0.90	1.07	0.95	1.01	0.85	0.97
Monterey, CA	0.59	0.59	0.58	0.69	0.58	0.68	0.69	0.79
Sacramento, CA	0.23	0.31	0.24	0.32	0.25	0.34	0.26	0.35
San Francisco, CA	0.57	0.57	0.58	0.70	0.54	0.63	0.62	0.73
San Mateo, CA	0.73	0.73	0.78	0.93	0.72	0.81	0.84	0.95
San Jose, CA	0.50	0.50	0.58	0.69	0.50	0.56	0.84	0.99
Santa Cruz, CA	0.59	0.59	0.67	0.81	0.58	0.65	0.74	0.87
Vallejo, CA	0.51	0.51	0.62	0.74	0.81	0.88	0.74	0.86
Santa Rosa, CA	0.97	0.97	1.02	1.22	0.95	1.04	1.02	1.13
Seattle, WA	0.56	0.56	0.60	0.72	0.61	0.73	0.71	0.83
Tacoma, WA	0.50	0.50	0.50	0.60	0.50	0.58	0.59	0.67
Everett, WA	0.52	0.52	0.51	0.62	0.54	0.64	0.55	0.68
Portland, OR	0.43	0.46	0.40	0.48	0.40	0.50	0.52	0.61
Salt Lake City, UT	0.67	0.67	0.70	0.84	0.72	0.77	0.70	0.75
Boise, ID	0.12	0.19	0.14	0.21	0.13	0.19	0.08	0.12
Reno, NV	0.50	0.50	0.62	0.74	0.54	0.59	0.60	0.69
Las Vegas, NV	0.20	0.28	0.28	0.37	0.27	0.34	0.23	0.31
St Louis, MO	0.23	0.31	0.27	0.36	0.33	0.33	0.45	0.44
Memphis, TN	0.50	0.50	0.61	0.73	0.74	0.70	0.80	0.75
Charleston, SC	0.75	0.75	0.93	1.12	1.07	0.97	0.99	0.89
Chicago, IL	0.068	0.11	0.059	0.094	0.072	0.081	0.10	0.11
New York City, NY	0.17	0.25	0.18	0.26	0.17	0.18	0.32	0.31
Washington DC					0.08	0.09	0.15	0.16
Atlanta, GA					0.11	0.12	0.15	0.16
Anchorage, AK					0.50	0.56	1.19	1.29
Fairbanks, AK					0.40	0.49	0.44	0.54

Hilo, HI					0.50	0.56	0.92	1.21
Honolulu, HI					0.26	0.35	0.23	0.33
San Juan, PR					0.40	0.54	0.39	0.50
Christiansted, VI					0.35	0.48	0.44	0.52

Notes: The PGA_M values are for default site conditions. This standard has replaced the notation PGA of ASCE 7-10 and ASCE 7-16 with “ PGA_M for Site Class BC,” abbreviated as “ PGA ,” same as in ASCE 7-22

As seen from Table C22-5, two of the SDC values increased from SDC D in ASCE 7-22 to SDC E in this provision, as a result of increases in ground motions at the locations in Oakland and San Jose, both of which are explained above. One location increased from SDC A to B at Washington DC, also explained above. At four locations of Century City, Long Beach, Concord, and Vallejo, the SDC values decreased from SDC E in ASCE 7-22 to SDC D in this provision. The SDC values increase at the two locations in Washington DC and Atlanta, GA from A to B and from B to C, respectively. Interestingly, the change in SDC is not directly reflective of the most significant changes in the 2023 NSHM, in most of these locations even though the percent change is small, the absolute values happened to change to just below or above the threshold defining an SDC. In Alaska, the SDC increases from D to E in Anchorage, but does not change in Fairbanks. The SDC does not change and remains D for the locations in Hawaii, Puerto Rico, and the US Virgin Islands.

Table C22-5. Comparison of Seismic Design Categories (SDC) from this provision, ASCE 7-22, ASCE 7-16, and ASCE 7-10, for Default site conditions and Risk Category I, II, or III.

	ASCE/SEI 7-10		ASCE/SEI 7-16		ASCE/SEI 7-22		2026 NEHRP	
Location Name		SDC		SDC		SDC		SDC
Los Angeles, CA		E		D		D		D
Century City, CA		E		E		E		D
Northridge, CA		D		D		D		D
Long Beach, CA		D		D		E		D
Irvine, CA		D		D		D		D
Riverside, CA		D		D		D		D
San Bernardino, CA		E		E		E		E
San Luis Obispo, CA		D		D		D		D
San Diego, CA		D		D		D		D
Santa Barbara, CA		E		E		E		E

Ventura, CA		E		E		E		E
Oakland, CA		D		D		D		E
Concord, CA		D		D		E		D
Monterey, CA		D		D		D		D
Sacramento, CA		D		D		D		D
San Francisco, CA		D		D		D		D
San Mateo, CA		E		D		E		E
San Jose, CA		D		D		D		E
Santa Cruz, CA		D		D		D		D
Vallejo, CA		D		D		E		D
Santa Rosa, CA		E		E		E		E
Seattle, WA		D		D		D		D
Tacoma, WA		D		D		D		D
Everett, WA		D		D		D		D
Portland, OR		D		D		D		D
Salt Lake City, UT		D		D		D		D
Boise, ID		C		C		C		C
Reno, NV		D		D		D		D
Las Vegas, NV		D		D		D		D
St Louis, MO		D		D		D		D
Memphis, TN		D		D		D		D
Charleston, SC		D		D		D		D
Chicago, IL		B		B		B		B
New York City, NY		B		B		B		B
Washington DC						A		B
Atlanta, GA						B		C
Anchorage, AK						D		E
Fairbanks, AK						D		D

Hilo, HI						D		D
Honolulu, HI						D		D
San Juan, PR						D	D	D
Christiansted, VI						D	D	D

In summary, the S_{MS} and S_{M1} values changed by more than 15% at eight of the locations, four of which (Ventura, Portland, New York City, and Boise) are common to both parameters, all due to different reasons. The PGA_M increased for more locations because the increase in the added sources and uncertainty was not countered by the decreases in GMMs and site de-amplifications observed at longer periods. 8 of the SDCs at the 34 example locations changed from ASCE 7-22 to this provision, mostly from SDC E to D, in two cases from D to E, in one case from B to C, and in one case from A to B, however these changes in SDC do not necessarily correspond to the most significant changes in hazard estimates even though they may result in significant design impacts due to requirements and limitations triggered by the change in seismic design category.

Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Maps

Using the most recent USGS National Seismic Hazard Models (NSHMs) for the conterminous United States, Alaska, Hawaii, Puerto Rico and the U.S. Virgin Islands, and other U.S. territories, MCE_R spectral response accelerations of this standard have been prepared in accordance with the site-specific procedures of Sections 21.2 through 21.4, with approximations for Guam, the Northern Mariana Islands, and American Samoa using procedures documented in FEMA P-2078 (2020b). To prepare the MCE_R maps in this chapter, which are for default site conditions, the site-specific procedures are followed for Site Classes C, CD, and D. In accordance with Section 11.4.2.1, the resulting multi-period response spectra are enveloped. It is from the envelope spectrum that S_{MS} and S_{M1} for default site conditions are computed in accordance with Section 21.4. To derive values of the parameters S_S and S_1 , the site-specific procedures are followed for spectral periods of 0.2 and 1.0 s and Site Class BC.

For the conterminous United States, Alaska, Hawaii, Puerto Rico, and the U.S. Virgin Islands, the risk-targeted probabilistic ground motions for each spectral response period (see Section 11.4.5.1) and Site Classes A through E have been calculated using corresponding USGS hazard curves for a grid of locations. The USGS hazard curves are first converted from geometric mean or median ground motions (output by the ground motion models available to USGS) to ground motions in the direction of maximum horizontal spectral response acceleration. The conversions were done by applying the factors specified in the site-specific procedures commentary (Section C21.2) (e.g., 1.2 at 0.2 s and 1.25 at 1.0 s). See Section C21.2.1 and Luco et al. (2007) for information on the development of risk-targeted probabilistic ground motions from hazard curves.

The ground motions for Guam, the Northern Mariana Islands, and American Samoa are unchanged with respect to ASCE 7-22, which were approximated by means of the procedure documented in FEMA P-2078 (2020b), awaiting future USGS updates for these U.S. territories.

Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Accelerations

Using the most recent USGS NSHMs for the conterminous United States, Alaska, Hawaii, Puerto Rico, and the U.S. Virgin Islands, the MCE_G peak ground accelerations (i.e., the PGA_M values for each site class) of this standard have been prepared in accordance with the site-specific procedures of Section 21.5. The ground motions for Guam, the Northern Mariana Islands, and American Samoa are unchanged with respect to ASCE 7-22, awaiting future USGS updates.

Long-Period Transition Maps

USGS Seismic Design Geodatabase and Web Service

Because maps of MCE_R and MCE_G ground motions for all the site classes (and all the MCE_R spectral periods) are too numerous to include in this standard, gridded values of the ground motions are contained in the online USGS Seismic Design Geodatabase archive, defined in Section 11.2. To spatially interpolate between these gridded ground motions for a user-specified latitude and longitude (and site class), the USGS has developed a web service. The web service returns S_S , S_1 , S_{MS} , S_{M1} , PGA_M , and T_L values, as well as MCE_R multi-period response spectra. These online resources are accessible via <https://doi.org/10.5066/F7NK3C76>.

LIQUEFACTION-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_L) PEAK GROUND ACCELERATIONS

Using the most recent USGS NSHMs for the conterminous United States and Alaska and Hawaii, the Liquefaction-Targeted Maximum Considered Earthquake (MCE_L) peak ground accelerations (i.e., the PGA_L values for each site class) of this standard have been prepared in accordance with the site-specific procedures of Section 21.5. Until updated hazard models are produced for other U.S. territories (Puerto Rico and the U.S. Virgin Islands, Guam and the Northern Mariana Islands, and American Samoa), the Maximum Considered Earthquake Geometric Mean (MCE_G) PGA_M may be used in lieu of the PGA_L for the assessment of liquefaction hazards. This approximation outside of the 50 U.S. states is temporary, awaiting future USGS updates.

The combined impacts of the introduction of the new MCE_L peak ground acceleration in these provisions and the USGS NSHM modifications, on peak ground accelerations used for liquefaction hazards assessment, are demonstrated in Figure C22-4 and Table C22-6 for the same 36 example locations listed in Table C22-1, for the Default and Site Class E conditions. To facilitate a more direct comparison between the PGA_L values that include magnitude-scaling effects and the PGA_M values that require additional selection of representative magnitude values for use in liquefaction triggering models, the ASCE 7-22 values are presented as magnitude-scaled (i.e., PGA_M/MSF), with MSF determined as the weighted-average MSF from the four liquefaction triggering models used to develop the liquefaction-targeted PGA_L (refer to Section C21.5.4 for more details).

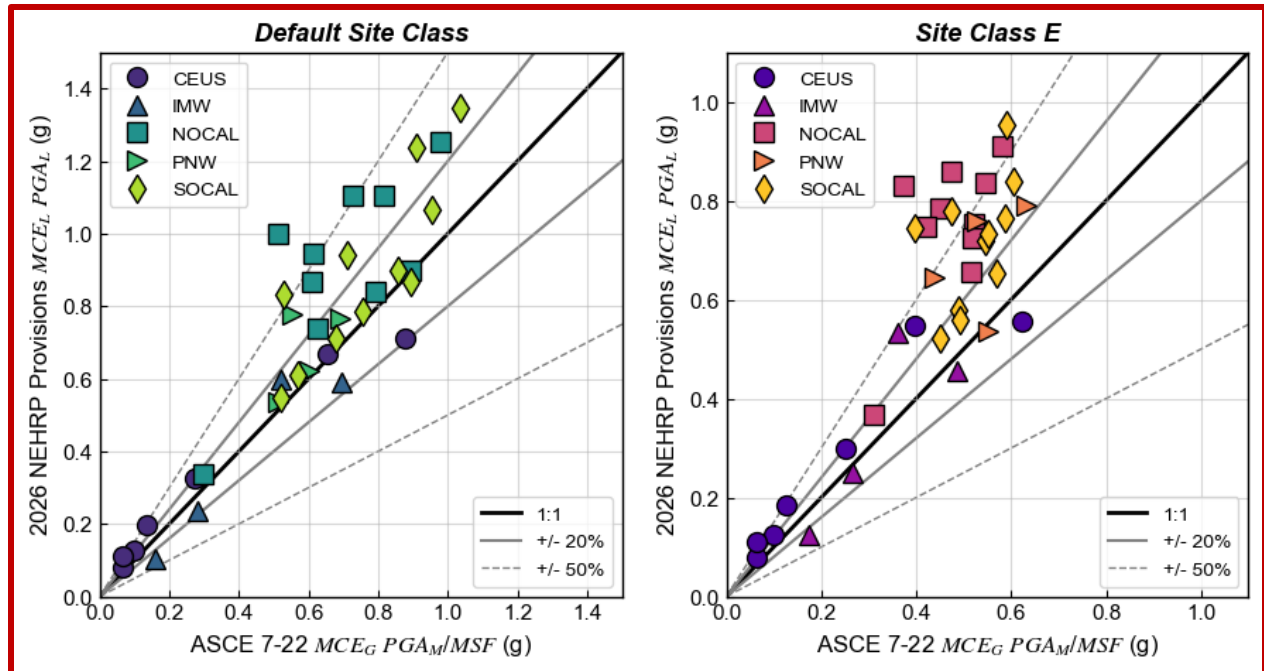


Figure C22-4: Comparison of $MCE_L PGA_L$ in this standard to ASCE 7-22 PGA_M scaled for duration effects for (a) Default Site Class and (b) Site Class E site conditions for 36 example values. **Table C22-6:** Comparison of $MCE_L PGA_L$ values with magnitude-adjusted ASCE 7-22 $MCE_G PGA_M$ values for Default site class and Site Class E conditions. “2018 NSHM” column is included to inform changes in PGA_L due solely to the 2023 NSHM update

Location Name	ASCE 7-22 $MCE_G PGA_M/MSF$ (g)		2018 NSHM $MCE_L PGA_L$ (g)		2026 NEHRP Provisions $MCE_L PGA_L$ (g)	
	Default Site Class	Site Class E	Default Site Class	Site Class E	Default Site Class	Site Class E
Los Angeles CA	0.86	0.55	0.86	0.70	0.90	0.73
Century City CA	0.89	0.55	0.89	0.72	0.87	0.72
Northridge CA	0.71	0.47	0.88	0.74	0.94	0.78
Long Beach CA	0.76	0.57	0.74	0.62	0.78	0.65
Irvine CA	0.57	0.49	0.59	0.56	0.61	0.58
Riverside CA	0.53	0.40	0.78	0.70	0.83	0.74
San Bernardino CA	1.03	0.59	1.20	0.88	1.35	0.95
San Luis Obispo CA	0.52	0.45	0.52	0.49	0.55	0.52
San Diego CA	0.68	0.49	0.61	0.50	0.71	0.56

Santa Barbara CA	0.96	0.59	0.89	0.68	1.07	0.76
Ventura CA	0.91	0.60	0.87	0.68	1.24	0.84
Oakland CA	0.73	0.47	1.02	0.81	1.11	0.86
Concord CA	0.89	0.52	1.06	0.80	0.90	0.75
Monterey CA	0.62	0.52	0.63	0.59	0.74	0.66
Sacramento CA	0.30	0.31	0.33	0.36	0.34	0.37
San Francisco CA	0.61	0.45	0.88	0.74	0.95	0.78
San Mateo CA	0.82	0.54	1.02	0.79	1.10	0.84
San Jose CA	0.51	0.37	0.99	0.83	1.00	0.83
Santa Cruz CA	0.61	0.42	0.78	0.69	0.87	0.75
Vallejo CA	0.79	0.52	0.93	0.76	0.84	0.72
Santa Rosa CA	0.98	0.58	1.09	0.80	1.25	0.91
Seattle WA	0.69	0.53	0.69	0.68	0.77	0.76
Tacoma WA	0.55	0.63	0.71	0.72	0.78	0.79
Everett WA	0.60	0.44	0.60	0.60	0.62	0.64
Portland OR	0.51	0.55	0.48	0.50	0.54	0.54
Salt Lake City UT	0.70	0.49	0.61	0.46	0.59	0.46
Boise ID	0.16	0.17	0.15	0.17	0.10	0.12
Reno NV	0.52	0.36	0.67	0.59	0.60	0.53
Las Vegas NV	0.28	0.27	0.25	0.25	0.24	0.25
Saint Louis MO	0.27	0.25	0.25	0.24	0.33	0.30
Memphis TN	0.65	0.40	0.56	0.46	0.67	0.55
Charleston SC	0.88	0.62	0.68	0.52	0.71	0.55
Chicago IL	0.066	0.064	0.063	0.062	0.080	0.080
New York NY	0.13	0.13	0.11	0.10	0.20	0.18
Washington DC	0.066	0.064	0.059	0.058	0.11	0.11
Atlanta GA	0.10	0.10	0.10	0.096	0.13	0.13

As seen in Table C22-6, the magnitude-adjusted PGA values changes from ASCE 7-22 to this standard by more than 20% (in either direction) at 16 of the 36 test sites for the Default site class condition, with changes at specific sites described as follows:

- The decrease in magnitude-adjusted PGA at the Charleston location (-23%, from 0.88 to 0.71 g) can be attributed to the stringent liquefaction design standard “implied” by the ASCE 7-22 PGA_M relative to the rest of the United States, which resulted in an effective acceptable liquefaction probability of approximately 1.5% in 50 years on average in Charleston. The MCE_L -based acceptable probability of liquefaction, applied consistently across the U.S., is substantially higher at 2.5% in 50 years, resulting in a less stringent design criteria relative to ASCE 7-22 and a corresponding decrease in PGA in Charleston.
- Elsewhere in the CEUS, increases in PGA at Atlanta (+26%, 0.10 to 0.13 g), New York (+47% 0.13 to 0.20 g), and Washington DC (+71%, 0.066 to 0.11 g) can largely be attributed to increases in PGA hazard at relatively lower-hazard locations, due to a number of improvements in the 2023 update of the USGS NSHM. The biggest drivers of these changes are modifications to the new updated background seismicity model with additions and revisions to the seismicity catalog (Field et al. 2024) and improvements to other CEUS sources such as the addition of the Central Virginia fault zone model. It is also important to note that these percent changes seem more extreme due to the much smaller values of ground motion parameters in these locations, whereas the absolute value changes (on the order of 0.05 g) may not be as substantial.
- Increases to PGA at northern California sites such as Oakland (+53%, 0.73 to 1.11 g), San Francisco (+55%, 0.61 to 0.95 g), San Jose (+95%, 0.51 to 1.00 g), San Mateo (+35%, 0.82 to 1.10 g), Santa Cruz (+42%, 0.61 to 0.87g), and Santa Rosa (+28%, 0.98 to 1.25 g) are due to the transition from deterministically-controlled in ASCE 7-22 to probabilistically-controlled ground motions in this standard, due to the new MCE_L definition.
- Increases to PGA at southern California sites such as Northridge (+32%, 0.71 to 0.94 g), Riverside (+58%, 0.53 to 0.83g), and San Bernardino (+30%, 1.03 to 1.35 g) are due to the same deterministic considerations described above for the NOCAL sites. The increase at the Ventura site (+36%, 0.91 to 1.24 g) is for the same reasons as explained for S_{MS} above.
- Elsewhere in the WUS, the increase to PGA in Tacoma (+41%, 0.68 to 0.78 g) is due to the same deterministic considerations described above for the NOCAL sites, while the decrease at the Boise sites (-34%, 0.16 to 0.12 g) is for the same reasons as explained for S_{MS} above.

Table C22-7: Summary of comparisons between MCE_L PGA_L in this standard to the magnitude-adjusted ASCE 7-22 MCE_G PGA_M for Default site class and Site Class E conditions across different regions of 36 example locations. Summary percent changes in PGA correspond to 50th percentile values, with range in parentheses corresponding to 15th to 85th percentile values.

Region	Number of Sites	Default Site Class			Site Class E		
		Change in PGA	Sites with +/- 20%	Sites with +/- 50%	Change in PGA	Sites with +/- 20%	Sites with +/- 50%
CEUS	7	22% (0 to 49%)	4	1	25% (16 to 48%)	5	1
IMW	4	-16% (-26 to 1%)	1	0	-6% (-19 to 24%)	2	0
NOCAL	10	31% (9 to 54%)	6	3	55% (31 to 80%)	9	6
PNW	4	8% (4 to 27%)	1	0	35% (10 to 46%)	3	0
SOCAL	11	7% (4 to 34%)	4	1	32% (15 to 63%)	7	3
ALL	36	15% (1 to 46%)	16	5	35% (14 to 71%)	26	10

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COMMENTARY TO APPENDIX A

Add Commentary to Appendix A as follows:

CA.1 GENERAL

CA.1.1 Scope

This appendix is written within the context that it is not a mandatory part of the provisions but may be adopted by an Authority Having Jurisdiction (AHJ). Therefore, it has been written in “mandatory language” for clarity of implementation if an AHJ chooses to adopt its use. It is recognized that an AHJ may choose to adopt this appendix as applicable to only certain structures. In such a case, the AHJ may need to modify certain portions of the appendix based on the intended scope of adoption. If not adopted by an AHJ, a project owner and/or a project team may voluntarily choose to implement the appendix on a given project when it is desired to consider functional recovery performance. In these instances, the project owner/team will need to perform roles assigned within the appendix to the AHJ. It is also recognized that an AHJ, owner, and/or project team may contemplate only adopting or implementing portions of this appendix. While there is no stated prohibition against doing so, it should be clearly understood that the partial use is not expected to achieve the same target recovery time nor meet the risk and reliability targets stated in the appendix.

Requirements of this appendix do not provide an exception to any requirement contained in any other chapter. These requirements are in addition to, not in lieu of, any other requirements. The requirements of this appendix address functional recovery for ground shaking and associated hazards (e.g., liquefaction, lateral spreading, etc.) but do not specifically address functional recovery for other hazards. Notably, functional recovery for tsunami hazards is not included by the fact that reference to Chapter 6 is not made in this appendix. However, the design team, owner, or jurisdiction may want to consider the effect of tsunami on functional recovery if a structure is within the inundation area.

The focus of this appendix is the design of new buildings and nonbuilding structures for functional recovery. However, additions, alterations or repairs to such structures, if not performed in conformance with the requirements of this appendix, could jeopardize the intended functional recovery performance. This appendix is therefore also intended for use in conjunction with the provisions for existing structures of the applicable building code.

CA.1.2 Seismic Design Category

This appendix does not modify the Seismic Design Category assigned by Chapter 11.

CA.1.3 Exemptions

Certain exemptions to this appendix are provided. Exemptions differ from deemed to comply in that there is no assurance for exemptions that the functional recovery objective is achieved. Structures assigned to Seismic Design Category A are exempt from seismic requirements altogether and it is therefore clarified that they are also exempt from the requirements of this appendix. Furthermore, the exemptions provided in Chapter 11 are carried forward to this appendix for consistency. The exemption of one- and two-story wood-frame homes is extended from only those designed per the International Residential Code (IRC) to also those designed per the International Building Code (IBC) without the requirements of this appendix. Functional Recovery Category E has no target recovery time and is therefore, by definition, exempt from the requirements of this appendix.

CA.1.4 Deemed to Comply

Deemed to comply differ from exemptions in that deemed to comply cases are considered to meet the functional recovery objective of this appendix, just without further explicit analysis or design beyond that already required by the Standard. The justification for Item 1 comes from ATC-138 project studies (ref). Item 2 for seismically isolated structures is based on generally favorable recovery performance in past earthquakes and committee judgment rather than detailed analytical studies.

CA.1.5 Applicable Requirements

Buildings There are generally two alternative paths to compliance for buildings, a prescriptive versus an explicit assessment. An explicit assessment requires a detailed calculation of the functional recovery time; a prescriptive assessment does not. Parameters for prescriptive assessment of buildings with damping have not yet been developed. Therefore, such buildings must use an explicit assessment.

Nonbuilding Structures Nonbuilding structures are only provided with a prescriptive assessment path to compliance, unlike buildings. Parameters for prescriptive assessment of nonbuilding structures with damping have not yet been developed. Therefore, such nonbuilding structures cannot be designed in accordance with this appendix at this time.

CA.1.6 Independent Review Requirements for Prescriptive Assessment of Buildings Assigned to Functional Recovery Category A

Independent review of buildings assigned to Functional Recovery Category A is required by this section. For the majority of this appendix, requirements for structures assigned to Functional Recovery Category A are the same as those for Functional Recovery Category B. Without independent review, structures assigned to Functional Recovery Category A are therefore expected to perform nearly equivalent to structures assigned to Functional Recovery Category B. These independent review requirements therefore intend to achieve a greater, though not quantified, reliability required for individual element, component, equipment and distribution design for structures assigned to Functional Recovery Category A than for B. Based on committee judgment, the introduction of

independent review plus the further limitations on structural irregularities in Section A.8 for structures assigned to Functional Recovery Category A improves functional recovery performance from 3 days (Functional Recovery Category B) to close to immediate.

Some of the independent review requirements in this section overlap with those in Section A.17. When Section A.17 is used for structures assigned to Functional Recovery Category A, there is therefore some benefit in combining the independent review scope of this section and Section A.17 under one or more reviewer(s). See also commentary Section CA.17.4.

CA.1.7 Target Risk and Reliability

Within the functional recovery provisions, the recovery metric is the time after an earthquake within which a structure is intended to recover its basic intended function. The functional recovery time is specific to the Functional Recovery Category of the structure and represents the maximum allowable time to recover for various community functions and services. These maximum allowable recovery times were established based on the perceived recovery time required to prevent short and/or long-term consequences such as population dislocation at the community level. Reference Section CA.5.

The reliability target is the probability that a structure will not recover its basic intended function within the target functional recovery time given the occurrence of FRE_R shaking. A reliability target equal to 25% therefore results in a 75% probability of achieving the target functional recovery time if the FRE_R occurs. The risk target, on the other hand, is the probability that a structure will not recover its basic intended function within the target functional recovery time over its entire design life considering all potential sources of seismic shaking intensity (i.e., the full hazard curve). A risk target equal to 10% therefore results in a 90% probability of achieving the target functional recovery time over a structure's design life. The design life of structures for this purpose is taken as 50 years.

For structures designed to Functional Recovery Category A, B, or C requirements in accordance with this appendix, the risk target results in a 90% probability that the structure will satisfy its target recovery time within its design life. Structures designed to Functional Recovery Category D requirements are expected to have a higher risk of exceeding the target recovery time within a structure's design life as explained in Section CA.6.3.2.

The target recovery times in Table A.1-3 specifically apply to Section A.17. However, the prescriptive provisions in Sections A.6. through A.16 (excluding Section A.9) were developed by targeting similar risk, reliability, and functional recovery times as presented in this section.

It should be noted that the assessment of functional recovery time is inherently probabilistic in nature and captures many uncertainties. Although there is very limited empirical data, recovery time after past earthquake events is highly variable (Comerio and Blecher 2010, Tomblason et al., 2018). The assessment procedure considers these uncertainties as well as uncertainties in representing structural response, damage, and recovery time. As a result, existing procedures for assessment of functional recovery or recovery times are associated with large uncertainties (e.g. Terzic et al. 2021, Molina Hutt et al. 2022, Cook et al. 2022). Additional uncertainty is also introduced by (1) the wide

variety of systems, configurations, and basic intended functions permitted under this appendix, (2) the various interpretations and implementation practices of individual design professionals, and (3) the specific socio-economic environment of the community. As such, the calculated functional recovery time may not precisely represent the true functional recovery time for a given building. Ultimately, structures designed in accordance with this appendix are expected to satisfy the “General Expectations” as presented in Table A.1-3. However, conformance to this appendix is not a guarantee of future performance.

References

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Cook, D. T., Liel, A. B., Haselton, C. B., and Koliou, M., 2022. A framework for operationalizing the assessment of post-earthquake functional recovery of buildings, *Earthquake Spectra*, Volume 38, Issue 3.

Molina Hutt, C., Vahanvaty, T., and Kourehpaz, P., 2022. An analytical framework to assess earthquake-induced downtime and model recovery of buildings, *Earthquake Spectra*, Volume 38, Issue 2.

CA.2 Definitions

This section includes explanation of terms used in the Provisions of this appendix. It also includes definition and explanation of additional terms that are considered important for conceptual understanding of the broader context in which these Provisions are intended to operate. These include the range of potential recovery states, factors important to recovery-based design, and additional considerations used in the development of the functional recovery design provisions contained herein,

BASIC INTENDED FUNCTION: Basic intended function is a level of function that can be less than full pre-earthquake functionality. It assumes that not all functions and services in a structure are necessary for serving its intended function and allows for the possibility of damage that renders structural and nonstructural systems serving non-critical functions or services (e.g., amenities) to be non-functional. The subset of activities or range of activities constituting basic intended function are

designated by the project team, based on the structure occupancy classification(s) and use designation(s).

In these Provisions, the precise definition of what constitutes basic intended function has been left to the knowledgeable parties of the design team because, outside of certain basic occupancy requirements, the determination of what is critical or non-critical to function can be highly specific to a particular structure, the resilience needs of a community, the business needs of an owner, or the level of tolerance to inconvenience that might be considered acceptable to users following a damaging earthquake. Defining basic intended function is about defining a relative level of functionality by choosing which structural and nonstructural systems will be subject to functional recovery design requirements, and therefore, expected to be functional within the target functional recovery time.

Certain structural and nonstructural systems are considered necessary for basic occupancy of most structures and therefore required for basic intended function. These include seismic and gravity force-resisting systems, egress systems, fire protection, electrical power, exterior enclosures, potable water, and sanitary drainage systems. Other nonstructural systems may or may not be necessary for a particular situation. Examples of possible basic intended function designations include the following:

- Low-rise residential occupancies in temperate climates. Basic intended function includes shelter and security. Heating, ventilation, and air -conditioning systems, elevator systems, and telecommunications and data systems might be considered unnecessary.
- Multi-story residential occupancies in extreme climates. Basic intended function includes shelter, security, vertical egress, and climate control. Heating, ventilation, and air -conditioning systems, and elevator systems might be considered necessary. Telecommunications and data systems might be considered unnecessary.
- Multi-story commercial office occupancies in extreme climates. Basic intended function includes housing of business operations, security, vertical egress, climate control, and internet and data capabilities. Heating, ventilation, and air -conditioning systems, elevator systems, and telecommunications and data systems might be considered necessary.
- Low-rise retail (e.g., grocery, pharmacy, or hardware) in temperate climates. Basic intended function includes support for retail operations, security, temperature-controlled storage, and internet and data capabilities. Heating, ventilation, and air -conditioning systems, and telecommunications and data systems might be considered necessary. Elevator systems might be considered unnecessary.

Structural and nonstructural systems serving the basic intended function of a structure must be designed to be functional within the target *recovery time* associated with the specified *functional recovery category*.

EXTERNAL FACTOR: External factors are conditions outside the *system boundary* or outside the project team’s control, which might impact functionality of the structure, but are not directly related to the response or performance of the structure itself. Examples of external factors include:

- Disruption of utilities (e.g., electrical power grid, potable water supply, or sanitary sewer).
- Disruption of regional transit or transportation systems.
- Release of hazardous materials in the vicinity.
- Staff availability.
- Supply chain disruption.
- Local or area-wide cordons related to safety, or performance of adjacent structures or infrastructure.
- Disruption of government services.
- Improvements required by the Authority Having Jurisdiction (AHJ)

External factors are characterized as beyond the control of the project team. For example, the project team can control the mechanical, electrical, and plumbing (MEP) systems up to the point where they connect to distribution lines installed by utilities or service providers. However, the project team cannot influence or control the performance of a service provider’s network, and lack of external utilities or services can impact the function of a structure. These and other similar effects are considered external factors.

“Improvements required by the AHJ” refers to regulations that would require components or systems to be retrofitted or improved beyond their “pre-earthquake condition.” Some examples include:

- Damage that exceeds certain thresholds (for example, substantial structural damage as defined in the IEBC) triggering a seismic evaluation and possible retrofit.
- Code requirements triggering new accessibility or fire safety requirements.
- The need to remove or replace prohibited or obsolete materials (such as asbestos, aluminum wiring, or combustible roofs) damaged in an earthquake, because the “pre-earthquake condition” is no longer permitted.

Any such regulations enforced after the original design would not have been known to the original project team and are considered outside of their ability to control. Although improvements required by the AHJ might be necessary for obtaining a permit and conducting repairs needed to achieve a desired *recovery state*, these *Provisions* do not intend to require consideration of regulations that are not already in place at the time of the design.

Although external factors are outside the control of the project team, their impacts on functionality can be mitigated through implementation of temporary work-around solutions. The possibility of work-around solutions for mitigating the effects of external factors does not change their categorization as external factors. Examples include:

- Loss of electric utility service can be mitigated by providing an emergency power supply using backup generators or solar panels with battery storage.
- Loss of water or sewer services can be mitigated with onsite storage tanks and external temporary toilets.
- Supply chain disruption can be mitigated by maintaining a surplus of materials on site.

The concept of a *system boundary* is introduced to define the extent of what is controlled by the project team and allow for consideration of structures on sites that might include appurtenant structures, site equipment, and interconnecting elements that directly serve the functions of the structure and might be included with the permit for construction.

FULL RECOVERY: Full recovery is the post-earthquake *recovery state* in which all damaged components are repaired, and *external factors* are not limiting function of the structure. Full recovery is intended to recognize that the recovery timeline continues beyond the point at which all damage has been repaired and represents a structure that has been returned to its pre-earthquake state, with any improvements required by the Authority Having Jurisdiction (AHJ), and with all functional impacts associated with *external factors* eliminated.

Although *external factors* are beyond the control of the project team, the full *recovery state* expects the structure to be functional, which includes, among other things, the availability of external utilities and services.

Some structures may be fully repaired and ready to support full function, but may take a long time to achieve, or may never reach, a full *recovery state* due to *external factors* such as long-term delays in services that have not yet been restored in the community, persistent supply-chain disruptions, regional transportation disruptions (e.g., bridges that might take decades to restore), or customers that are no longer in the community to return businesses to pre-earthquake levels of profitability.

FUNCTIONAL RECOVERY: Functional Recovery is an intermediate post-earthquake *recovery state* between the reoccupancy and full *recovery states*. Achieving reoccupancy is considered a necessary precursor for achieving functional recovery, therefore, the functional *recovery state* can be achieved concurrent with, but not before, the reoccupancy state. Similarly, the functional *recovery state* can be achieved concurrent with, but not after, the full *recovery state*.

Functional recovery is focused on *basic intended function*. A structure can offer amenities that are not counted as part of its *basic intended function*, which means that functional recovery can be achieved if certain non-critical systems are not yet restored to a functional condition. In the functional *recovery state*, such amenities are not expected or required to be functional, and

restoration of these amenities would require additional time to complete repairs or other actions to restore functionality.

Functional recovery excludes the effects of *external factors* because they are beyond the control of the project team. As a result, a structure that achieves a functional *recovery state* might not be functional because of service disruptions caused by *external factors*. The term “adequately support” is intended to allow for the possibility that a structure in the functional *recovery state* is itself ready to support its *basic intended function* but might not be functional because of a lack of utilities or other services external to the structure.

FUNCTIONAL RECOVERY CATEGORY: Structures are assigned to a functional recovery category based on their relative necessity to the recovery of community functions or services following an earthquake. Structures that support more critical functions or services are assigned to higher functional recovery categories, and those that support less critical functions or services are assigned to lower functional recovery categories. A project team can voluntarily select a higher functional recovery category to achieve shorter *functional recovery times* in the event of an earthquake. Similarly, an Authority Having Jurisdiction (AHJ) adopting these provisions can adjust the functional recovery category table to meet the resilience needs of their community.

FUNCTIONAL RECOVERY EARTHQUAKE (FRE_R): The Functional Recovery Earthquake (FRE_R) is a risk-based characterization of the earthquake hazard used to design buildings to meet functional recovery objectives. The FRE_R is defined as the ground shaking intensity at which both the functional recovery risk target and the functional recovery reliability target in Section A.1.7 are satisfied.

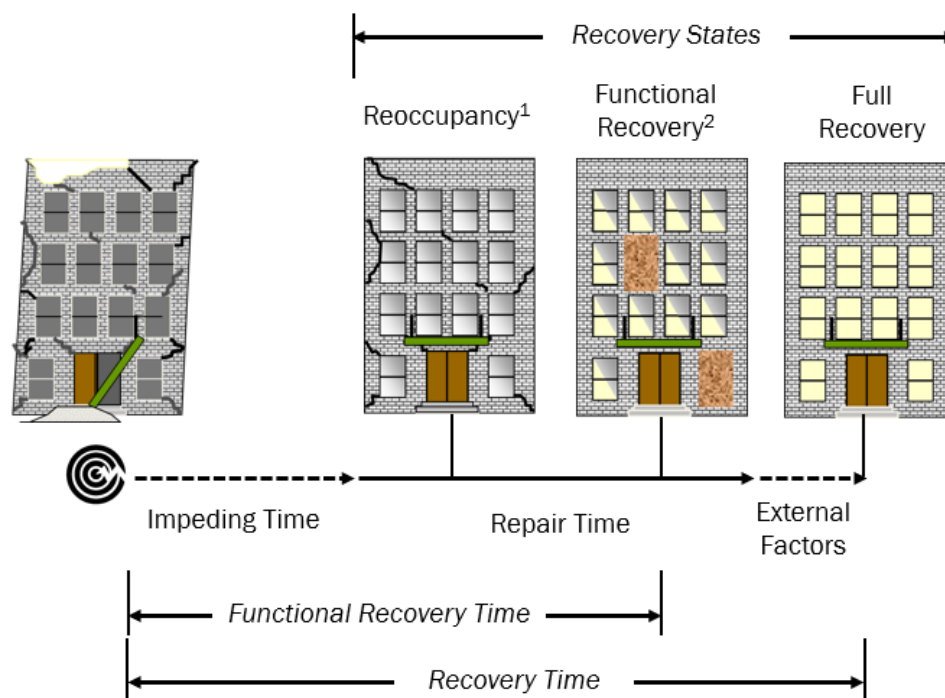
IMPEDING FACTOR: Impeding factors are related to the process of repairing and recovering from damage to structural and nonstructural components. Impeding factors delay or extend the time to complete repairs and, consequently, the time to restoration of function. Although impeding factors most commonly delay the start of repairs, they can also occur at any time during the process causing a delay in the completion of repairs. Examples of impeding factors include:

- Time to conduct post-earthquake inspections.
- Hiring an engineer, including mobilization and design time.
- Hiring a contractor, including mobilization time.
- Procuring financing for repair work, including time to process an insurance claim and receive an insurance payout.
- Obtaining permits or approvals for repair work.
- Supply chain delays and demand surge associated with construction materials or labor, including replacement of equipment with long lead times.

Although considered an external factor, improvements required by the Authority Having Jurisdiction (AHJ) might be necessary for obtaining a permit and conducting repairs needed to achieve a desired *recovery state* and could further delay time to restoration of function. Discussion on improvements required by the AHJ is provided with the commentary on external factors.

RECOVERY OBJECTIVE: A target *recovery time* for achieving a specified *recovery state*, at a target reliability, and at a target risk over a defined time period. A recovery objective is a description of the targeted performance state in terms of time to recovery of function in combination with a risk-based characterization of the earthquake hazard. Risk-based characterization of the earthquake hazard relies on both a specified risk target and a specified reliability target. The risk target is the probability of not achieving a design recovery objective over a specified time period (e.g., design life, or 50 years). The reliability target is the conditional probability of not achieving a design recovery objective, given the occurrence of a specified ground motion (e.g., the FRE_R).

RECOVERY STATE: Consideration of functional performance involves the identification of *recovery states* that define performance in terms of different levels of functionality along a theoretical recovery timeline. In concept, *recovery states* identify a progression from lower levels of functionality to increasing levels of functionality. Possible *recovery states* include reoccupancy, functional recovery, and full recovery. In general, requirements would be expected to vary among *recovery states*, ranging from minimum requirements for reoccupancy, to more extensive or more restrictive requirements for *functional recovery*, and ultimately full recovery. These *Provisions* provide requirements for achieving a functional *recovery state*, therefore, the reoccupancy and full *recovery states* are not explicitly mentioned or used in the *Provisions*.



1. A structure in a reoccupancy state lacks the functionality of certain nonstructural systems serving the basic intended function. However, a reoccupancy state might be sufficient to serve the resilience needs of an individual community or jurisdiction.
2. A structure that achieves a functional recovery state might not be functional because of disruptions to utility services caused by external factors.

Figure CA.2-1 Recovery timeline showing the relationship between the reoccupancy, functional recovery, and full recovery states.

Impeding factors are considered in all *recovery states*. Inclusion or exclusion of *external factors* is a matter of the specific *recovery state*. *External factors* are not considered in the reoccupancy and functional *recovery states* but are considered in the full *recovery state*.

RECOVERY TIME: The time to achieve a specified *recovery state* after an earthquake. Recovery time includes the effects of *impeding factors* but may or may not include the effects of *external factors*. Recovery time is specific to the *recovery state* (i.e., reoccupancy time, *functional recovery time*, and full recovery time). Because each *recovery state* requires a different subset of components and systems to be functional, recovery times will be different for each state. The time to achieve a specified *recovery state* involves repair of damage to systems necessary for a specific level of functionality, but can involve more than just repair, including adaptation, temporary measures, and other work-around solutions that serve to restore levels of functionality.

Impeding factors have the effect of delaying or extending the time to complete repairs. *Impeding factors* are considered in the recovery times associated with all *recovery states* because the presence (or absence) of *impeding factors* depends on damage to the structure, which can be controlled through design.

Inclusion or exclusion of *external factors* in recovery time is specific to the *recovery state*. *External factors* are not considered in reoccupancy and *functional recovery times* but are considered in full recovery times.

REOCCUPANCY: Reoccupancy is the post-earthquake *recovery state* in which a structure is maintained, or restored, to allow safe re-entry of occupants or to provide shelter or protection to contents. In the reoccupancy *recovery state*, a structure is expected to be free from earthquake-caused hazards that might negatively impact the general health and safety of occupants. While in this state, a structure provides limited functionality of sheltering occupants or protecting contents, but not the *basic intended functions* associated with *functional recovery*.

Permissible occupancy of a structure is subject to the discretion of the Authority Having Jurisdiction (AHJ), who may consider legal and practical precedents among a myriad of federal, state, and local jurisdictional requirements establishing minimum criteria for occupancy. Reoccupancy of buildings following a damaging earthquake is often a complex and nuanced process involving consideration of many, often competing, interests and priorities, including consideration of possible environmental hazards. Guidance on the development of temporary habitability standards for use in a post-disaster setting can be found in FEMA P-2055, *Post-Disaster Building Safety Evaluation Guidance* (FEMA, 2019).

Reoccupancy excludes consideration of *external factors* because the availability of external utilities and services are beyond control of the project team. Although not specifically identified, improvements related to applicable Authority Having Jurisdiction (AHJ) requirements might be necessary for achieving a reoccupancy state, which could increase reoccupancy time. Additional discussion on improvements required by the AHJ is provided with the commentary to full recovery.

REPAIR TIME: The time to repair damage necessary to achieve a specified *recovery state* after an earthquake, excluding the effects of *impeding factors* and *external factors*. Repair time is specific to the *recovery state* (reoccupancy, *functional recovery*, and full recovery). Repair time is the time needed to conduct repair actions once the contractor is mobilized and construction has commenced. Because each *recovery state* requires a different subset of components and systems to be functional, repair times will be different for each state.

Repair time differs from *recovery time* because repair time excludes consideration of *impeding factors*, which can delay or extend the time to complete repairs, and are considered part of *recovery time*. Repair time excludes consideration of *external factors* because the availability of external utilities and services are beyond control of the project team.

STRUCTURE: *Provisions* are applicable to both buildings and nonbuilding structures. As defined in ASCE 7, this term encompasses both. As used herein, it is intended to refer to both structural and nonstructural systems within both buildings and nonbuilding structures.

SYSTEM BOUNDARY: The concept of a system boundary allows for consideration of structures on sites with appurtenant structures, equipment, and interconnecting mechanical, electrical, and

plumbing (MEP) systems that are beyond the outside edge of the structure itself but are included with the permit. The intent is to allow for the inclusion of interconnected structures and MEP systems up to the defined utility connection point.

CA.3 SYMBOLS

No commentary proposed.

CA.4 BASIC INTENDED FUNCTION

CA.4.1 Responsibility

It is recognized that basic intended function may be very project-specific and identification of it is therefore assigned to the knowledgeable parties of the design team. It is expected that the responsible parties will need to meet multiple times with the owner to understand what services need to be provided to serve basic intended function.

In this appendix, the precise definition of what constitutes basic intended function has been left to the knowledgeable parties of the design team because, outside of certain basic occupancy requirements, the determination of what is critical or non-critical to function can be highly specific to a particular structure, the resilience needs of a community, the business needs of an owner, or the level of tolerance to inconvenience that might be considered acceptable to users following a damaging earthquake. Defining basic intended function is about defining a relative level of functionality by choosing which structural and nonstructural systems will be subject to functional recovery design requirements, and therefore, expected to be functional within the target functional recovery time. See also Section CA.2 for further description of basic intended function.

CA.4.2 Systems Supporting Basic Intended Function

In this section, the systems which potentially serve the basic intended function are separated into minimum required versus need to be considered but may not be required. Certain structural and nonstructural systems are considered necessary for basic occupancy of most structures, and represent the minimum required for basic intended function. These include seismic and gravity force-resisting systems, egress systems, fire protection, electrical power, exterior enclosures, potable water, and sanitary drainage systems. Without these systems, it is presumed that a structure would not meet the basic intended function likely to be designated for the majority of structures designed using this appendix. For example, it is unlikely, if not impossible, for the basic intended function of any structure to be maintained without the seismic and gravity structural components being designed and constructed to meet functional recovery objectives. Other systems listed may be needed to support the basic intended function of a certain structure but may not be needed to support the basic intended function of another. For example, elevators may not be necessary for a building with a limited number of stories but may be for a taller building. As another example,

telecommunication and data systems may not be necessary for the basic intended function of a residential building but may be for the basic intended function of an office occupancy.

Examples of possible basic intended function designations include the following:

- Low-rise residential occupancies in temperate climates. Basic intended function includes shelter and security. Heating, ventilation, and air -conditioning systems, elevator systems, and telecommunications and data systems might be considered unnecessary.
- Multi-story residential occupancies in extreme climates. Basic intended function includes shelter, security, vertical egress, and climate control. Heating, ventilation, and air -conditioning systems, and elevator systems might be considered necessary. Telecommunications and data systems might be considered unnecessary.
- Multi-story commercial office occupancies in extreme climates. Basic intended function includes housing of business operations, security, vertical egress, climate control, and internet and data capabilities. Heating, ventilation, and air -conditioning systems, elevator systems, and telecommunications and data systems might be considered necessary.

Low-rise retail (e.g., grocery, pharmacy, or hardware) in temperate climates. Basic intended function includes support for retail operations, security, temperature-controlled storage, and internet and data capabilities. Heating, ventilation, and air -conditioning systems, and telecommunications and data systems might be considered necessary. Elevator systems might be considered unnecessary. The systems described are an intentionally fairly exhaustive list of systems which occur within buildings and nonbuilding structures. Where a system does not occur within a structure (e.g., a nonbuilding structure that does not have an exterior enclosure), that system need not be designed and constructed in accordance with this appendix. Examples of systems falling under the last item of Section A.4.2.2 include medical gas systems in a hospital or fire engine door systems in a fire station. It would also include any structural systems not part of the gravity or seismic force-resisting system (e.g., for wind or flood resistance only) if damage to such systems during an earthquake could affect the basic intended function of the structure.

A means of egress system, in accordance with applicable building codes such as the International Building Code, consists of three separate and distinct parts: the exit access, the exit, and the exit discharge. It should be noted that the means of egress system could be interpreted to extend from any point within a building to the final exit (e.g., from a bedroom in a multiple unit apartment building all the way to the exit). In other words, it could be interpreted as the entire building. This is not the intent of these provisions. Rather, the provisions intend to address the common means of egress (e.g., common hallways, stairs, etc.).

Note that some systems listed as additional (i.e., not minimum required) may still need to meet the requirements of this appendix even if they are not required to support the basic intended function where failure of that system could lead to functional failure of a required system. This is explained further in Section CA.4.3.

CA.4.3 Exemptions

By definition, anything not directly supporting basic intended function is exempt so long as its damage does not adversely affect the basic intended function. Elements, components, equipment, and distribution systems which may adversely affect basic intended function even if they do not directly support it (i.e., sequential damage originating in a component not directly supporting basic intended function but progressing to an element which does) cannot be exempted. An example of such a component is a water line serving an ornamental fountain in the lobby of an office building. The building can meet its basic intended function (i.e., occupants able to work in their offices) without the fountain operating. However, breakage and leakage of the water line would adversely affect function by damaging ceiling, drywall, carpet, computer servers, etc. Another example is an in-line component in a mechanical duct serving a non-basic intended function system which is installed in close proximity to a distribution system which serves the basic intended function. If the in-line component is not properly braced, it could swing into and damage the distribution system, thus affecting basic intended function.

Note that certain elements, components, equipment or distribution systems that are part of a system supporting basic intended function may not be required to be designed in accordance with this appendix. For the example of the ornamental fountain, the rest of the potable water system would need to be designed in accordance with this appendix but the fountain itself may not if it can meet the first exemption.

The exemption for furniture reflects the fact that furniture is not part of a building permit and therefore cannot be enforced by this standard. This fact should not imply that damage to furniture or damage by furniture to other elements, components, equipment, or distribution systems cannot affect the structure's basic intended function. For example, falling of high aspect ratio (e.g., greater than 2:1) furniture may require significant clean-up and can damage other equipment serving the basic intended function. Additionally, lower aspect ratio furniture that is not restrained can slide, potentially severing electrical power to necessary equipment. Furthermore, careful attention should be paid to avoid misclassifying components or equipment as "furniture" when they fall under the purview of the building permit. The exemption for temporary components reflects that temporary components are judged to have a low enough probability of being present at the time of the earthquake. Additionally, mobile components are permitted to not be designed for structures assigned to Functional Recovery Category C and D. Mobile equipment/components are defined as equipment/components, with or without wheels or rollers, that are typically used in a different location than where they are stored and are moved from one location in the building to another during ordinary use.

The exemptions of Chapter 13 are generally pulled forward and modified or eliminated in this appendix. For example, exemptions for furniture, temporary components, and mobile units are pulled forward and modified in this section. Further exemptions can be found in Section A.10 and A.11.

CA.5 FUNCTIONAL RECOVERY CATEGORY

CA.5.1 Designation of Functional Recovery Category

Scope.

Table A.5-1 is intended to inform decisions pertaining to the functional recovery category chosen for structures depending on their contribution to a community's timely recovery in the aftermath of an earthquake event. The shift from a focus on life safety to functional recovery for new structures sets the stage for whole community resilience, in accord with the NIST-FEMA report to Congress (FEMA P-2090/NIST SP-1254, 2021) and FEMA's National Resilience Guidance (NRG, 2024).

The determination of what functional recovery category a structure should be assigned to should be left to the owner or the operator of the facility (when this appendix is adopted on a voluntary basis), the applicable building code, or the authority having jurisdiction (FEMA P-2082-1 Sec 1.1.5 Functionality of Critical or Essential Facilities). While the establishment and prioritization of the Functional Recovery Categories has been informed by the consensus of subject matter experts, the target recovery time of a structure should be determined and assigned based on the degree to which its basic intended function contributes to short-term, intermediate, and long-term community recovery or the recovery goals of the owner.

The highest recovery category, Functional Recovery Category A, is intended for those structures that provide a basic intended function that a community relies on for immediate post-earthquake safety and survival. Committee expertise and discussions identified that structures that support: (1) emergency and acute healthcare, and (2) housing of non-ambulatory populations and people incapable of self-preservation (e.g., nursing homes, assisted living) and people in custodial care (e.g., prisons) should be placed in FRC A. These facilities are critical for life safety. They are necessary to stabilize an emergency situation, prevent further harm, and support the establishment of response and recovery efforts.

Structures assigned to Functional Recovery Category B are those with basic intended functions that the community relies on to support urgent and basic human survival needs and that prevent the escalation of adverse disaster consequences. This includes structures that support lifeline infrastructure for the community, as well as structures located in the community that support national economic stability (e.g., oil refinery). Committee expertise and discussions identified that structures that provide basic human needs, such as non-urgent healthcare, food, shelter, and water, should be placed in FRC B because these structures support self-preservation and sustain the community in the short term. These structures help maintain day-to-day life, prevent or deter short- and long-term population displacement, and provide a reprieve for populations who may not have options for housing and sustenance due to disruption in the community. By facilitating access to the basic services and resources that these structures support, these facilities help prevent the spread of disease and injury, support recovery, and contribute to the well-being of the community while minimizing disruptions to normal life. Examples of structures in FRC B might include structures that support: (1) food supplies and potable water services, both wholesale and retail, (2) vital utility

services (e.g., electrical, fuel, water, wastewater, telecommunication, internet), and (3) multifamily housing, particularly in communities with limited available and/or affordable housing.

Structures assigned to Functional Recovery Category C are those with basic intended functions that the general community relies on to support recovery-focused self- and group-preservation, and that sustain short- and long-term economic, educational, and governance activities and services. Committee expertise and discussions identified that structures that support everyday core life activities, such as education, childcare, and economic activity should be placed in FRC C because these structures aid in the early stages of community recovery while minimizing significant disruptions to daily life or economic stability. While the community can tolerate the temporary loss of these services without severely affecting basic survival or critical recovery, their restoration is nonetheless necessary to minimize long-term population displacement, support economic recovery, and minimize long-term disruptions to the community's social fabric. Examples in Functional Recovery Category C might include structures that support (1) PK-12 education, and (2) services critical to economic stability (e.g., employers, supply chain dependencies).

Structures assigned to Functional Recovery Category D are those with basic intended functions that the community relies on to enhance a community's general well-being and expedite the return to normalcy. Committee expertise and discussions identified that structures that support long-term community vitality, such as courthouses, universities, and local commercial employers, should be placed in FRC D because while these structures are essential for longer-term economic recovery and community revitalization, they are not immediately critical for community recovery. These services contribute to the broader recovery process by supporting a broader range of employers, facilitating economic activity, and providing community amenities. While they can be temporarily disrupted or operate with alternative arrangements (e.g., virtual court hearings), their restoration is crucial for returning to a fully functioning community. These facilities, though not essential for basic survival, play a key role in re-establishing normality and driving long-term economic health. Examples in Functional Recovery Category D might include structures that support: (1) post-secondary education and (2) customary local government services.

Finally, structures not assigned to Functional Recovery Categories A-D fall into Functional Recovery Category E and are excluded from any functional recovery design requirements. Examples in Functional Recovery Category E might include structures that support entertainment and recreation.

Basis for recovery categorization, timeframes, and descriptions. While the general public may wish, and even expect, to have the majority of structures functionally recovered in the shortest timeframe following an earthquake, economic realities dictate the setting of priorities. The need for multiple, tiered categories of structures reflects the consensus of subject matter experts that community-level recovery goals are reached by structure-level prioritization and performance as defined by the attainment of functional recovery within a short-, intermediate-, or long-term timeframe. These recovery timeframes are recognized in FEMA's National Disaster Recovery Framework (NDRF, 2016) and NIST's Community Resilience Planning Guide (CRPG, 2016).

The descriptions of each functional recovery category are based on input from subject matter experts (e.g., Abrahamson et al., 2021; Sattar et al, 2022), highlighting the priority of distinct functions and services in the recovery of the community and in minimizing negative social and economic consequences at the individual and community levels.

Intended community recovery goals. The provisions' framework aims to meet specific asset-level performance goals that, in turn, support community recovery objectives. Ensuring that a structure is able to support its basic intended function within an acceptable time after an earthquake event reduces the community's overall recovery time, helping it return to its pre-earthquake state as quickly as possible. Doing so is expected to reduce negative impacts at the individual level (e.g., physical, psychological and economic consequences) as well as at the community level (e.g., short- and long-term population displacement) among other negative consequences that may accrue to communities engaged in recovery.

Community considerations. The authority having jurisdiction (AHJ), design team, and individual owners might consider a variety of factors in their selection of Functional Recovery Category assignment and design criteria for a structure, including anticipated future use, characteristics or redundancies in the community's existing building stock, existing community characteristics or capacity, and local socioeconomic concerns. When adopting these provisions, policy makers, AHJs, and building owners can prioritize or de-prioritize different functions than indicated in Table A.5-1 when developing or assigning functional recovery categories in a manner that best suits the needs and characteristics of their community.

In general, it is recommended that community functions not be assigned to a lower category (e.g., from B to C), particularly if the intent is to achieve the more rapid recovery time of the higher category.

Table CA.5-1 is an example of a Functional Recovery Category Table, developed by a committee of experts in structural and seismic engineering, architecture, community recovery after disasters, economics, emergency management, and insurance. The table relates functional recovery categories and community services to structure types and IBC Chapter 3 occupancies for ease of use. It prioritizes the recovery of those functions shown to be the most critical for facilitating community recovery: emergency services, housing, schooling, and major jobs. While this table is intended to serve as a guidance to policy makers, AHJs, and building owners, it represents the generally observed relationship of how functions and services in a community facilitate post-disaster recovery, without consideration of cost impact of functional recovery design or pre-existing community capacity or other characteristics that could impact recovery. Consistent with the statement for Table A.5-1, policy makers, AHJs, and building owners can prioritize or de-prioritize different functions when developing or assigning functional recovery categories in a manner that best suits the needs and characteristics of their community.

Table CA.5-1 Sample Functional Recovery Category Assignments

Functional Recovery Category	Contribution of Structure to Community Resilience, with Example Structure Type(s) and/or Occupancy Classification(s)
A	<p>Structures with a basic intended function that the community relies on for immediate post-earthquake safety and survival, including but not limited to:</p> <ol style="list-style-type: none"> 1. Emergency response and communication services, including but not limited to: <ul style="list-style-type: none"> • Fire, rescue, ambulance and police stations • Emergency vehicle facilities • Aviation traffic control facilities • Structures having critical national defense functions • Water storage and pumping facilities supporting fire suppression 1. Emergency and acute healthcare services, including but not limited to: <ol style="list-style-type: none"> 1. Hospitals 2. Trauma centers 3. A subset of Group I facilities that provide such services (e.g., Group I-2 Condition 2) <ol style="list-style-type: none"> 1. Housing and care services for populations incapable of self-preservation, including but not limited to: <ul style="list-style-type: none"> • Skilled nursing, assisted living, and memory care facilities • Correctional care facilities • Residential care facilities for occupants not capable of self-preservation • A subset of Group I and R facilities that provide such services (e.g., Group I-1 Condition 2, Group I-2 Condition 1, Group I-3, Group R-3, and Group R-4 condition 2)

B	<p>Structures with a basic intended function that supports urgent and basic human survival needs and prevents the escalation of adverse disaster consequences, including but not limited to:</p> <ul style="list-style-type: none"> Emergency response services not included in Functional Recovery Category A, including but not limited to: <p>Designated earthquake shelters</p> <p>A subset of Group A and R facilities that provide such services (e.g., Groups A-3 and R-1)</p> <ul style="list-style-type: none"> Food and potable water services, both wholesale and retail, including but not limited to: Grocery stores and food distribution facilities A subset of Group B, F, M and S facilities that provide such services (e.g., Group S-2) Urgent, non-emergency, life-sustaining healthcare services, including but not limited to: Urgent care facilities Dialysis facilities Pharmacies A subset Groups B, F, and M facilities that provide such services <ul style="list-style-type: none"> Housing and sheltering services, including but not limited to: <ul style="list-style-type: none"> Residential or non-transient housing Transient lodging A subset of Group R facilities that provide such services (e.g., Group R-1, Group R-2, Group R-3 not classified in Functional Recovery Category A, Group R-4 Condition 1, and Group I-1 Condition 1) <ul style="list-style-type: none"> Urgent veterinary services, including but not limited to: <ul style="list-style-type: none"> Animal hospitals Urgent-care veterinary facilities A subset of Group B facilities that provide such services <ul style="list-style-type: none"> Vital utility services, including but not limited to: Public utilities, power-generation, and storage facilities Wastewater treatment facilities and potable water facilities Telecommunications hubs and switch facilities A subset of Groups B, F, H, and U facilities that provide such services (e.g., Group F-1) <ul style="list-style-type: none"> Services and employing organizations critical to national or multi-regional economic stability, including but not limited to: Government-designated facilities essential for sustaining national or multi-regional economic stability
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Functional Recovery Category	Contribution of Structure to Community Resilience, with Example Structure Type(s) and/or Occupancy Classification(s)
	<ul style="list-style-type: none"> Petroleum refining and petrochemical processing facilities Major manufacturing or distribution facilities that support critical national or multi-regional supply chains A subset of Groups B, F, H, and S facilities that provide such services (e.g., Groups F-1 and S-1)
C	<p>Structures with a basic intended function that supports recovery-focused self- and group-preservation, and sustains short- and long-term economic, educational, and governance activities and services, including but not limited to:</p> <ol style="list-style-type: none"> Community recovery services, including but not limited to: <ol style="list-style-type: none"> Local government, administrative, and civic facilities that support community operations A subset of Group B facilities that provide such services Child and adult daycare services, including but not limited to: <ol style="list-style-type: none"> Childcare centers A subset of Group E and I facilities that provide such services (e.g., Groups E and I-4) Outpatient healthcare services, including but not limited to: <ul style="list-style-type: none"> Clinics Medical offices A subset of Group B and M facilities that provide such services Primary and secondary education, including but not limited to: <p>Appendix A: K-12 schools</p> <p>Appendix B: A subset of Group E facilities that provide such services</p> Non-urgent veterinary services, including but not limited to: <ol style="list-style-type: none"> Veterinary clinics A subset of Group B and M facilities that provide such services Services and employing organizations critical to regional economic stability

Functional Recovery Category	Contribution of Structure to Community Resilience, with Example Structure Type(s) and/or Occupancy Classification(s)
D	<p>Structures with a basic intended function that enhances a community's general well-being and expedites the return to normalcy, including but not limited to:</p> <ol style="list-style-type: none"> 1. Post-secondary education, including but not limited to: <ul style="list-style-type: none"> • University lecture halls and classroom facilities • Higher-education laboratories and offices • A subset of Group B and E facilities that provide such services 1. Customary local government services, including but not limited to: <ul style="list-style-type: none"> ▪ Government structures not listed in Functional Recovery Category A, B, or C ▪ A subset of Group B facilities that provide such services 1. Services and employing organizations critical to local economic stability, including but not limited to: <ul style="list-style-type: none"> ☐ General commercial office buildings ☐ A subset of Group B facilities that provide such services
E	<p>Structures not assigned to FRC A, B, C, or D, including but not limited to:</p> <ol style="list-style-type: none"> 1. Non-critical community functions, including but not limited to: <ul style="list-style-type: none"> • Entertainment and recreational facilities such as theaters and bowling alleys • A subset of Group A facilities that provide such services (e.g., Groups A-1 and A-3)

Multiple functions. For structures that can be assigned to multiple functional recovery categories, the requirements for the most stringent Functional Recovery Category (i.e., highest, where A is higher than B) govern. An exception is given to structures where nonstructural components and services for distinct parts of the structure that can be assigned to different Functional Recovery Category are independent. For example, in a laboratory building, separate HVAC systems may be used for the laboratory and office parts of the building. Further, it needs to be shown that the damage to nonstructural components or systems in one portion of the building or structure will not affect the functionality of other portions of the building or structure. The functionality can be affected either directly (e.g., a damaged component toppling on a partition wall), or indirectly (e.g., a burst pipe flooding other portions of the building).

Where this is satisfied, the design requirements for certain nonstructural components only may be based on different Functional Recovery Categories. The structural requirements of Appendix A must still satisfy the most stringent Functional Recovery Category. The structural requirements are kept as such so that part of the building or structure does not undergo structural damage that would render the entire structure non-functional. This requirement applies even when parts of the structure are seismically separated, because it is unlikely that if seismically separated part of the structure would undergo significant damage, other parts of the structure would be allowed to remain occupiable.

The designer should be aware that satisfying this exception may require complex design of nonstructural systems and they should coordinate with other specialties early in the design process. Documentation of how the exception is satisfied is also important and the designer needs to show on drawings which elements correspond to which category according to A12.2 Item 4.

CA.5.1.1 Designation of Functional Recovery Category for Structures Assigned to Risk Category IV

In general, structures are assigned to Risk Categories based on the number of people whose lives would be endangered or whose welfare would be affected in the event of failure. This concept is also referred to as “Lives at Risk” and is described in commentary C1.5.1.

Risk Category IV has traditionally included structures the failure of which would inhibit the availability of essential community services necessary to cope with an emergency situation. As such, community functions served by these structures are necessary to recover in a short time frame after the earthquake. This time frame corresponds to Functional Recovery Categories A and B. The provisions of Appendix A impose structural, nonstructural, and other requirements that provide higher target performance of even Functional Recovery Category B structures at FRE_R hazard shaking relative to Risk Category IV at DE hazard shaking.

Structures assigned to Risk Category IV because they contain hazardous materials, can be assigned to a Functional Recovery Category lower than B based on their function. This is because the assignment to Risk Category IV is based on preventing release of the material rather than functional recovery needs. For this purpose, design requirements of Risk Category IV that are intended to decrease the likelihood of failure should provide desired performance. Designers should consider operational requirements of the facility to contain the hazardous materials. This consideration can further affect the selection of the functional recovery category. For example, where electrical power is needed for containment of the hazardous material, the designer may need to elevate the structure to Functional Recovery Category A to provide back-up power and higher reliability in electrical equipment to function after the earthquake. Alternatively, the designer can elect to implement these measures on a voluntary basis (if not required by other industry standards).

CA.5.1.2 Designation of Functional Recovery Category for Nonbuilding Structures

All nonbuilding structures are assigned to Functional Recovery Category based on Table A.5-1. However, since the design provisions for structural systems in Section A.9 currently only distinguish between expected performance of Functional Recovery Category B and E, all nonbuilding structures assigned to Functional Recovery Category higher than E are designed according to same requirements of Functional Recovery Category B. For these structures, the design to requirements of Functional Recovery Category B is only applicable for structural design as prescribed in Section A.9.

CA.5.1.3 Designation of Functional Recovery Category for Appurtenant Structures

Functional failure of any structure and/or pathway between that structure and the main structure that lies within the system boundary of the main structure could compromise the function of the

main structure. As such, both the supporting structure and any pathways between them shall be designed to the same criteria as the main structure. As an example, an electrical back-of-the-house (BOH) located outside of the main structure needs to be designed to the same performance criteria as the main structure. For electrical system, the pathway between the BOH and main structure could include cable trays with electrical conduits. These cable trays would be then again subjected to the same performance design criteria as the main structure.

Exception is given to supporting structures that do not affect the function of the main structure. For example, in some situations fencing or outside storage structures may not be needed for function of the main structure. In these cases, the appurtenant structure is assigned to the functional recovery category according to its own function and not the function of the main structure.

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CA.6 SEISMIC GROUND MOTION VALUES

CA.6.2 Functional Recovery Earthquake (FRE_R) Response Spectrum

The FRE_R spectral response acceleration parameters, S_{afr} , S_{DSfr} , and S_{D1fr} , come directly from the FRE_R response spectrum and are used in Section A.9 to determine the earthquake design loads. A factor of 2/3 should not be applied when determining response acceleration parameters from the FRE_R response spectrum. The FRE_R spectral response acceleration parameters, S_{afr} , S_{DSfr} , and S_{D1fr} , are archived in the USGS Seismic Design Geodatabase at gridded locations across United States regions of interest and provided online by the USGS Seismic Design Web Service, for user-specified

site location (latitude and longitude) and site class. The USGS web service spatially interpolates between the gridded values of these parameters based on site location (latitude and longitude).

CA.6.3.2 Site-Specific Functional Recovery Earthquake (FRE_R) Ground Motion Hazard Analysis

The general provisions of Section 21.2 for ground motion hazard analysis were judged to be acceptable for developing a site-specific probabilistic FRE_R response spectrum, particularly Section 21.2.1 and Section 21.2.3. Given the less stringent risk and reliability targets for functional recovery design, FRE_R ground motions are less rare than MCE_R ground motions. In ASCE 7-22, deterministic ground motions are used to limit probabilistically derived maximum considered ground motions to a reasonable upper bound given uncertainties regarding probabilistic contributions from very rare rupture scenarios. However, contributions from uncertain, very rare rupture scenarios are less significant in FRE_R ground motions. Additionally, in the 2026 NEHRP Provision, deterministic grounds were removed from the determination of MCE_R in favor of fully probabilistic ground motions. Therefore, deterministic ground motions were not deemed relevant for FRE_R. The provisions language defining probabilistic FRE_R ground motions in section A.6.3.2 is mostly adapted from Section 21.2.1 and Section 21.2.3. The provisions language defining FRE_R response parameters in section A.6.3.3 is mostly adapted from Section 21.4.

FRE_R spectral accelerations are defined as the median scalar intensity of the two-dimensional response spectrum in any orientation for a given period. Except for sites that are within a few kilometers of the rupture zone, the orientation of the maximum scalar spectral response of a given ground motion pair with respect to the strike of the fault is random (Watson-Lampre and Boore, 2007). Therefore, the likelihood that either direction of a given pair of orthogonal seismic force-resisting systems will be oriented in the maximum direction of shaking is low. Such a low probability is not consistent with the more moderate target probability levels associated with Functional Recovery performance objectives set in this Standard. For procedures of the nonlinear response history analysis, the structural performance assessment should not be significantly affected by the definition of spectral acceleration used, as long as each step of the process uses a consistent definition. However, the choice of median-component spectral acceleration values avoids unnecessary conservatism for the linear design procedure of section A.8 (Stewart et al., 2011). The median-component can be taken as either the RotD50 (Boore 2010) or the geometric mean of the response spectra, where applicable.

The lognormal standard deviation, or fragility shape, of functional recovery fragility was determined by quantifying the ATC-138 functional recovery of a suite of building models complying with Appendix A design requirements across a range of shaking intensities. The assessment of the functional recovery performance of each building model is probabilistic and explicitly considers uncertainties in ground motion recordings, structural modeling, construction quality, component damage, building repair time, and impeding factors that delay the start of repairs; uncertainties related to worker availability and allocation and material supply and demand are not considered. For each building model assessed, a continuous lognormal distribution was fit to the simulated functional recovery time outcomes using a Maximum Likelihood Estimation, resulting in fragility curves that relate the

probability of exceeding a particular recovery time to the spectral acceleration at the building's modeled fundamental period.

General results from the assessments showed that for building models with functional recovery times of less than 2 months (FRC A, B, and C compliant models), the lognormal standard deviations of the fragilities ranged between 0.5 and 0.7, with structural response and record-to-record variability making up a substantial portion of the uncertainty. For lognormal standard deviations in this range, FRE_R ground motion values only vary $\pm 5\%$. Therefore, to be consistent with the current provisions for life safety design in Chapter 21, a value of 0.6 is prescribed for all Functional Recovery Categories.

For models with longer recovery times (FRC D compliant models), performance uncertainty is much larger, with lognormal standard deviations up to 1.1, dominated by the highly uncertain nature of estimating long repair timelines. However, to maintain consistency with the other categories, the lognormal uncertainty value of 0.6 is maintained for FRC D. While FRC D designs are still expected to meet the 25% reliability target at the FRE_R shaking intensity level, the underrepresentation of uncertainty will result in unconservative ground motion values (up to 25% smaller) for buildings designed to FRC D, implying that FRC D designed buildings are less likely to satisfy the 10% in 50 years risk target.

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CA.7 GEOLOGIC HAZARDS

CA.7.1 Surface Fault Rupture

Structures designed for functional recovery cannot be located on sites where a known potential exists for surface fault rupture except as provided in the exception. This is similar to the prohibition of Section 11.8.1 for Seismic Design Categories E and F. It should be noted that the terminology "known potential" is used, similar to Section 11.8.1, in lieu of a specific probability of occurrence.

CA.7.2 Other Geologic Hazards

In general, geologic hazards must either not be present or, if present, be eliminated through mitigation in order to meet the functional recovery objective when using the prescriptive provisions (i.e., approaches other than Section A.17). This is necessary because the effect of geologic hazards on functional recovery time was not considered in the development of the prescriptive provisions (i.e., the prescriptive provisions were developed for sites without significant geologic hazards). Since geologic hazards in this section are assessed at the FRE_R level, some geologic hazards which may express at the design earthquake or MCE_R level may not at the FRE_R level. For example, liquefaction may not trigger for shaking up to and including the FRE_R level but will at the MCE_R level. A simple case where a geologic hazard may be present but can be essentially eliminated through mitigation is the condition of liquefiable shallow soils where over-excavation and back-filling with non-liquefiable material can be completed. Another is an initially unstable slope that is reinforced to prevent sliding with an appropriate factor of safety. Section C11.8 and C12.13.9 include additional, general commentary on geologic hazards.

Exception 1 recognizes that appropriately designed deep foundations in combination with a structural floor system capable of spanning between deep foundation elements is considered an appropriate design solution for the geologic hazards listed. The requirement that the structural floor system be able to span to the deep foundations refers to both the vertical (i.e., gravity) and lateral (i.e., diaphragm) spanning capability and may include structural elements other than solely a slab (e.g., slab spanning to beams which then span to the deep foundations). It should be noted that, for sites where the geologic hazards are significant (e.g., large lateral spreading displacements), such designs may be either impractical or cost prohibitive compared to other solutions (e.g., ground improvement).

Exception 2 is based on commentary in Section 12.13.9 which states that the differential settlement threshold limit used in Exception 2 is intended to be sufficiently small to permit post-earthquake use (e.g., allow for continued operation of doors). Including the effect of the geologic hazards in Section A.17 per Exception 3 may include approaches like adding differential settlement to peak and residual drift, including repair/replacement time of cracked slabs-on-grade, etc.

As discussed in Section CA.1, functional recovery for tsunami hazards is not included in this appendix.

CA.8 ANALYSIS AND DESIGN OF STRUCTURAL SYSTEMS FOR BUILDINGS

CA.8.1 General

For non-seismically isolated buildings, analysis and design for functional recovery is performed using either (a) all of Chapter 12 with the modifications listed in this section, or (b) all of Chapter 16 with the modifications listed in this section. However, analysis and design for functional recovery in

accordance with this section does not allow exemptions from requirements of previous chapters. Therefore, the provisions clarify that all of Chapter 12 or 16, without modifications, still need to be satisfied for the design earthquake or MCE_R hazards, respectively. In other words, two separate analyses will need to be performed (e.g., a linear analysis for the FRE_R in combination with a linear analysis for the design earthquake, a linear analysis for the FRE_R in combination with a nonlinear response history analysis for the MCE_R , or a nonlinear response history analysis for the FRE_R in combination with a nonlinear response history analysis for the MCE_R). Note that, when nonlinear response history analysis is used for the FRE_R , nonlinear response history analysis must also be used for the MCE_R . This is required in order to avoid under-design of certain components, particularly force-controlled actions, where functional recovery increases system strength (i.e., strength of deformation-controlled actions), the effect on force-controlled actions which may not be adequately captured by a linear analysis in accordance with Chapter 12 without modifications.

For seismically isolated buildings, unlike for non-seismically isolated buildings, a separate analysis in accordance with this section is not required. Instead, the analysis and design in accordance with Chapter 17 is modified.

CA.8.2 Seismic Importance Factor

For almost all cases, the importance factor is neglected since R_r is adjusted directly by Functional Recovery Category in this appendix instead of using R/I_e as Chapter 12 does by Risk Category. The exception is for out-of-plane forces, anchorage of structural walls, and transfer of design forces into diaphragms or other supporting structural elements where a margin against failure which changes with Functional Recovery Category is still required. That margin against failure is achieved by adopting the I_{pfr} value from Section A.10. The exception in this section clarifies that the additional 1.4 force amplification in Section 12.11 for steel elements of structural wall anchorage need not be applied simultaneously with I_{pfr} since I_{pfr} already addresses an appropriate margin against brittle failure, one that scales with Functional Recovery Category.

CA.8.3 Material-Specific Seismic Design and Detailing Requirements for Functional Recovery

CA.8.3.1 General The material-specific seismic design and detailing requirements of Chapter 14 continue to apply even for functional recovery. For certain systems, additional design and detailing requirements beyond those for safety-based designs are necessary to achieve functional recovery.

CA.8.3.2 Precast Concrete Precast concrete structures, particularly diaphragms and connections of structural members in such buildings, may be more susceptible to damage that affects functional recovery than cast-in-place construction. Additionally, less data to establish fragility functions for precast concrete gravity systems (e.g., gravity beam-to-column, beam-to-girder, double-T-to-beam, hollow-core plank-to-beam, etc. connections) exist. Therefore, additional requirements that increase the deformation capacity of precast gravity systems are required. These include:

- Requirement to have a minimum thickness cast-in-place topping slab with reinforcing bars. Very thin topping slabs and those only reinforced with welded wire reinforcement have been shown to suffer more damage in past earthquakes.
- Requirements for detailing of precast concrete structural members at bearing connections. The use of bearing pads allows for increased deformations to occur at bearing connections prior to spalling and cracking of precast concrete members when compared to high-friction bearing surfaces. Providing bearing plates and armored edges protects bearing connections from loss of effective bearing length when subjected to significant seismic displacements, by preventing local failures of supported and supporting members. The requirements for bearing connections are intended to promote more reliable behavior under significant seismic displacements in combination with the current requirements for precast concrete for safety-based design. Justification for the requirements are supported by the observations following recent New Zealand earthquakes (Corney et al. 2014; Henry et al. 2017) and subsequent experimental testing to investigate low damage solutions for precast concrete systems (Henry et al. 2021).

More background on these additional requirements as well as precast concrete gravity system fragilities can be found in RDC 2026.

The deformation compatibility requirements of Section A.8.4.10 also apply to precast concrete structures and are in addition to the requirements listed in this section. Similar/complementary deformation compatibility requirements for unbonded post-tensioned precast concrete special moment frames and unbonded post-tensioned precast concrete special shear walls are included in ACI 550.3 and ACI 550.7, respectively (ACI 2019; ACI 2022).

CA.8.3.3 Reinforced Masonry In the development of functional recovery seismic performance factors for special reinforced masonry shear walls (RDC 2026a), only archetypes and fragility curves for fully grouted walls were included. Therefore, a limitation requiring fully grouted walls for special reinforced masonry shear walls designed for functional recovery is included.

CA.8.4 Modifications to Chapter 12

CA.8.4.1 Spectral Acceleration Parameters

Refer to the commentary to Section A.6 for discussion of the FRE_R spectral acceleration parameters.

CA.8.4.2 Ground Motions

These requirements replace the MCE_R spectrum with FRE_R spectrum as the target spectrum but do not adjust the number of ground motions nor required method of record modification for the analysis in accordance with this appendix.

CA.8.4.3 Prohibited Vertical Structural Irregularities

These vertical structural irregularities are prohibited since they tend to concentrate story drift in the soft/weak stories, leading to increased structural and drift-sensitive nonstructural damage compared to a building without such irregularities. That concentration of damage is, by committee judgment, determined to inordinately increase the functional recovery time of such structures beyond that able to be captured by the current prescriptive provisions.

CA.8.4.4 Prohibited Analysis and Design Procedures

These analysis and design procedures are inherently different than the more standard Chapter 12 procedures. The provisions in this section were written to modify and have been benchmarked against the more standard Chapter 12 procedures. For example, see Section CA.8.4.6 and its references. Similar studies would be necessary to incorporate the analysis and design procedures referenced in this section. Until that time, these analysis and design procedures are not permitted for functional recovery, though they may be used in the unmodified Chapter 12 checks (i.e., life-safety checks).

CA.8.4.5 Design Coefficients, Factors, and Allowable Drift Limits

When re-performing a Chapter 12 analysis but for the FRE_R instead of the design earthquake, this section addresses how to replace R , the allowable drift limit, C_d and Ω_0 with appropriate parameters for functional recovery analysis and design.

Functional recovery story drifts should be computed under the same conditions (e.g., center of mass versus diaphragm edges depending on torsional irregularity, neglecting diaphragm deformation, etc.) as required in Chapter 12.

In this appendix, the functional recovery deflection amplification factor, C_{dfr} , is generally taken equal to R_{fr} . However, for steel damped special moment frames, the effect of the damping devices (i.e., fluid viscous dampers) on the system drift performance is not included explicitly. Instead, it is represented through a reduced C_{dfr} . The value of $R_{fr}/1.8$ comes from the numerical coefficient for damping reduction from Table 18.7-1 corresponding to approximately 30% effective damping (i.e., 25% effective damping from the fluid viscous dampers plus some inherent damping). In other words, C_{dfr} would equal R_{fr} for steel damped special moment frames, similar to all other systems in Table A.8-1, except that the reduction of displacement resulting from the fluid viscous dampers is “lumped” into the C_{dfr} value.

The redundancy factor is neglected for the FRE_R evaluation since redundancy is more related to redistribution of loads following significant strength loss, a less likely occurrence in functional recovery designs which target lower ductility demands. Note that setting the redundancy factor to unity in this section is not intended to imply that system redundancy is not helpful for functional recovery. Lack of redundancy may lead to decreased reliability as the presence of fewer components amplifies the effect of component-level damage on system-level performance.

Table A.8-1 is based on ATC-138 (FEMA, 2021) analyses of structural system-only archetype models following the requirements of Section A.8.4.6 (Blowes et al., 2026a, RDC 2025a, RDC 2025b, RDC 2025c; RDC 2025d; RDC 2026a; RDC 2026b; RDC 2026c; RDC 2026d; RDC 2026e; RDC 2026f). Germane to the analyses used to determine the functional recovery seismic performance factors is the concept of safety-critical structural damage. The definition of safety-critical structural damage originated in Blowes et al. 2026a and is a necessary concept in the analyses used to determine functional recovery seismic performance factors for all structural systems. Safety-critical structural damage is a cumulative building-level damage state where structural damage is substantial enough that the building must be shut down and repaired before occupants are able to return safely. When safety-critical structural damage does occur in the ATC-138 methodology, functional recovery times typically exceed 2 months, thus making structural-system-only functional recovery time calculated by ATC-138 somewhat binary (i.e., fairly short to address rapid or temporary repair solutions such as shoring of isolated damage or greater than 2 months to address significant repair). Impeding factors, many of which trigger when safety-critical structural damage occurs, contribute significantly to the somewhat binary results of functional recovery time in the ATC-138 methodology. Note that safety-critical structural damage can often be addressed between 2 and 6 months within the ATC-138 methodology. This is why a greater probability of exceedance is permitted for Functional Recovery Category D as discussed in Section CA.8.4.6. Severe but limited structural damage (e.g., damage to a single structural component) is often not enough to trigger a loss of function due to safety-critical structural damage in the ATC-138 methodology. Instead, this type of damage is often dealt with in the ATC-138 methodology using rapid or temporary repair solutions, such as shoring and barricading. The somewhat binary result for structural-system-only functional recovery time in the ATC-138 methodology considering safety-critical structural damage (i.e., either fairly short or greater than 2 months) is believed to be a best-estimate of recovery trajectories for the majority of buildings. While this assumption is present in the ATC-138 methodology, it is acknowledged that individual buildings after an actual earthquake which experience safety-critical structural damage may be able to recover sooner than 2 months depending on their particular circumstances (e.g., motivation of the owner to compress enabling work and impeding factors).

It should be noted that the more global parameters of strength and drift limits are used as controls on component-level damage which ultimately aggregates to safety-critical structural damage as described in the preceding paragraph. However, while story drift and component-level deformation correlate reasonably well for most systems and configurations, it is recognized that this varies by system. In general, component-level deformation can be both de-amplified and amplified compared to story drift. An example where component-level deformation is de-amplified compared to story drift is for wall chord rotation in reinforced concrete shear walls and brace deformation in buckling-restrained braced frames. Particularly in the upper stories of a multi-story building, the story drift is comprised of two significant components: (a) rigid-body rotation due to overturning in the stories below, and (b) within-story racking. The rigid-body rotation component does not transform into wall chord rotation or brace deformation. Thus, the relevant portion of the story drift ratio is only that due to within-story racking. An example where component-level deformation is amplified compared to story drift is in coupling beams in reinforced concrete shear walls or steel and concrete coupled composite plate shear walls. In these coupling beams, the geometry of the coupling beams,

particularly their length, can result in coupling beam hinge rotations which significantly exceed the story drift ratio as a result of axial elongation/shortening of the walls on either side of the coupling beam. This amplification for coupling beams depends not only on the coupling beam length, with shorter coupling beams experiencing a more pronounced effect, but also on the adjacent wall length, with longer adjacent walls having greater axial elongation/shortening at the wall ends that attach to the coupling beams (for the same story drift ratio). The amplification is therefore better correlated with the ratio of coupling beam length to adjacent wall lengths, though coupling beam length alone may be used for simplicity. See Figure CA.8-1. Another example of story drift amplification is gravity members in bays adjacent to shear walls and braced frames where overturning effects/rocking can result in large gravity member hinge rotations (especially for shorter bays) as compared to estimates based solely on story drift ratio. Because of these effects, the user should be attentive to conditions where component-level deformation demands are amplified relative to story drift. Nonlinear analysis is a suitable method of better quantifying component-level deformation demands.

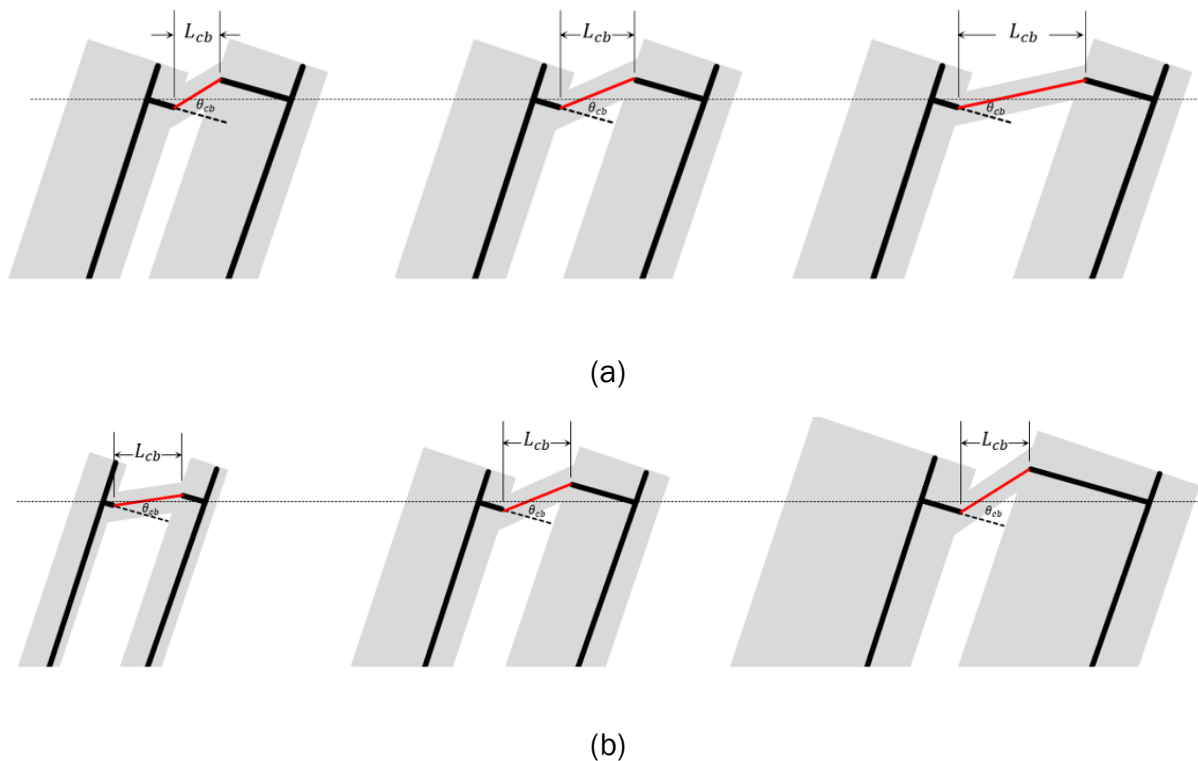


Figure CA.8-1. Effect on coupling beam rotation demand for a constant story drift ratio demand seen as (a) decrease in coupling beam rotation demand as coupling beam length, L_{cb} , increases if adjacent wall length is held constant, and (b) increase in coupling beam rotation demand as adjacent wall length increases if coupling beam length is held constant.

Special reinforced masonry shear walls are limited in structural height simply because the archetype space did not include taller structures, not necessarily because taller special reinforced masonry shear wall buildings cannot be designed for functional recovery (RDC 2026a). Additionally, a footnote to Table A.8-1 indicates that, for special reinforced masonry shear walls where all wall piers are flexure governed, more generous values of R_{fr} are permitted. In quantifying the functional recovery

seismic performance factors for special reinforced masonry shear walls (RDC 2026a), inclusion of archetypes with shear-governed wall piers in the performance groups significantly decreased the R_{fr} values for the system, even when the significant additional overstrength expected of shear-governed wall piers designed in accordance with TMS 402 was included. It was therefore decided to provide more permissive values of R_{fr} in a footnote, subject to a narrower design space as permitted by Section A.8.4.6.

Functional recovery seismic performance factors for special reinforced masonry shear walls are provided for both configurations (i.e., bearing wall and building frame). A bearing wall condition (i.e., where the wall resists a greater portion of the building's gravity load) generally results in lower recommended values of R_{fr} and Δ_{afr} due to the more severe damage state to safety class mapping required by the ATC-138 methodology (FEMA, 2021). The study of special reinforced masonry shear walls in RDC (2026a) assumed a bearing wall condition and then conservatively applied the recommended R_{fr} and Δ_{afr} values to the building frame condition so that both conditions (matching Chapter 12) would be provided.

The value of $R_{\mu fr}$ for special reinforced masonry shear walls is less than 1.0 and substantially less than the value of R_{μ} calculated in Chapter 13. This may appear inconsistent with the basis of the R_{μ} factor, which generally results in reduction of peak floor acceleration compared to an elastically-responding structure. However, as discussed in Section CA.8.4.6, it is possible for $R_{\mu fr}$ to be less than 1.0 in certain cases. Specifically for special reinforced masonry shear walls, the value of $R_{\mu fr}$ is controlled by the ratio of the largest peak floor acceleration at any level (i.e., Section A.8.4.6 Item 2) rather than the ratio of peak floor acceleration averaged across all structural levels (i.e., Section A.8.4.6 Item 1). For special reinforced masonry shear walls designed for functional recovery, particularly those where all wall piers are not flexure-governed in accordance with the Table A.8-1 footnote, a low value of R_{fr} and Δ_{afr} are required. The low value of R_{fr} and Δ_{afr} for non-flexure-governed masonry wall piers along with the shear requirements of TMS 402 (i.e., overstrength exceeding 2.5) combine to produce essentially elastic designs, which do not limit floor accelerations like ductile structures do. Furthermore, these designs result in a strong “spike” in peak floor accelerations at the roof of the archetypes (i.e., a whip effect). This then requires a low value of $R_{\mu fr}$ to correct the magnitude of the seismic floor acceleration calculated in accordance with Section A.10.7, which is based on a floor acceleration shape with height that does not include a peak floor acceleration “spike” at the roof. Values of $R_{\mu fr}$ for special reinforced masonry shear walls determined only using the ratio of peak floor acceleration averaged across all structural levels (i.e., Section A.8.4.6 Item 1 and neglecting Item 2) would be greater than 1.0 but still less than the value of R_{μ} calculated in Chapter 13.

Values for R_{fr} and Δ_{afr} for light-frame (wood) walls sheathed with wood structural panels rated for shear resistance included in Table A.8-1 (RDC 2026f) are based on archetype models and structural responses generally consistent with baseline parameters of FEMA P-2139-2. It is recognized that further study of light-frame (wood) walls has occurred, most notably alternative backbones and hysteretic parameters were studied in FEMA P-2139-2 and in FEMA P-2343. The alternative backbone curves and parameters are believed to better represent typical construction while the baseline FEMA P-2139-2 curves are believed to better represent lower-bound construction (minimum

nail edge distances, standard tie-downs, etc.). Although the backbone curves and hysteretic parameters from FEMA P-2343 were not fully incorporated in the determination of R_{fr} and Δ_{afr} for light-frame (wood) walls, the baseline backbone curves from FEMA P-2139-2 were scaled up to the same peak strength and ultimate displacement capacity of the backbone curves recommended in FEMA P-2343. Thus, much of the conservatism of the baseline parameters of FEMA P-2139-2 has been removed in determination of R_{fr} and Δ_{afr} for light-frame (wood) walls.

For strength and stiffness, only lateral force-resisting system (LFRS) walls are considered in RDC (2026f). However, the nonstructural finishes on the LFRS walls (e.g. stucco or gypsum wall board) are included in determining their strength and stiffness (in addition to the designated sheathing). Non-LFRS walls are not included for strength and stiffness. Both LFRS and non-LFRS walls are included for checking acceptance criteria (i.e., damageability).

Functional recovery seismic performance factors for light-frame (wood) walls are provided for both configurations (i.e., bearing wall and building frame). A bearing wall condition (i.e., where the wall resists a greater portion of the building's gravity load) generally results in lower recommended values of R_{fr} and Δ_{afr} due to the more severe damage state to safety class mapping required by the ATC-138 methodology (FEMA, 2021). The study of light-frame (wood) walls in RDC (2026f) assumed a bearing wall condition and then conservatively applied the recommended R_{fr} and Δ_{afr} values to the building frame condition so that both conditions (matching Chapter 12) would be provided.

Steel special concentrically braced frames studied in RDC (2026b) only included heights up to approximately 140ft. However, a 140ft height limit was not imposed since (a) the tallest archetype studied generally performed better than the next tallest archetype studied, and (b) the difference between 140ft and 160ft, the latter being the height generally permitted by Chapter 12, was deemed sufficiently close.

Buckling-restrained braced frames are distinguished between those with non-moment versus moment-resisting beam-column connections. A similar distinction was made in past versions of Chapter 12 of this standard (i.e., ASCE 7-05), though Chapter 12 no longer currently makes that distinction. Blowes et al. 2026a and RDC 2026c demonstrate a difference in functional recovery performance between these two configurations, primarily due to the greater likelihood of residual drift in a buckling-restrained braced frame with non-moment-resisting beam-column connections. Where the larger R_{fr} and/or functional recovery allowable story drift limits are used for buckling-restrained braced frames with moment-resisting beam-column connections, all beam-column connections within the braced frame should be detailed to be moment-resisting (i.e., none should be simple/non-moment resisting) per AISC 341. Note that buckling-restrained braced frames with non-moment-resisting beam-column connections have very minimally different values of R_{fr} between Functional Recovery Categories A, B and C compared to D. This is explainable as seen in Blowes et al. (2026a) and RDC 2026c by the fact that the story drift response of this system tends to localize in one or a few stories. Story drift localization compresses/tightens the probability contours, resulting in a negligible difference in the recommended values for Functional Recovery Category A, B and C versus D.

A footnote to Table A.8-1 indicates that, for steel and concrete coupled composite plate shear walls (SpeedCore) where all coupling beams are equal to or longer than 7.5 ft (2.3 m), more generous values of R_{fr} are permitted. In quantifying the functional recovery seismic performance factors for SpeedCore (RDC 2025c), inclusion of archetypes with short coupling beams (e.g., 6 ft (1.8 m) in length) in the performance groups significantly decreased the R_{fr} values for the system. It was therefore decided to provide more permissive values of R_{fr} in a footnote, subject to a narrower design space as permitted by Section A.8.4.6, by placing restrictions on coupling beam length. Where the length of one or more coupling beams is less than 7.5 ft (2.3 m), the R_{fr} values in Table A.8-1 (without the footnote) apply. As discussed above, the dependence of the functional recovery seismic performance factors on coupling beam length is likely really a dependence on the ratio of coupling beam length to adjacent wall lengths. However, for simplicity, a restriction on coupling beam length alone is used here, acknowledging that the adjacent wall lengths in actual designs also have an effect.

More generally, there are differing opinions on treatment of coupling beam damage and its impact on the likelihood of impeding functional recovery (e.g., triggering safety-critical structural damage). Some argue that even significant damage in coupling beams, while potentially visually concerning, would not necessarily prevent the continued function of the building while coupling beam repair is taking place, so long as the walls themselves are sufficiently undamaged to retain structural system strength and stiffness. Others assert that damage to coupling beams such as bar buckling in reinforced concrete or plate buckling in composite coupling beams, especially since coupling beams are often in areas quite visible to building occupants (e.g., over elevator lobby doorways), would disrupt continued building function. The assignment of the degree to which coupling beam damage impedes function has a significant effect on the R_{fr} and Δ_{affr} values for reinforced concrete ductile coupled walls and steel and concrete coupled composite plate shear walls. In quantifying R_{fr} and Δ_{affr} for these systems, the relationship between coupling beam damage and building function (i.e., assignment of safety class to damage state in the FEMA P-58 functional recovery time methodology (FEMA 2021)) was assumed to be on the slightly more permissive end of this debate so as not to have the R_{fr} and Δ_{affr} values overly influenced by an assumption on which there continue to be differing opinions in the profession. More information on the assumptions made can be found in the reports for reinforced concrete ductile coupled walls and steel and concrete coupled composite plate shear walls (RDC 2025a, RDC 2025c). Users of such systems may consider especially identifying coupling beams within the functional recovery plan of Section A.15, where required.

The functional recovery seismic performance factors for steel special moment frames were developed based on fragility curves appropriate for reduced beam section connections (RDC 2025b). However, other prequalified beam-column connection types, notably proprietary connections such as SidePlate, DuraFuse and Simpson Yield Link, are permitted by AISC 341. While full qualification analyses were not performed for steel special moment frames using every beam-column connection type, the fragility curves for these other connection types, when available, were reviewed. In general, median capacities of critical damage states in other connection types were found to be equivalent, and sometimes greater than, those for a reduced beam section. Thus, results from analysis

assuming a reduced beam section connection is considered appropriate, if not conservative, for other potential connection types.

In the qualification of certain systems for Table A.8-1, the $R_{\mu fr}$ value calculated in accordance with the exception of Section A.10.5 was not sufficient to meet the two ratio limits of Section A.8.4.6. Therefore, a prescribed $R_{\mu fr}$ value for the system in Table A.8-1, in accordance with Section A.8.4.6, was required. See commentary Section CA.8.4.6. System-specific values for $R_{\mu fr}$ were generally rounded to the nearest 0.05. Where a system-specific value of $R_{\mu fr}$ is not provided in Table A.8-1, the exception of Section A.10.5 can be used to calculate $R_{\mu fr}$.

It is recognized that Table A.8-1 does not include an exhaustive list of all structural systems in Table 12.2-1. Exception 1 provides a path for determining seismic performance factors for systems not currently listed.

Exception 2 recognizes that, for light-frame (wood) shear wall buildings, additional stiffness is often present in the non-designated system (i.e., the light-frame (wood) walls not specifically designated as part of the lateral force-resisting system but nonetheless present). Unlike most other seismic force-resisting systems, the stiffness contribution of the non-designated system is significant compared to the stiffness of the designated system alone. Such non-designated walls may be sheathed similarly to the designated walls (e.g., exterior walls sheathed with plywood) or be sheathed with materials not typically considered structural but which still provide stiffness and strength (e.g., interior walls with gypsum board both sides), especially in buildings with high non-designated system wall density (e.g., multi-unit residential). Such non-designated walls must extend from diaphragm-to-diaphragm to be considered within this exception. For example, non-designated walls which stop above a dropped ceiling and are only braced up to the diaphragm above are not permitted to be included. The additional stiffness of these non-designated walls is allowed to be considered when checking against the functional recovery allowable story drift ratio per this exception. However, their strength cannot be included when determining base shear resistance. When the additional stiffness of a non-designated wall is used, the wall must be indicated in the construction documents for the awareness of future renovations/alterations.

CA.8.4.6 Quantification of Functional Recovery Seismic Performance Factors

This section addresses how seismic performance factors may be determined for structural systems not currently listed in Table A.8-1. It represents the functional recovery parallel to the FEMA P-695 (FEMA, 2009) method for determination of seismic performance factors for collapse performance of a structural system. As such, it borrows similar nomenclature (e.g., archetypes, performance groups, etc.) from FEMA P-695. The scope of archetypes, models, analysis, etc. for quantification of functional recovery seismic performance factors for a system not currently listed in Table A.8-1 would likely need to be similar to that in Blowes et al. (2026a). This section permits quantification of functional recovery seismic performance factors for a subset, rather than the entirety, of the full design space. For example, functional recovery seismic performance factors may be determined for a system in Table 12.2-1 but with further restrictions on allowable height by Seismic Design

Category. Another example is shown in Table A.8-1 for BRBFs with and without moment-resisting beam-column connections.

Since this section addresses quantification of functional recovery seismic performance factors for use in structural design, it only considers the structural system (i.e., gravity and seismic force-resisting systems). Nonstructural systems are not considered here and are instead addressed in Section A.10 and A.11. However, for determination of engineering demand parameters in Section A.17, it may be acceptable to include the strength and stiffness of the gravity structural system and nonstructural systems, where such strength and stiffness is indicated as necessary for functional recovery in the construction documents. For example, in the development of the seismic performance factors for light-frame (wood) walls in Table A.8-1, some contribution of walls not sheathed with wood structural panels (e.g., perimeter and interior walls with gypsum sheathing) was included.

Acceptable Procedures

When used on a project-specific basis, the explicit assessment procedure of Section A.17 permits selection of the functional recovery assessment method. However, the systems in Table A.8-1 were qualified using the FEMA P-58 Functional Recovery Module methodology developed in the ATC-138 project (FEMA, 2021; FEMA, 2025). The FEMA P-58 Functional Recovery methodology is therefore recommended for qualification of other structural systems in order to ensure a consistent approach across all structural systems.

Analysis Limitations

Explicit functional recovery assessment procedures require estimation of engineering demand parameters such as peak story drift ratio, peak floor acceleration, and residual story drift ratio. Estimation of engineering demand parameters using linear analysis becomes less accurate as the structural system ductility demand increases. Therefore, nonlinear response history analysis is generally required unless results of linear analyses are shown to match those of nonlinear analyses (e.g., for structural systems with low functional recovery response modification coefficients).

An exception in Section A.17 allows for the use of more simplified nonlinear analysis, at the discretion of the independent reviewer(s). This addresses the possibly onerous computational demand required to analyze an entire archetype design space for every iteration of functional recovery seismic performance factors (i.e., R_{fr} and Δ_{aff}), allowing for simplified analysis methods to be leveraged.

Simplified methods of structural response analysis, such as those outlined in FEMA P-58 Chapter 5 (FEMA, 2018), ATC-138 (FEMA, 2021) and Blowes et al. (2026a) incorporate concepts of linear response of multiple degree-of-freedom systems as well as adjustments for the expected level of nonlinear demand. However, a key commonality to these simplified analysis procedures is that they are based on large datasets of explicit linear and nonlinear response history analyses in order to include trends across a range of building configurations and seismic intensities for a given structural

system. For such simplified nonlinear methods to be utilized, a substantial set of nonlinear structural analyses must be run for the specific structural system being evaluated, and those nonlinear response data must be utilized to calibrate and validate the simplified nonlinear analysis procedure. This calibration and validation process must (a) demonstrate that the mean structural responses are accurately predicted, (b) include appropriate treatment of the record-to-record variabilities in responses, and (c) address the joint correlations between structural responses. This is because mean demands, variabilities, and response correlations all have substantial impact on the functional recovery analysis results and therefore the resulting functional recovery design requirements that come out of such analyses. This simplified nonlinear method approach is allowable, provided that the documentation and implementation of the method is reviewed and approved by the independent reviewer(s).

Acceptance Criteria

The probability of exceeding the target recovery time is set to 10% for Functional Recovery Category A, B and C. At first, this seems contradictory to the 25% probability of exceeding the functional recovery time specified in Section A.1. However, the probability calculated in this section is only for structural implications on functional recovery time. In other words, no impact of nonstructural components on functional recovery time is included. This can be thought of as approximately “partitioning” the 25% probability into 10% for structural and 15% for nonstructural. For Functional Recovery Category D, targeting a 10% probability of exceeding six months considering only structural impacts does not lead to consistent outcomes in terms of achieving the 25% probability of exceeding six months considering structural and nonstructural contributions. Instead, a 15% probability of exceeding two months considering only structural impacts was found to produce better full-building outcomes at six months and is thus what is required in this section (Blowes et al., 2026a). Similar to FEMA P-695 (FEMA, 2009), the average probability within each performance group is the primary metric for evaluating suitability of the functional recovery seismic performance factors. But, also similar to FEMA P-695, the probability for an individual archetype is also limited, though somewhat more generously than for the average.

Regardless of the explicit assessment procedure analysis results, the R_{fr} and Δ_{afr} for Functional Recovery Categories A, B and C and Functional Recovery Category D are capped at the values approximately corresponding to Risk Category IV and III, respectively. These limits are required in order to (a) recognize the historical precedent for improved structural performance set by Risk Category III and IV, (b) protect drift-sensitive nonstructural elements, components, equipment and distribution systems that are not explicitly designed for the actual drift demand (e.g., partition walls), and (c) provide a practical check against relying too heavily on the analytical results. The slightly relaxed drift limit caps of 1.25% and 1.75% reflect an approximately 20% more permissive limit than the 1% and 1.5% for Risk Category IV and III, respectively, in Chapter 12. This keeps with precedent set elsewhere in the standard where more rigorous analysis is still limited to no more than a 20% deviation from standard acceptance criteria (e.g., site-specific spectra cannot fall below 80% of USGS spectra). While the limiting values here are adapted from the inferred intent and precedent set by Risk Category III and IV, these limits are not intended to achieve exactly comparable designs as for Risk Category III and IV in Chapter 12. Different designs are expected primarily because (a) the

FRE_R and design earthquake produce different ground motions, and (b) C_{dfr} equals R_{fr} for most systems in this appendix whereas C_d is less than R for most systems in Chapter 12. To ensure that these allowable judgement-based drift limits still sufficiently protect drift-sensitive nonstructural components, full analyses were conducted as part of the original study (Blowes et al. 2026b). Steel moment frames were the only structural system for which the judgement-based drift limits controlled the design, so these were reconfirmed to meet the target functional recovery times using the 1.25% and 1.75% judgement-based drift limits.

Ground Motion Parameters

Similar to FEMA P-695 (FEMA, 2009), a ground motion spectrum representing the maximum for a given Seismic Design Category must be defined to provide a consistent basis for qualifying new structural systems. In FEMA P-695, these are referred to as, for example, the Seismic Design Category D_{max} and C_{max} spectra which are the highest intensities required for FEMA P-695 analysis by Seismic Design Category.

For qualifying new structural systems for use Chapter 12 in accordance with FEMA P-695 (FEMA, 2009), the current D_{max} and C_{max} (also known as D_{min}) spectra are defined in Section 12.2.1.1 with respect to the Design Earthquake (DE) and Risk-Targeted Maximum Considered Earthquake (MCE_R) levels. However, for qualifying a structural system for use in this appendix in accordance with this section, the Risk-Targeted Functional Recovery Earthquake, FRE_R , is used. Given variations in the shape of the hazard curve for sites across the U.S., there is not a one-to-one relationship between the FRE_R and the DE. The relationship depends on, for example, site, period, and soil conditions. Therefore, to determine a general $D_{max,fr}$ and $C_{max,fr}$ condition for the FRE_R , the ratio of FRE_R to DE was quantified for 160 sites (USGS, 2024) across the U.S. The FRE_R -to-DE ratio used to establish the general $D_{max,fr}$ and $C_{max,fr}$ spectra was taken as the average ratio for all sites that fell within 10% of the current D_{max} and C_{max} spectra, respectively. For this purpose, S_{DS} and S_{D1} were taken as 1.5g and 1.0g, respectively, for the D_{max} spectrum and 0.5g and 0.2g, respectively, for the C_{max} spectrum. In summary, the general $D_{max,fr}$ and $C_{max,fr}$ spectra are the product of the FRE_R -to-DE ratio and the D_{max} and C_{max} spectra, respectively, for default site conditions in accordance with Section 11.4.2.1 and the applicable periods.

It should be noted that functional recovery performance is dependent on shaking intensity even when that shaking intensity is used for both design and evaluation. As shaking intensity increases, functional recovery times increase. Therefore, functional recovery seismic performance factors (i.e., R_{fr} and Δ_{afr}) determined for a D_{max} spectrum will produce conservative designs when used in Seismic Design Category B or C. However, in Seismic Design Category E and F, the functional recovery times may exceed the target times.

Archetype Models

A single archetype is a prototypical representation of a seismic-force resisting system. An entire archetype design space is a collection of archetypes intended to represent the overall range of permissible configurations. The scope of an adequate archetype design space should allow for a

reasonable range of parameters to be investigated that are feasible in design and construction, rather than every conceivable configuration.

Performance groups partition (or bin) the archetype design space into archetypes sharing common features or behavioral characteristics. For example, a common parameter for performance group definition is building height, where archetypes could be separated into low-, mid-, or high-rise which indirectly separates the archetypes according to fundamental period range as well. Chapter 4 of FEMA P-695 (FEMA, 2009) and Chapter 7 of NIST GCR 10-917-08 (NIST, 2010) provides additional guidance on developing an archetype design space and defining performance groups.

Peak Floor Accelerations

While this section primarily serves as a procedure for quantifying functional recovery seismic performance factors for the structural system, limits on peak floor accelerations are also required. This is because, as discussed under “Acceptance Criteria” above, the structural system probability target (e.g., 10%) is predicated on a “remaining” probability for the rest of the building (i.e., nonstructural systems). If a structural system achieves an acceptable probability but produces peak floor accelerations which dramatically exceed the seismic floor accelerations predicted by Section A.10, the full building probability will exceed the 25% target. This is because the nonstructural system contribution, particularly from acceleration-sensitive nonstructural components, will be larger than assumed in the development of the Section A.10 provisions (e.g., which established the value of I_{pfr}).

In this section, two checks are required. The first is an average check for the full profile up the height of the median analysis estimate of peak floor acceleration against the simplified estimate of peak floor acceleration from Section A.10. The second is a check of maxima, which may occur at different structural levels for the peak floor acceleration from analysis versus that from the simplified estimate. Similar to the criteria for structural system probability of exceeding a target functional recovery time, the peak floor acceleration ratios are checked as a mean over archetypes within a performance group. For the maximum check, a relaxed limit of 1.2 instead of 1.0 is permitted to avoid a slightly high peak floor acceleration at a single structural level from driving the results for the entire structure.

System-specific $R_{\mu fr}$ values are provided in Table A.8-1 when required by Section A.8.4.6. The system-specific $R_{\mu fr}$ values determined following Section A.8.4.6 may represent several effects, including (a) structure ductility reduction as normally implied by R_μ in Chapter 13, and (b) adjustment from the simplified estimate of seismic floor acceleration determined in Section A.10 to a more detailed prediction of peak floor acceleration from models of the specific structural system. Even where the standard R_μ equation in Chapter 13 may adequately predict the effect of reduced structure ductility for functional recovery (when R_{fr} and Ω_{Ofr} are substituted in that equation), a system-specific $R_{\mu fr}$ value may still be required as a result of the latter effect. Using $R_{\mu fr}$ in this appendix in this way was deemed preferable to introducing another separate factor in Section A.10 which was structural system specific. Because $R_{\mu fr}$ accounts for more than just reduced structure ductility for functional recovery, it is technically possible for $R_{\mu fr}$ to take on values less than 1.0.

Independent Review of Seismic Performance Factors

The independent review is similar to the project-specific review of Section A.17, but with the additional aspects that apply to quantification of seismic performance factors for a system in general, as opposed to a specific building at a specific site. Since the analyses in this section do not require assignment of basic intended function nor do they include nonstructural systems, the review scope in Section A.17 referencing these items does not apply. An additional requirement is the review of the archetype design space, and that it covers the range of cases in which the structural system may be used. The requirement that at least one independent reviewer has experience with the behavior of the seismic-force resisting system of interest, or similar systems in the case of reviewing a novel structural system, is necessary so that they are able to assess whether the archetype design space adequately investigates design parameters that can affect structural response, and possibly control, functional recovery seismic performance factors.

CA.8.4.7 Equivalent Lateral Force Procedure Lower Bound C_s

The lower bound C_s equation historically originated to capture near-field effects in the longer periods of the design response spectrum. In its development, $0.5S_1$ was calibrated back to 1997 UBC near-fault factors in the velocity domain. Since there is no rational way to modify the equation to represent the different characteristics of the FRE_R compared to the design earthquake, the equation is simply neglected for this appendix. Note that the equation would still apply in the unmodified Chapter 12 checks.

CA.8.4.8 Alternate Design Provisions for Diaphragms, including Chords and Collectors

These requirements place limits on the R_s factor for the horizontal lateral force-resisting system (i.e., diaphragms, chords and collectors) to ensure they do not exceed R_r of the vertical lateral force-resisting system. Without this requirement, the diaphragm could be designed for a lower force than the vertical lateral force-resisting system, thus concentrating yielding/damage in the diaphragm instead of in the vertical lateral force-resisting system. Significant damage in the diaphragm instead of the vertical lateral force-resisting system would result in different structural system performance than is (a) intended by the requirements in the alternate design provisions for diaphragms, and (b) studied with respect to its impact on functional recovery time. For example, diaphragms designed for lower forces than the vertical lateral force-resisting system are expected to be damaged sooner, and likely to a greater extent, than is permitted for the vertical lateral force-resisting to achieve the target recovery times.

CA.8.4.9 Diaphragm Design Provisions for One-Story Structures with Flexible Diaphragms and Rigid Vertical Elements

This section addresses diaphragm design for one-story structures with flexible diaphragms and rigid vertical elements, though it does also require limited checks outside the diaphragm itself (e.g., checks of the vertical lateral force-resisting system). Two design procedures are permitted in this section: (1) a version of the “standard” diaphragm design procedure of Section 12.10.1 modified for functional recovery in accordance with Section A.8.4.9.3, and (2) a version of the “alternative”

diaphragm design procedure of Section 12.10.3 modified for functional recovery in accordance with Section A.8.4.9.4. Either can be used; both need not be satisfied. The requirements in this section and the values in Table A.8-2 are based on analytical work in RDC 2026g and RDC 2026h in accordance with Section A.8.4.5. In general, the limitations that define a structure as having “flexible diaphragms and rigid vertical elements” are carried forward from Section 12.10.3. Where a seismic force-resisting system is included in Table A.8-1, it must also be checked for those requirements. The exception to Section A.8.4.9.1 permits a seismic force-resisting system not included in Table A.8-1 to be used so long as the seismic force-resisting system is essentially capacity-protected by the diaphragm. This is implemented by requiring the seismic force-resisting system to be designed for the overstrength loads from the diaphragm design, where the loads from the diaphragm design are determined in accordance with either Section A.8.4.9.3 or A.8.4.9.4. In this way, this exception produces a design where damage is expected to concentrate in the diaphragm rather than the seismic force-resisting system such that repair of the vertical elements of the seismic force-resisting system would not meaningfully contribute to functional recovery time.

Section A.8.4.9.2 requires two checks of functional recovery story drift: (1) a check of the drift at any point on the diaphragm including deformation from both the vertical elements of the seismic force-resisting system as well as the diaphragm, and (2) a check of the drift including only the deformation from the vertical elements of the seismic force-resisting system (i.e., neglecting diaphragm deformation). The first check is compared against a functional recovery allowable story drift specified in Table A.8-2 (i.e., an allowable drift that is dependent on the diaphragm type); the second check is compared against a functional recovery allowable story drift specified in Table A.8-1 (i.e., an allowable drift that is dependent on the seismic force-resisting system). Both checks are required in order to ensure that each of the diaphragm and the vertical elements of the seismic force-resisting system are not experiencing more damage than is permitted to achieve the target functional recovery time. Where a seismic force-resisting system is not listed in Table A.8-1, the exception to Section A.8.4.9.2 permits the second check to be performed using a conservatively low value of functional recovery allowable story drift based on committee judgment. In that exception, C_{dfr}/R_{fr} is taken as 1.0, allowing drifts to be calculated without the need for an individual value of C_{dfr} and R_{fr} .

CA.8.4.10 Deformation Compatibility

These requirements are nearly identical to those in Chapter 12. However, in this appendix, they are not restricted to only certain Seismic Design Categories but rather apply to design for functional recovery in any Seismic Design Category (e.g., Seismic Design Category B and C).

CA.8.4.11 Additional Foundation and Superstructure-to-Foundation Connection Design Forces

The intent of these requirements is to protect brittle actions in foundations for the FRE_R hazard. Significant foundation damage at the FRE_R hazard is expected to require extensive repairs, the time to commence such repairs potentially being further exacerbated by the fact that foundation damage may not be immediately detected. The exception permits several actions, which are deemed ductile and therefore not likely to result in significant damage requiring repair, to be designed for non-

overstrength loads. Soil actions and geotechnical limit states include, for example, bearing and passive pressure for shallow foundations and skin friction, end bearing, and lateral soil bearing for deep foundations. Flexure in foundations is also included under the exception although, notably, shear, torsion, and axial actions are not. Also included in the exception is vertical reinforcing crossing the foundation-superstructure interface in walls and columns. An example of this would be vertical reinforcing bar dowels between the foundation and a special reinforced concrete shear wall or special reinforced concrete moment frame column which are fully developed and thus can act in a ductile manner for shear friction and wall flexure.

Where an element experiences combined force actions, overstrength versus non-overstrength loads apply to each force action independently. For the example of a drilled shaft experiencing combined axial and flexural actions, within an axial-moment-moment (PMM) interaction check of the structural limit states of the drilled shaft, the axial actions would be taken as overstrength loads while the flexural actions, in accordance with the exception, need not consider overstrength loads.

It is noted that analysis of foundation models with overstrength forces applied may not converge since global overturning stability is not required to be satisfied for overstrength load combinations. As such, it is often necessary to apply non-overstrength loads in the model and then scale the force effects (e.g., shear) requiring to be designed for overstrength.

CA.8.5 Modifications to Chapter 16

CA.8.5.1 Ground Motions

Refer to the commentary to Section A.6 for discussion of the FRE_R ground motions.

CA.8.5.2 Unacceptable Response

These requirements clarify that unacceptable responses are not permitted for the FRE_R hazard regardless of Risk Category. Unacceptable responses are typically associated with collapse, extensive ductility demands on deformation-controlled actions, excessive force demands on force-controlled actions, or lack of convergence. All of these situations are unexpected for the FRE_R hazard which is significantly lower than the MCE_R used in a Chapter 16 analysis without modifications. The occurrence of an unacceptable response for the FRE_R hazard likely signifies a critical issue with the design or analytical model and is therefore prohibited.

CA.8.5.3 Minimum Design Strength

These requirements clarify that a minimum base shear strength is needed to achieve the functional recovery objective when using nonlinear response history analysis (unless the explicit assessment of Section A.17 is used). A minimum base shear strength is set in this section based on the design base shear from Section A.8 to ensure that a building designed for functional recovery using nonlinear response history analysis has at least a base shear strength equal to the minimum requirements for a building designed for functional recovery using the linear analysis procedures.

CA.8.5.4 Transient Story Drift

Drift limits for nonlinear response history analysis for the FRE_R are identical to those for linear design for the FRE_R . This differs from a Chapter 16 analysis without modifications which uses a larger drift limit than Chapter 12 without modifications because those chapters also differ in the ground motion level (i.e., design earthquake versus MCE_R , respectively).

CA.8.5.5 Residual Story Drift

The residual story drift ratio limit is selected based on committee judgment and referencing the FEMA P-58 fragility for an irreparability damage state for residual drift.

CA.8.5.6 Force-Controlled Actions

Since force-controlled actions will always be governed by the Chapter 16 analysis without modifications for the MCE_R , it is not necessary to also check them for the lower hazard of the FRE_R .

CA.8.5.7 Deformation-Controlled Actions

Use of Immediate Occupancy and Damage Control limit states from ASCE 41 for Functional Recovery Category A/B/C and D, respectively, is based on committee judgment. These deformation-controlled action limits are considered appropriate, if not conservative. However, it should be noted that detailed studies demonstrating that buildings conforming to these component-level limits achieve acceptable probabilities of structural system-only functional recovery time (as required by Section A.8.4.6) have not been performed. Particularly, the deformation-controlled action checks here are assessed based on mean (i.e., approximately 50th percentile) results from nonlinear response history analysis whereas it is known that consideration of variability (i.e., more extreme portions of the demand and capacity distributions) is required to achieve the low probabilities of exceeding a target recovery time.

CA.8.5.8 Elements of the Gravity Force-Resisting System

These requirements are similar to those in Chapter 16. However, they are re-iterated here for emphasis on the importance of gravity system damage in preventing functional recovery.

CA.8.6 Modifications to Chapter 17

For seismically isolated buildings, unlike for non-seismically isolated buildings, a separate analysis in accordance with this section is not required. Instead, the analysis and design in accordance with Chapter 17 is modified by limiting the allowable drift limit at the MCE_R to 1.5%. Taken together with other requirements in Chapter 17 at the MCE_R (e.g., a response modification coefficient of 2 or less), the performance at the FRE_R is expected to be essentially elastic (i.e., effective response modification coefficient of 1.5 or less) and exhibit low story drifts (i.e., 1% or lower).

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CA.9 ANALYSIS AND DESIGN OF STRUCTURAL SYSTEMS FOR NONBUILDING STRUCTURES

CA.9.1 General

Nonbuilding structures are analyzed and designed for life safety using Chapter 15 and are analyzed and designed for functional recovery using Appendix A. Nonbuilding structures are assigned to Functional Recovery Category B or E according to Section A.5. Insufficient data exists for nonbuilding structures to allow provisions to be developed that can provide designs meeting the longer target recovery times allowed under Functional Recovery Categories C and D or the immediate return to functionality required for Functional Recovery Category A. It is the judgement of the committee that the provisions of this section will provide a robust design of nonbuilding structures meeting the target recovery time expected for Functional Recovery Category B.

CA.9.2 Seismic Importance Factor

The seismic importance factor does not apply because the Functional Recovery Response Modification Factor, R_{fr} , is adjusted directly for use in Functional Recovery Category B in this section instead of using R/I_e as is done in Chapters 12 and 15.

CA.9.3 Modifications to Chapters 12 and 15

To obtain functional recovery, certain provisions of Chapter 12 and Chapter 15 require modification.

CA.9.3.1 Spectral Acceleration Parameters

This section simply replaces the MCE_R acceleration parameters with the corresponding FRE_R acceleration parameters.

CA.9.3.2 Ground Motions

This section simply replaces the MCE_R spectrum with the FRE_R spectrum as the target response spectrum for the linear response history analysis procedure of Section 12.9.2 but does not adjust number of ground motions nor required method of record modification.

CA.9.3.3 Prohibited Vertical Structural Irregularities

Vertical Structural Irregularity Types 1b and 4b of Table 12.3-2 are not permitted because they tend to concentrate story drift in the soft/weak stories, leading to increased structural and drift-sensitive nonstructural damage compared to a building without such irregularities. That concentration of damage is, by committee judgment, determined to inordinately increase the functional recovery time of such structures beyond that able to be captured by prescriptive provisions. However, an exception is provided that allows these vertical structural irregularities if an analysis can demonstrate acceptable performance. This is particularly important for nonbuilding structures, as many industrial structures must have weak or soft stories as a result of their basic function and configuration.

Experience shows that the use of $R_{fr} = 1$ ensures elastic behavior in the weak story portion of the structure, resulting in acceptable performance. A detailed discussion of structural irregularities in nonbuilding structures and the concept of “story” in a nonbuilding structure can be found in C15.1.3 and Appendix 4.g of ASCE (2020).

CA.9.3.4 Design Coefficients, Factors and Drift Limits

This section addresses how to replace R , C_d and Ω_o with appropriate parameters for functional recovery analysis and design. The deflection amplification factor C_{dfr} is taken equal to R_{fr} , which reflects current research recommendations. The values of R_{fr} and Ω_{ofr} found in Table A.9-1 are based on engineering judgment and experience. Most nonbuilding structures in the facilities under consideration for Functional Recovery Category B have historically been designed as Risk Category III structures. The performance of these structures globally, when designed to modern codes (1988 UBC and later), has been excellent as documented in Soules, Bachman, and Silva (2016), Wozniak (1980), Harper and Wozniak (1995), and Nishimura (2015). However, the Provisions Update Committee believes that the performance of Risk Category IV structures is well documented and a better justification for the basis of design. Therefore, the values of the functional recovery response modification factors, R_{fr} , in Table A.9-1 have been set approximately equal to $R/I_e = R/1.5$. The values of the functional recovery overstrength factors, Ω_{ofr} is based on the definition of overstrength from the 1999 SEAOC Blue Book. A lower overstrength factor from those listed in Chapter 15 is warranted because as the design seismic force goes up, the ratio of the elastic seismic force to the design seismic force (definition of R_o) decreases. Overstrength was defined as $1.1R_o$, where R_o was an approximation for the system overstrength. The response modification factor was taken as the product of R_o and R_d , where R_d was an approximation of the system ductility. Assuming that R_o and R_d are approximately equal, the functional recovery overstrength factor for nonbuilding structures can be approximated as:

$$1.1 \sqrt{R_{fr}}$$

The values of Ω_{ofr} in Table A.9-1 are based on this approximation.

Similar to Chapter 15, no specific drift limits are specified for nonbuilding structures. Because many nonstructural components are drift sensitive, the allowable drift for a nonbuilding structure is dictated by the maximum drift that the nonstructural component can sustain and still remain functional. However, maximum drift limits based on Table 12.12-1 (for low R value systems) or Table A.8-1 (for high R value systems) are imposed as an upper bound for nonbuilding structures similar to buildings as a committee compromise. For Section A.9, high R value systems are designated as those with R values greater than 4.0 and low R value systems are designated as those with R values less than or equal to 4.0. In this first edition of Table A.8-1, not all high R value systems allowed for use by Chapter 15 are included. Experience has shown that the high R value systems listed in Table 15.4-1 are rarely if ever used for nonbuilding structures. High R value systems tend to deflect too much and are generally not cost effective. For the development of the Section A.9 requirements, the committee has relied on this industrial experience. Because the committee’s experience with high R

value systems in industrial applications is limited, the committee chose to defer to the requirements of Section A.8. While Section A.8 does not currently contain all systems, it is anticipated that the number of systems covered will expand in future editions.

The redundancy factor is neglected for FRE_R evaluation since redundancy is more related to redistribution of loads following significant strength loss, which is unlikely in functional recovery designs that target lower ductility demands.

CA.9.3.5 Deformation Compatibility

This section is simply emphasizing the importance of gravity system in functional recovery.

CA.9.3.6 Additional Foundation Design Forces

The goal of this section is to reduce damage and maintain functionality of the superstructure by limiting damage to the foundation during the functional recovery earthquake. See Section CA.8.4.11 for additional commentary on the design of foundations to provide functional recovery. However, there is one significant difference between the requirements of Sections A.8.4.11 and A.9.3.6. For buildings, Section A.8.4.11 requires both the foundation and superstructure-to-foundation connection be designed for the additional design forces of Section A.8.4.11. Section A.9.3.6 only requires the foundation to be designed for these additional forces. Anchorage of nonbuilding structures to foundations is governed by Section A.9.3.7. Section A.9.3.7 requires nonbuilding structure anchorage to act as a seismic fuse. To achieve this requirement, the anchors are prohibited from being designed for seismic load effects including overstrength in order to promote yielding and stretching of the anchor.

CA.9.3.7 Nonbuilding Structure Anchorage

These provisions are intended to have the anchors act as seismic fuses for all nonbuilding structures as is currently done for tanks and vessels in ASCE 7 Chapter 15. The requirements presented force the anchor rods to yield and stretch, thus moving damage from the structure to the anchor rods. These requirements have been used for tanks and vessels for many decades and proven to be successful. The structure will remain functional as seen after the 2010 Chilean earthquake (Soules, Bachman, and Silva 2016). In most cases washers can be added so the bolts can be retightened. These requirements may also result in the adoption of removable (and therefore replaceable) anchor bolts as used in Chile (Soules, Bachman, and Silva 2016).

CA.9.3.8 Additional Requirements for Tanks and Vessels

The added provisions in this section make mandatory several performance improving requirements that previously only applied to Risk Category IV tanks and vessels. Section A.9.3.8.1 simply makes adjustments to Chapter 15 requirements from the MCE_R to the FRE_R . Section A.9.3.8.2 repeats the Chapter 15 requirement the piping flexibility requirements for piping connections to tank and vessels. Section A.9.3.8.3 requires that all tanks and vessels be provided with the full theoretical seismic freeboard. This requirement minimizes or eliminates damage to a tank fixed roof or floating

roof. Section A.9.3.8.4 requires that the pedestal or skirt support of all vessels to remain elastic during the functional recovery earthquake. The Chapter 15 equivalent of Sections A.9.3.8.3 and A.9.3.8.4 previously only applied to Risk Category IV tanks and vessels.

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CA.10 ANALYSIS AND DESIGN OF NONSTRUCTURAL SYSTEMS

CA.10.1 General

All of Chapter 13 without modification needs to be satisfied for the design-basis earthquake hazard. Additionally, all of Chapter 13 with modifications based on this appendix needs to be satisfied for the FRE_R hazard. In other words, two separate analyses will need to be performed.

CA.10.2 Exemptions

The exemptions of this section are generally taken from the 2022 California Building Code (CBC) Section 1617A.

CA.10.3 Seismic Functional Recovery Component Importance Factor

The requirements of this section define a functional recovery component importance factor, I_{pfr} . It should be noted that, for the analysis and design in accordance with this appendix, I_{pfr} replaces (i.e., is not taken in combination with) the component importance factor, I_p , from Chapter 13. In

application in this section, I_{pfr} separates capacity from demand in a similar way that load and resistance factors separate capacity and demand in load and resistance factor design (LRFD). The value of I_{pfr} has been validated for full building functional recovery models, including consideration of multiple structural system types, in Blowes et al. (2026b). See Section CA.10.10 for further discussion. However, certain nonstructural components in the ATC-138 (FEMA, 2021b) database were excluded. Specifically, elevators, fire break partitions, and egress doors were excluded from consideration for calculating functional recovery time. At the time of the studies in Blowes et al. (2026b), they were excluded due to (a) concerns over the conservatism of their current fragility curves and repair time consequence functions, and (b) high sensitivity of functional recovery time on the assumptions made for these components. It is expected that improved characterization of these specific components will be the subject of on-going research.

While ATC-138 (FEMA, 2021b) uses the term fire break partitions and the International Building Code (IBC) uses the term fire barrier wall, the terms are generally synonymous and refer to fire-resistance-rated wall assemblies typically found around, for example, egress stairs.

Molina-Hutt and Zimmerman (2024) also provides a framework for a more approximate derivation of I_{pfr} . Note that since I_{pfr} applies to the FRE_R hazard and is based on Functional Recovery Category while I_p applies to the design earthquake hazard and is dependent on Risk Category, it is possible for the Chapter 13 forces without modifications in this appendix to control.

CA.10.4 Spectral Acceleration Parameters

No commentary proposed.

CA.10.5 Seismic Design Forces

The exception to this section provides a practical upper-bound on seismic design forces for nonstructural components designed for functional recovery. It is based on capping the seismic design forces for nonstructural components in this appendix to no more than Chapter 13 design forces for Risk Category IV structures (i.e., using the design earthquake (DE) and $I_p = 1.5$). Where the FRE_R is close or equal to the DE and when $I_{pfr} = 1.75$, this exception is expected to produce smaller forces than calculations neglecting this exception. In all other cases, this exception is unlikely to affect seismic design forces.

CA.10.6 Structure Ductility Reduction Factor

Where a value of $R_{\mu fr}$ is provided in Table A.8-1 or A.8-2, it is used. In all other cases, re-calculation of R_{μ} to establish an $R_{\mu fr}$ is required in accordance with the exception. See paragraph describing peak floor accelerations in Commentary Section CA.8.4.6 for more background on $R_{\mu fr}$. In the exception, I_e does not apply since it is neglected per Sections A.8 and A.9 for the structural system design. Furthermore, the lower limit on $R_{\mu fr}$ of 1.3 is removed since it is possible for reduced structural ductility for designs in accordance with this appendix to produce a value of $R_{\mu fr}$ lower than that for Chapter 12 designs, in some cases even below a value of 1.3.

CA.10.7 Piping Spanning Between Points of Lateral Restraint

Piping in structures assigned to Functional Recovery Category A and B and piping carrying hazardous substances in structures assigned to any Functional Recovery Category must be capable of spanning between points of lateral restraint (e.g., a pipe must span between brace locations). Where seismic floor accelerations are sufficiently low and the piping does not cross a seismic joint or span between floors/adjacent buildings, the piping may be deemed to comply with this section without further analysis and design beyond those of Chapter 13 without modifications. Where these conditions are not satisfied, the piping must be designed for the imposed forces and displacements explicitly. An example of a calculation demonstrating the spanning capability of piping can be found in FEMA P-2192-V1 Section 8.6 (FEMA, 2021a). Additionally, particularly where piping spans across a seismic joint, between adjacent buildings, or from floor-to-floor, specialty-designed flexible connections may be necessary. Note that meeting the deemed to comply criteria does not exempt the requirements of other sections in this appendix (e.g., the nonstructural system-specific requirements for mechanical, electrical, plumbing and fire protection components). The seismic floor acceleration thresholds in the provided table are based on FEMA P-58 database fragility curves and 2.5%, 5% and 7.5% probability of triggering damage which could impede function for Functional Recovery Category B, C and D, respectively. These target component-level probabilities were inferred from back-calculated component-level probabilities within models in Blowes et al. (2026b) for which the building-level 25% reliability of not exceeding the target functional recovery time was achieved. The methodology for determining the thresholds is described in Molina-Hutt and Zimmerman (2024) and involves convolving two lognormal distributions, one representing demand and the other capacity, until the stated target probabilities are satisfied. Note that the table is restricted to piping fragilities available in FEMA P-58. FEMA P-58 Volume 3 (FEMA, 2018) provides the background on the component data used in establishing the fragility curves.

It should be noted that reducing the probability of triggering damage which could impede function, as has been done for determining the values in the provided table, is especially necessary for achieving short to very short functional recovery times associated with Functional Recovery Categories A and B where repair/replacement (other than temporary) is not possible within the targeted functional recovery time goal. For longer functional recovery times, increased probability of triggering damage which could impede function can be offset by speedier repair/replacement. However, even for longer functional recovery times, reducing the probability of triggering damage which could impede function is important to avoid required repair of a significant number of components/systems where the repair sequencing could build up to longer than the functional recovery time goal.

CA.10.8 Seismic Floor Acceleration

Estimation of seismic floor acceleration from the nonstructural force equation is a new concept introduced in this appendix. Forces from the nonstructural equation are assumed to represent best-estimate peak floor accelerations. It is recognized that floor acceleration prediction is more difficult than other engineering demand parameters (e.g., story drift, story shears, etc.) even when using more advanced techniques like nonlinear response history analysis. However, the nonstructural

force equations are judged to provide the most robust, simplified method for estimation available in the Standard.

Modifications to the nonstructural force equation are made to determine peak floor acceleration. First, C_{AR}/R_{p0} is taken as 1.0 since this ratio represents the within-component/within-equipment acceleration adjustment (i.e., adjustment from peak floor acceleration to peak component/equipment acceleration). Second, the component importance factor is taken as 1.0 since the component importance factor is a design force amplifier, not an estimator of peak floor acceleration. Third, a revised R_μ is required to reflect reduced structural ductility as discussed in Section CA.10.6.

It should be noted that the seismic floor acceleration established in this section is related to, but not equal to, the force coefficient used to design nonstructural elements or components (e.g., equipment anchorage). That force coefficient is established using the Chapter 13 equations as modified by other sections in this appendix.

CA.10.9 Nonlinear Response History Analysis

These requirements permit direct calculation of floor accelerations for use in nonstructural design when nonlinear response history analysis is used for the FRE_R hazard.

CA.10.10 Seismic Relative Displacements

Seismic relative displacements are calculated for the FRE_R using the factors (e.g., functional recovery deflection amplification factor) from Sections A.8 and A.9. For use in this section when Chapter 13 references allowable story drift, the functional recovery allowable story drifts for the FRE_R in Section A.8 should be used, not the allowable story drifts for the design earthquake from Chapter 12. In the validation of I_{pfr} in Blowes et al. (2026b) referenced in Section CA.10.3, I_{pfr} was equally applied to acceleration- and displacement-sensitive nonstructural components. As such, it is necessary for it to be applied to seismic relative displacements in this section. In other words, just as I_{pfr} is necessary to separate force capacity and demand for acceleration-sensitive nonstructural components, it is also necessary to separate displacement capacity and demand for displacement-sensitive nonstructural components.

Addressing detailing of key areas such as at corners of interior and exterior wall systems is especially important for achieving functional recovery. See Section CA.10.11.4 for greater discussion.

Item 3 in this section permits the seismic relative displacements to be conservatively based on the maximum functional recovery allowable story drift permitted rather than that calculated by analysis when functional recovery allowable story drift limits apply (e.g., building structures). In cases where functional recovery allowable story drift limits do not exist (e.g., certain non-building structures designed in accordance with Section A.9), the analytically determined drifts and displacements for the FRE_R hazard must be used.

CA.10.11 Architectural Components

CA.10.11.1 Suspended Ceilings The requirements of this section are generally taken from the 2022 California Building Code (CBC) Section 1617A. The provisions applicable to all Functional Recovery Categories clarify the requirements for suspended ceiling design which is otherwise lacking in this standard and its referenced standards. Additional requirements specific to structures assigned to Functional Recovery Category A and B exist to further reduce the likelihood of ceiling repair following an FRe event.

Note that, for acoustical tile ceilings, these provisions do not ensure complete prevention of ceiling tiles falling out of their supporting grid. To achieve that, clipping or taping of individual ceiling tiles may be required. This is often undesirable or impractical for day-to-day operations and may lead to clips or tape being removed entirely, thus short-circuiting the seismic performance benefits. In conditions where it is needed to more robustly secure ceiling tiles, alternatives such as using Velcro fasteners have demonstrated good performance (Tiwari et al. 2024).

CA.10.11.2 Partitions

No commentary proposed.

CA.10.11.3 Adhered Veneer

Adhered veneer has been observed to be prone to damage during earthquake shaking due to in- and out-of-plane story drift and is thus restricted in Functional Recovery Category A and B along paths of egress. While this section does not explicitly address the deformation capacity of the veneer, adhered veneers are prone to separating from their substrate and falling. Falling veneer along paths of egress is a concern both for safety reasons and because it must be cleared/removed prior to regaining building function. Certain especially brittle adhered veneers include those of masonry, stone, ceramic or porcelain tile, and terra cotta. Adhered veneer in this section does not refer to stucco or lath and plaster. Exception 1 allows adhered veneer when story drifts are sufficiently low that the veneer is not expected to loosen/fall.

CA.10.11.4 Window Wall, Curtain Wall, and Storefront Systems Additional testing of envelope systems provides greater reliability that such systems can support the basic intended function of the structure. Additional testing may include establishing acceptance criteria for each element of the envelope system (e.g., backup framing, connections of the backup framing to the primary structure, caulking, glazing, waterproofing membrane, finish material, and thermal control components). It may be necessary to focus especially on key areas such as corners and at the interface between different systems (e.g., between curtain wall and storefront systems). For example, corners of exterior systems which must accommodate story drift in two simultaneous horizontal directions are often challenging to appropriately detail without significant damage. This is true both for systems which slide (e.g., via a slip detail at the top of the wall) because the perpendicular sliding details conflict at the corner as well as for racking systems (e.g., curtain wall) where racking displacements cause binding at the corner. Thus, attention to such details is important to achieve functional recovery.

CA.10.12 Mechanical, Electrical, Plumbing and Fire Protection Components

CA.10.12.1 Distribution Systems Building-level functional recovery analyses (Blowes et al. 2026b) demonstrate that essentially all components necessary to support basic intended function must have a sufficient margin against initiation of damage which affects function. For the distribution systems studied, this requires positive attachment and bracing and that these supports be designed for the combined effects of forces and seismic displacements.

This section requires that distribution systems be positively attached (i.e., cannot rely on friction) at all lateral supports. While it is not uncommon, particularly in rooftop conditions, for the gravity supports of distribution systems to simply rest on the roof, this provision clarifies that positive attachments (e.g., mechanical anchors) are required at lateral supports/restraints.

Three conditions are listed where flexible connections or flexible configurations of distribution systems must be provided, at a minimum. An example of a flexible configuration of a distribution system is illustrated in Figure C13.3-5. However, flexible connections or flexible configurations of distribution systems may be necessary at other locations to address differential seismic displacements and/or rotations. The three conditions include:

1. Vertical riser pipes which can experience significant relative displacements and/or rotations due to imposed story drift.
2. Pipes, large ducts, and non-small conduit crossing a seismic joint which can experience significant relative displacements and/or rotations due to differential seismic movement between the two sides of the seismic joint. The thresholds to distinguish “large” ducts and “non-small” conduits are consistent with thresholds used in Section A.10.2.
3. Pipes, large ducts, and non-small conduit spanning between a component or piece of equipment and a brace point attached to a different structural level than the component or piece of equipment which can experience significant relative displacements and/or rotations due to imposed story drift. This is especially concerning where the clear length between the component or piece of equipment and the brace point attached to the different structural level is small (i.e., concentration of the story drift over a shorter length).

CA.10.12.2 Fire Sprinkler Drops Flexible fire sprinkler drops significantly reduce the potential for sprinkler and surrounding ceiling damage and have been observed to be a common failure point in recovery reconnaissance in past earthquakes.

CA.10.12.3 Elevators The requirements of this section are generally taken from the 2022 California Building Code (CBC) Section 1617A. The provisions applicable to all Functional Recovery Categories clarify the requirements for design of elevators and their associated components, especially guiderails, which is otherwise lacking in this standard and its referenced standards. Additional requirements specific to structures assigned to Functional Recovery Category A and B exist to expedite the safe return to function of elevators following an earthquake event, even if that return to function is achieved with the elevator operating at a reduced travel speed. Note that the deflection

limits given in the table in this section are intended to maintain a consistent factor of safety against disengagement of retainer plates from the guide rails during an earthquake.

REFERENCES

CCR (California Code of Regulations). 2022. *California Building Code*, Sacramento, CA: CCR.

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Molina Hutt, C. and Zimmerman, R.B. 2024. “Toward Prescriptive Seismic Design Provisions for Functional Recovery Performance: Nonstructural Systems.” In *Proceedings of the 18th World Conference on Earthquake Engineering*, Milan, Italy.

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CA.11 SEISMIC CERTIFICATION REQUIREMENTS FOR EQUIPMENT AND COMPONENTS

CA.11.1 General

Seismic certification is critical to ensuring the operation of a component or equipment after a seismic event. Seismic certification may occur for the entirety of the equipment or, where a piece of equipment is an assembly of multiple components, only for the non-rugged components within that assembly. Note that, unlike the requirements of Chapter 13, a factor, I_{qualfr} , is included when establishing the seismic certification level. This is necessary to ensure a low enough probability of component or equipment failure across Functional Recovery Categories considering the uncertainty

in peak floor/equipment acceleration demand and the uncertainty in the equipment capacity, particularly the equipment capacity beyond the certification level.

Exceptions 1 through 9 are taken from the California Department of Health Care Access and Information (HCAI) requirements in the 2022 California Building Code.

Exception 10 is based on the same procedure (Molina-Hutt and Zimmerman, 2024) and target probabilities (i.e., 5% and 7.5% for Functional Recovery Category C and D, respectively) referenced in Section CA.10.7. This exception acknowledges that, when floor accelerations are sufficiently low, standard equipment may have a high enough probability of functioning without further seismic certification. Standard equipment is assumed to be represented by the fragility curves of FEMA P-58 (FEMA, 2018). FEMA P-58 Volume 3 (FEMA, 2018) provides the background on the component data used in establishing the fragility curves. The limit for “Other Mechanical Equipment” and “Other Electrical Equipment” is necessary due to the fact that FEMA P-58 does not contain fragility curves for all components and equipment used in a structure. These limits have simply been taken as the lowest values of the seismic certification thresholds for known equipment in each category. No limit for “Other Interior Elements” is provided due to the wide variety of other possible interior elements and the lack of fragilities available in FEMA P-58 to represent the breadth of those possible elements. Exception 10 does not apply to structures assigned to Functional Recovery Category A and B given the critical nature of these structures. In other words, for components and equipment in structures assigned to Functional Recovery Category A and B, seismic certification is required unless they meet one of the other exceptions.

Exception 11 permits waiving of seismic certification of equipment in structures assigned to Functional Recovery Category C and D where it can be shown by the explicit assessment procedure of Section A.17 that floor accelerations are sufficiently low such that non-seismically certified equipment will still be able to function.

Note that it is possible that certain pieces of equipment would be required to be seismically certified in accordance with Chapter 13 but exempt from seismic certification in this appendix. In those cases, the Chapter 13 requirement would govern and seismic certification would still be required per the requirements in Chapter 13 without the modifications of this appendix.

CA.11.2 Seismic Certification Methods

The three seismic certification methods permitted in this section match those permitted by Chapter 13. Certification by shake-table testing is considered highly preferable where possible. The requirement that active parts or energized components be certified exclusively based on shake-table testing is also required by Chapter 13 but is reiterated here given its importance. While performing shake-table testing with the equipment energized/operating throughout the test is valuable, the vast majority of available seismically certified equipment has not been tested this way to date nor do common shake table acceptance criteria (e.g., ICC-ES AC156) currently require equipment to be energized/operating throughout the test.

CA.11.2.1 Seismic Certification by ICC-ES AC156 Shake-Table Testing

The current version of ICC-ES AC156 uses the F_p equation from ASCE 7-16 and earlier editions to establish the required response spectrum. Since this appendix establishes a seismic floor acceleration based on modifications to the F_p equation from ASCE 7-22 and later editions, a conversion of this seismic floor acceleration for use in ICC-ES AC156 is necessary. This section describes that conversion. In short, the seismic floor acceleration of this appendix replaces $0.4S_{DS}(1+2z/h)$ for the horizontal component in ICC-ES AC156. It should be noted that the flexible and rigid within-component amplifications/adjustments (i.e., a_p in ASCE 7-16 and earlier editions and C_{AR}/R_p in ASCE 7-22 and later editions) are assumed equal to that prescribed in ICC-ES AC156 (i.e., 2.5 and 1.0, respectively) for simplicity even though C_{AR}/R_p can take on different values depending on component in Chapter 13. For the vertical component of the required response spectrum in ICC-ES AC156, S_{DS} is simply replaced by S_{DSfr} to address the adjustment from design earthquake to FRE_R .

REFERENCES

CCR (California Code of Regulations). 2022. *California Building Code*, Sacramento, CA: CCR.

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ICC (International Code Council). 2010. *Acceptance Criteria for Seismic Certification by Shake-Table Testing of Nonstructural Components*. ICC-ES AC156. Washington, DC: ICC.

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CA.12 DOCUMENTATION AND REVIEW

CA.12.1 Scope

The exception notes that a functional recovery coordinator is not required for Functional Recovery Category D. This does not mean that this section does not apply for Functional Recovery Category D. Rather, it means that the specific requirements related to the functional recovery coordinator are not required. Requirements for documentation, delegated design, seismic certification, and design changes during construction still apply to Functional Recovery Category D.

CA.12.2 Documentation

Similar to how many building codes require the lateral force-resisting system and lateral force-resisting elements and components to be identified in construction documents, this standard requires multiple design parameters and criteria for functional recovery to be identified in construction documents. Elements, components, equipment, and distribution systems which support the basic intended function may be identified in the construction documents in the form of a table by

discipline (e.g., architectural, structural, mechanical, electrical, plumbing, fire protection, etc.) in the general notes section of each discipline's drawings or on one discipline's drawings (e.g., architectural drawings).

Note that documentation of nonstructural elements, components, equipment, and distribution systems required by this section exceeds current state-of-the-practice. However, while layout of nonstructural systems, especially critical interdependencies between systems, is required, extensive and precise dimensioning of all nonstructural elements, components, equipment, and distribution systems for every project is not. Examples may include:

- More thorough plans and details of ceilings including supports, bracing, edge conditions, splices, vertical offsets, and interfaces between dissimilar ceiling types as well as documentation of individual items within suspended ceilings.
- More thorough plans and details of distribution systems including size and type, locations for bracing, and bracing details.
- More thorough plans and details for partitions including wall finishes and connections, stud/track/header sizes, top and bottom anchorage, bracing and bridging details where required, header sizes and details, backing plates, and permissible penetrations.

The registered design professional who is delegating the design to a delegated designer (e.g., engineer of record for the primary structure) and the functional recovery coordinator are required to specify the functional recovery criteria for delegated design even though actual delegated design is by separate registered design professional. This is necessary because the registered design professional responsible delegating the design and the functional recovery coordinator have the most complete picture of the elements, components, equipment, and distribution systems which are required to support basic intended function. Delegated designers often are not given this holistic perspective when they are performing their work.

CA.12.3 Delegated Design

Delegated design of certain elements, components, equipment, and distribution systems can be critical to achieving functional recovery performance. Historically, delegated design items have been more likely to be overlooked or given less attention than, for example, design of the primary structural system. Having a registered design professional perform delegated design is a necessary, but not sufficient, requirement.

Required review by registered design professional delegating the design (e.g., engineer of record for the primary structure) is limited to criteria and general implementation only. Enhanced review by functional recovery coordinator is necessary for structures assigned to Functional Recovery Category A and B to ensure not only that the criteria and general implementation have been performed appropriately but that detailed design and detailing aspects related to function recovery have not been missed.

Note that delegated design may be deferred (i.e., occurring after permit issuance for the main structure) or not. Regardless of whether the delegated design is part of a deferred approval or not, it is subject to the requirements of this section.

CA.12.4 Seismic Certification

The method of compliance/exemption for seismic certification for each component or equipment is required to be tracked to avoid a component or equipment “slipping through the cracks” and compromising the functional recovery performance. Note that the registered design professional for the architectural design is typically not sufficiently qualified to review and accept seismic certification documentation, unless they simultaneously meet the requirements to serve as the functional recovery coordinator.

CA.12.5 Design Changes During Construction

The characterization of “substantially altered or changed” is, to a degree, a subjective assessment by the registered design professional. When in doubt, it is recommended that the relevant parties be notified for assessment of whether “substantially altered or changed” applies.

CA.12.6 Functional Recovery Coordinator

The functional recovery coordinator role is a distinct position on the design team which has historically not been required for life-safety-based designs. The functional recovery coordinator is essentially the overseer of the entirety of the functional recovery design and actualization of that design for the building or nonbuilding structure. This role is necessary for functional recovery specifically so that one individual/entity has a holistic perspective of the entirety of the functional recovery aspects of a project and can prevent items necessary for functional recovery from “slipping through the cracks”. It is known that functional recovery can be “short-circuited” by one or more items (e.g., a main air handling unit) not being correctly designed or installed. The addition of the functional recovery coordinator to the design team provides greater confidence that such “short-circuits” are not missed. The functional recovery coordinator has significant responsibilities per the requirements of this section.

While the registered design professional responsible for any portion(s) of the design (e.g., engineer of record for the primary structure) may also serve as the functional recovery coordinator, where they do not have sufficient experience with functional recovery (e.g. structural engineer without sufficient experience with nonstructural systems), a separate registered design professional will need to serve in the role of functional recovery coordinator.

Note that construction documents include, in addition to the drawings, specifications from disciplines other than the architect and engineer of record (e.g., mechanical, electrical, plumbing, fire protection, etc.). The portions of those specifications which address functional recovery aspects are within the scope of review by the functional recovery coordinator.

While not required by this appendix, the functional recovery coordinator and general contractor may explore possible ways to assure nonstructural system coordination. These considerations may include one or more of the following:

- Development of a detailed building information modeling (BIM) model managed by the general contractor that includes nonstructural systems and their gravity and seismic restraints. This model could then be used to perform clash detection prior to commencement of work.
- Requiring the design professionals responsible for each element, component, equipment, or distribution system to provide alternative seismic restraint details or a library of seismic restraint details that can be used if a clash with the typical detail occurs on site (Masek et al. 2022).
- Development of “zones” for each distribution system. The system routing and gravity and seismic restraints may then only be installed within the given zone to avoid clashes.
- Identifying a “Seismic Restraint Installation Coordinator” to be responsible for coordination of all seismic restraint for nonstructural elements, components, equipment, and distribution systems (BIP, 2024).

References

Masek J.P., McMullin P., and Larsen B., (2022). “Practical Considerations for Non-Structural Bracing Design of Multiple Suspended Utilities in Congested Areas of Facilities.” Fifth International Workshop on the Seismic Performance of Non-Structural Elements (SPONSE), Stanford, CA.

BIP, (2024). “Code of Practice for the Seismic Performance of Non-Structural Elements”. External Research Report ER95, Building Innovation Partnership, University of Canterbury, New Zealand.

CA.13 OBSERVATION, CONFERENCES AND WALK-THROUGHS

CA.13.1 Scope

The process of functional recovery observation is intended to be similar to structural observation required by the applicable building code but focused on functional recovery aspects of design.

CA.13.2 Observation Responsibility and Reports

While there is often significant value in the registered design professional responsible for the design performing the required functional recovery observations, this is not required. Rather, a separate registered design professional may perform the required functional recovery observations so long as that registered design professional is capable and qualified in aspects of the construction under

their observation. Where the registered design professional responsible for the design performs the functional recovery observations, having the individual engineer who performed specific designs observe those specific parts of construction may be especially beneficial. In this section, the term functional recovery observation is used similarly to structural observation except that it is not exclusive to structural (e.g., includes nonstructural).

CA.13.3 Pre-Construction Functional Recovery Conference(s)

The pre-construction functional recovery conference(s) required in this section are intended to mimic other pre-construction conferences commonly held such as those prior to the start of major concrete and reinforcing steel or structural steel work. The pre-construction functional recovery conference(s) serve to ensure that the functional recovery requirements are passed from the design team to the construction, observation, and inspection teams. For example, pre-construction conference(s) may include reviewing together the seismic certification requirements for HVAC equipment or helping educate the contractor on expected additional functional recovery observations required (e.g., more frequent visits by architect to observe partition wall metal stud construction matches detailing).

The provisions for pre-construction functional recovery conference(s) are intentionally written to permit flexibility for project size and complexity. For some small projects, a single conference may be sufficient. On the other hand, for larger or more complex projects, it may be necessary to have several conferences: one for a general orientation on the functional recovery requirements, one or more for structural work, and one or more for nonstructural work. The number and timing of pre-construction conferences for structural and nonstructural work will depend on which trades and construction teams will be present at which point in construction. For example, it may be necessary to hold a nonstructural-focused pre-construction functional recovery conference as the envelope system work commences which is months to years before interior architectural, mechanical, electrical and plumbing begins. IN such cases, an additional conference when significant interior architectural, mechanical, electrical, and plumbing work begins may therefore be prudent.

CA.13.4 Post-Construction Functional Recovery Walk-Through(s)

Post-construction functional recovery walk-through(s) serve to ensure that construction procedures and installation have generally followed the design intent and that unforeseen conflicts which could impair functional recovery are addressed. While coordination prior to and during construction is essential, the full picture of potential interactions between functional recovery systems is only available once everything is installed, especially for projects involving complex nonstructural systems. In such situations, it is common that multiple trades and construction teams were installing different components and at different times. Post-construction functional recovery walk-through(s) therefore provide the crucial opportunity for a final check. Similar to provisions for pre-construction functional recovery conference(s) as discussed in Section CA.13.3, the provisions for post-construction functional recovery walk-through(s) are intentionally written to permit flexibility for project size and complexity.

CA.13.5 Final Written Statement of Functional Recovery Observations

No commentary proposed.

CA.14 QUALITY ASSURANCE

CA.14.1 Scope

The requirements in this section are those which are more traditionally found in a applicable building code than in this Standard. However, it is recognized that quality assurance requirements are essential for and particularly intertwined with expected outcomes for functional recovery. As a result, they have been incorporated directly into this appendix.

Based on industry surveys/workshops (Valigura et al., 2024) and committee discussions during development of this appendix, several shortcomings in current code provisions and existing practice were identified and addressed as shown in Table CA.14-1.

Table CA.14-1. Quality assurance issues identified and requirements introduced

Issue Identified	Requirements Introduced
Lack of review of substantive changes during construction	Functional recovery coordinator (Section A.12)
Lack of tracking special inspections	Acceptance reports (Section A.14) Contractor responsibility - functional recovery statement of responsibility (Section A.14)
Lack of nonstructural component coordination and ownership for performance	Functional recovery coordinator (Section A.12)
Need for construction team “buy-in”	Pre-construction functional recovery conference (Section A.13) Contractor responsibility - functional recovery statement of responsibility (Section A.14)
Lack of ownership of observations	Final written statement of observations (Section A.13)
Unclear requirements on construction quality assurance	Statement of special inspections (Section A.14)
Vague definition of periodic and continuous special inspection	Definition of continuous and periodic inspection (Section A.14)
Difficulty in prescribing special inspection requirements for nonstructural components	Special inspection of nonstructural elements, components, systems, equipment, and distribution systems (Section A.14)

Testing of post-installed anchors for nonstructural components not required

Tests of post-installed anchors (Section A.14)

Lack of planning for required post-earthquake actions

Post-earthquake functional recovery plan (Section A.15)

CA.14.2 Special Inspection and Testing Responsibility

No commentary proposed.

CA.14.3 Statement of Special Inspections

The requirements for the statement of special inspections in this section are intended to complement and supplement those required by the applicable building code (e.g., Chapter 17 of the International Building Code) by adding special inspections and tests relevant to functional recovery. It is assumed that the special inspections and tests typically addressed by the applicable building code will also be included (e.g., special inspections and tests of the seismic force-resisting system) and are therefore not duplicated in this appendix. The statement of special inspection outlines the frequency and comprehensiveness of the special inspection. This is carried out by definitions of special inspection defined in this section, building code, or referenced standards. The requirements of this section expend the requirements of special inspection defined in code or standard by requiring inspection of all components.

In some cases, the design for functional recovery could be performed using lateral force resisting systems that are not designated as special. In such cases, the building and/or material standards usually do not require such an involved special inspection. In these, or similar cases, where requirements are not provided, the functional recovery coordinator and/or RDP should exercise their judgment and should consider requiring higher level of special inspection than required by minimum requirements of the code and/or material standard.

While not required, it is recommended that the statement of special inspection or internal log lists the registered design professionals responsible for the systems required to support the basic intended function. Their inclusion can streamline the development of remedies when inspection identifies a deficiency as well as provide clear direction for who should be reviewing the inspection reports.

Registered design professionals may not always possess enough understanding to specify exact requirements on special inspection or field testing in order to prevent loss of functionality after the earthquake. In these cases, the registered design professional should work closely with the supplier, manufacturer, or other trade specialists to develop special inspection tasks.

The special inspector should possess the necessary expertise for inspecting the components they are in charge of inspecting. The required qualifications may include specific programs or certifications, if available, that are necessary to verify a special inspector is qualified. For example individual(s) performing the special inspection of post-installed anchors in concrete and masonry

could be required to be certified in accordance with ACI's Post-Installed Concrete Anchor Installation Inspector program.

CA.14.4 Continuous and Periodic Special Inspections

The definition of continuous and periodic special inspection for functional recovery builds upon and clarifies the definitions in the International Building Code. Specifically, the definition explicitly requires that all components that require special inspection are inspected irrespective of whether the special inspection is continuous or periodic.

The difference between the continuous and periodic special inspection is that for periodic, the special inspector is not required to inspect all components while they are being installed. The requirement for periodic inspection is that all components need to be inspected pre- and post-installation.

In absence of definition in applicable building code, the designer can elect to adopt definition from model building code with addition stated in Section A.14.4 or adopt following definition:

“Continuous and periodic special inspections for functional recovery shall include verification of materials and any required qualification prior to the commencement of the work, continuous or periodic, respectively, inspections of the work in progress, and inspection of all completed elements of the work or portions prior to acceptance or covering.”

CA.14.5 Contractor Responsibility

CA.14.5.1 Functional Recovery Statement of Responsibility

Only the general contractor is required to sign the functional recovery statement of responsibility since requiring every subcontractor to sign is deemed impractical. However, the requirements include that the general contractor must communicate the functional recovery requirements to their subcontractors. Ultimately, the general contractor is responsible for overseeing and enforcing the functional recovery requirements for their subcontractors.

CA.14.5.2 Schedule and Log

The general contractor is best equipped to maintain a schedule of work and required special inspections. This log is periodically updated and shared among the construction and design team to ensure all special inspection and tests are performed. The frequency of updating and sharing of the schedule is project specific and should be discussed during pre-construction meeting. The specialty/component engineers often work as subcontractors to either the general contractor or the design team and need to be kept up-to-date on the schedule to help facilitate special inspections and tests.

CA.14.6 Functional Recovery Compliance Statements

The requirement for functional recovery compliance statements is modeled after California Department of Health Care Access and Information (HCAI) requirements for verified compliance reports. The functional recovery coordinator, special inspector, and contractor each sign and submit these statements at certain milestones of the project to verify that all of the construction has been performed in accordance with the construction documents. The intent of these statements is to aid in the proper construction and installation of both structural and nonstructural components and to ensure that all special inspection, testing and observations are performed.

Besides the milestones required in the provisions, the functional recovery coordinator can specify additional intervals for submitting compliance statements. The functional recovery coordinator may consider the size of the project in deciding whether to include additional intervals. For large projects, the time between the milestones identified in the provisions may be months or even more than a year. In these cases, the functional recovery coordinator should consider intermediate milestones to ensure that if any non-compliant items are identified in the statements, they can be resolved in a timely manner.

For the compliance statement:

- The functional recovery coordinator should identify all observations performed and observations reports issued. It is also suggested that all special inspection and testing reports received be identified.
- The special inspector should document all special inspection and tests performed and reports issued.
- The contractor should document all elements of construction (including nonstructural components) installed.
- All parties, to best of their knowledge, should identify any unresolved non-compliant items.

For identified non-compliant items, the functional recovery coordinator should first attempt to lead their resolution. For example, for a non-compliant attachment of a nonstructural component to the main structure, this might include coordination by the functional recovery coordinator between the structural engineer of record, engineer responsible for the nonstructural attachment, and contractor in developing a one-off detail for the given condition.

Where these attempts do not lead to resolution of the non-compliant item, the functional recovery coordinator needs to notify AHJ.

CA.14.7 Required Additional Special Inspections for Functional Recovery

Functional recovery performance is highly governed by performance of nonstructural components. To support this performance, Table A.14-1 primarily focuses on nonstructural components and their

seismic restraint, with requirements for main gravity and lateral force resisting system components referencing the applicable building code. Some jurisdictions do not adopt the special inspection requirements of model code. In these cases, the designer should consider specifying special inspection requirements according to model code or material standard. Further, the design team can elect to go beyond the requirements of the building code and increase the level of special inspection from periodic to continuous for some critical structural or nonstructural elements.

Table A.14-1 is structured based on the categories used for establishing basic intended function in Section A.4. Each of the categories lists the most common components. However, the table is not exhaustive and each of the categories also contains the additional row “other ...” to allow the registered design professional preparing the statement of special inspection to include other components in each category specific to their project.

The special inspection requirements vary based on the functional recovery category assignment. Special inspection is required for structures assigned to Functional Recovery Category A through D for items that are identified as:

- Long lead items since such items, by definition, require an extended time to repair or procure a replacement. Damage to long lead items are expected to compromise the ability of the structure to achieve its functional recovery time target.
- Items that can cause flooding (such as water piping), as it is expected that flood repair would compromise the ability of the structure to achieve its functional recovery time target.
- Life-safety items such egress, fire suppression, and gas and hazardous material piping items as a precaution to ensure the enforcement of the design requirements.

Special inspection is required for structures assigned to Functional Recovery Category A through C for items that are identified as:

- Equipment not flagged as a long lead item. It is assumed that procurement of new equipment would take longer than a couple days as required for Functional Recovery Category B.
- Piping (not already covered for Functional Recovery Category A through D), ducts, electrical distribution systems and other items that would require a specialized contractor and/or where repair/replacement may not be easily accessible.

Special inspection is required for structures assigned to Functional Recovery Category A through B for items that are identified as:

- Architectural items and nonstructural walls. It is assumed that any required repair or fixes could be performed relatively quickly but not in sufficient time to achieve the performance objective of Functional Recovery Category B.

Special inspection is required for structures assigned to Functional Recovery Category A for items that are identified as:

- Backup systems. Per A.16, these systems are required only for Functional Recovery Category A. It is assumed that damage to the backup systems would compromise the ability of the structure to achieve its functional recovery time target.

The requirements on special inspection in Table A.14-1 are minimum requirements. The statement of special inspection can extend the special inspection requirements to lower functional recovery categories. For example, in structures assigned to Functional Recovery Category B, where owner voluntarily elected to have some or all backup systems, the statement of special inspection can require special inspection of backup systems to ensure the systems are properly installed, even though it is not required for Functional Recovery Category B per Table A.14-1.

The section also provides minimum requirements on actions that need to be performed for special inspection of the items presented in Table A.14-1. These actions are based on committee judgment. The statement of special inspections can include additional actions required on a specific project basis.

In general, special inspection of nonstructural components focuses on seismic restraint installation and displacement compatibility. Special inspection ensures that seismic restraints are installed using details and at locations as shown in construction documents. Further, the special inspection focuses on required clearances and that components and their connections can accommodate the anticipated seismic displacements either through strength or flexibility. All of these tasks can be still inspected after the component is installed. Hence, only periodic special inspection is required. However, there might be components for which the installation procedure also matters. For these components, the registered design professional should require continuous special inspection. An example are post-installed anchors. While the default special inspection is periodic, because of the testing requirements, the registered design professional can lower the testing requirements if they prescribe continuous special inspection.

CA.14.7.5 Special Inspection of Structural Elements

Special inspection for functional recovery for structural elements that are part of the either main gravity or main lateral force-resisting systems is intended to match that currently required for life safety in accordance with the International Building Code. Required special inspection of the components that are not part of main gravity or lateral systems (e.g., egress stairs, nonstructural CMU walls, frames that support nonstructural elements, and others) is included to achieve a similar level of scrutiny as is required for the main gravity force-resisting system.

CA.14.7.6 Special Inspection of Nonstructural Walls

Nonstructural walls are susceptible to damage due to interstory drifts. Special inspection of these walls, in addition to other requirements, ensures that drift compatibility can be achieved in accordance with the details shown in the construction documents. Drift compatibility is usually

achieved by providing specialized connections at the top of the wall to the structure above, Nested track or proprietary slotted connections are examples. Special inspection ensures that these connections are correctly installed.

CA.14.8 Required Tests of Post-Installed Anchors for Functional Recovery

Post-Installed anchors for structural and nonstructural components are required to be tested when the component that they anchor is required to be inspected in accordance with Table A.14-1. This section permits three options for the testing:

1. Proof load testing in accordance with Section A.14.8.1 with a testing frequency in accordance with Table A.14-3. Under this option, special inspection of the post-installed anchors is permitted to be periodic since the testing frequency provides an approximately 95% confidence level that all anchors are installed properly.
2. An exception to Option 1 is given for post-installed anchors with a low total number of anchors (up to 1000). This section permits lowering the testing frequency to 25% as long as special inspection is performed continuously. Essentially, this exception trades confidence in performance of the anchors based on testing frequency for confidence in performance based on continuous inspection.
3. Failure testing in accordance with Section A.14.8.2 which essentially “prequalifies” the installers to install a given category of anchors on a project without the need for additional proof load testing. Such “prequalified” installers would be the only ones allowed to install that category of anchors without additional proof load testing per Options 1 or 2. However, in this option, the statement of special inspections can still require up to 5% proof load testing.

This section also exempts testing of anchors in connections where all anchors have a low demand-to-capacity ratio. In this case, it is assumed that the connection has sufficient redundancy such that the connection would still perform adequately even if some of the anchors were to fail.

The use of power actuated fasteners in seismic design is limited in ASCE 7 Section 13.4.5. Given these limitations, power actuated fasteners are exempt from testing.

Where failure testing (Option 3 above) is selected, it is important that the contractor keeps records of which installer installed anchors in which general area for the special inspector’s review. In addition, for adhesive anchors, this section requires that the contractor keeps records of the first anchors installed. This requirement exists because, for many adhesive anchors, it is recommended that the first few “squeezes” of the adhesive are disposed since the adhesive may not yet be fully mixed. Improper disposal of the first few “squeezes” of the adhesive would almost certainly occur at the first anchor of the day and hence would be a preferred location for any proof load testing if required by the statement of special inspections.

For both tension proof load testing (Section A.14.8.1) and failure testing (Section A.14.8.2), no restriction is imposed on the distance between the anchor and the loading reaction equipment for

adhesive anchors. This is because the testing required here focuses on proper installation of the anchor and any bonding issues between the anchor and surrounding concrete. This approach may restrict some concrete failure modes (e.g., concrete breakout). However, these failure modes are generally unrelated to the anchor installation process and therefore not the scope of the testing required herein. The testing is done on anchors in uncracked concrete in confined conditions, while the anchor capacity is calculated using cracked concrete and will be loaded in unconfined manner. The ratio between capacity in uncracked and cracked concrete is assumed as 1.4 per ACI 318. The commonly used value for ratio in capacity between unconfined and confined test is 0.75. The testing loads are calibrated to target fraction of nominal capacity as follows:

- For proof load, the testing value is set to be sufficiently high to test correct installation, but not high enough to cause damage. Value $1.25 \times$ design strength results in testing to approximately 60% of anchor nominal capacity at the time of testing. This is because the design strength is calculated for cracked concrete and in seismic conditions. These conditions have modifiers on capacity in ACI 318 of $1/1.4$ and 0.75 , respectively. However, at the time of testing, the concrete is uncracked and test is pseudo-static, hence the actual capacity of the anchor will be significantly higher than design strength. This value is also consistent with derivation presented in literature (Mattis and Silva, 2019).
- For failure test, the anchors are tested to nominal strength without modification for cracked concrete or seismic loading. Since that is the state at the time of testing, the properly installed anchors should develop this capacity. Furthermore, it is expected that, for failure tests, the testing load will often be governed by the yielding of the anchor, since the design loads for all of the conditions where the anchors may be used during construction would likely not be known at the time of qualification of the installers. There might be a special case, where the design intent is to provide fully ductile behavior of the anchor and achieve tensile failure of the anchor as the desired failure mechanism. Under this condition, the failure test load would need to be increased to the ultimate capacity of the anchor rather than yield capacity. However, since this condition is not required but rather is a design option, it is left to the designer to specify this testing load where necessary.

For proof load testing (Section A.14.8.1), the test frequency in Table A.14-3 is prescribed to provide an approximately 95% confidence level that all anchors within the given category are installed properly. This confidence level aligns with the probabilities of failure that were used to determine I_{pr} in Section A.10 (i.e., 2.5%, 5% and 7.5% for Functional Recovery Category A and B, C, and D, respectively). However, the distinction between Functional Recovery Categories is not made here since the frequency would not be significantly affected. The test frequency is determined based on Cochran's formula (Cochran, 1977). Given the lack of information on the quality of anchor installation, the population proportion (one of the inputs in the formula) is set to 50% (i.e., it is equally likely that the anchor is faulty or that it is properly installed). This assumption provides conservative values for the test frequency. It is acknowledged that this assumption may be conservative. As such, the test frequency is lowered for small populations based on committee judgment as shown in Figure CA.14-1. Similarly, the 10% test frequency for large population size is also based on committee judgment.

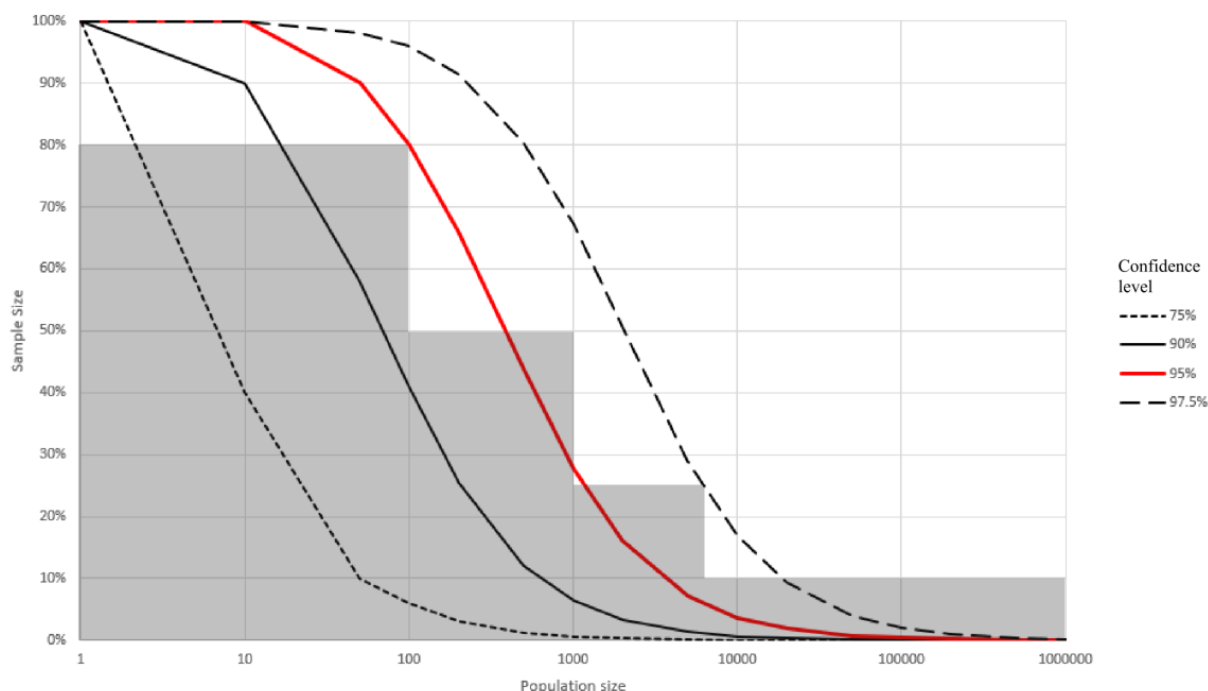


Figure CA.14-1. Sampling frequency versus population size for different confidence levels

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CA.15 POST-EARTHQUAKE FUNCTIONAL RECOVERY PLAN

CA.15.1 Scope

No commentary proposed.

CA.15.2 Responsibility

While there is value in the registered design professional responsible for the structural design or the functional recovery coordinator preparing the functional recovery plan, this is not required. Rather, a separate registered design professional may prepare it.

CA.15.3 Functional Recovery Plan

A functional recovery plan is an extension of accelerated building reoccupancy programs in place in certain jurisdictions (e.g., Building Occupancy Resumption (BORP) or Back-to-Business (B2B) programs). The provisions of this section do not require formalization of the post-earthquake functional recovery plan with the Authority Having Jurisdiction, rather only that the plan be submitted to the Authority Having Jurisdiction and be made accessible (e.g., through a printed copy kept on site or electronically). If the functional recovery plan and drawings are made accessible electronically, consideration should be given to electronic access in a post-earthquake environment (e.g., external power loss at building location).

FEMA P-2055 *Post-Disaster Building Safety Evaluation Guidance* (FEMA, 2019) and FEMA P-2055-1 *Guidance for Accelerated Building Reoccupancy Programs* (FEMA, 2023) are good resources for guidance on developing aspects of a functional recovery plan. Additional considerations which may be appropriate to address in a functional recovery plan include: where PPE and tools for evaluation are stored, elevator firm name and contact if needed for function, a building information narrative, detailed instructions regarding where to look, what to look for, and how to obtain access to areas to be evaluated, detailed instructions regarding how to evaluate specific structural and non-structural elements, components, equipment and distribution systems, detailed instructions regarding how to interpret observed damage, and detailed instructions regarding additional evaluation procedures to be performed following aftershocks. Where it is possible to keep materials on site for expedient repair, this may be prudent, especially for long-lead items.

Note that the post-earthquake functional recovery plan is meant to supplement, not replace, post-earthquake evaluation (e.g., red, yellow or green tagging via ATC-20). Where the Authority Having Jurisdiction permits advance preparation of an earthquake safety evaluation plan, it may be prudent to combine with the functional recovery plan.

Facility personnel (e.g., building maintenance), where present, are likely to be able to perform the majority of the post-earthquake functional recovery evaluation. Due to their familiar knowledge of the structure and, particularly, nonstructural systems and their regular presence on site, they are likely to be able to respond faster than a registered design professional under a contractual arrangement. The provisions of this section therefore require identifying items which must specifically be evaluated by a registered design professional (e.g., engineer).

The provisions of this section do not require the post-earthquake arrangements with a contractor or engineer to be executed, only that qualifications for such individuals be described. It is ultimately up to the owner or tenant to decide whether to contractually engage a contractor or engineer. When such arrangements are set up, they typically include regular renewal (e.g., every two years) to remain

current. In addition to arranging an engineer for evaluation and a contractor for repairs/replacement, an arrangement for post-earthquake clean-up by a clean-up company may be prudent.

The requirement to include instructions for rapid measures of addressing jammed egress doors, instituting a fire watch, and establishing alternatives for non-functioning elevators is informed by the studies in Blowes et al. (2026b) which demonstrated that the target recovery times were often prevented from being achieved due to these components. Requiring measures to be included in the functional recovery plan is intended to help soften the impact damage to these components may have on functional recovery time. For example, the functional recovery plan could identify methods by which, after jammed doors had been rammed open, they could be temporarily secured/latched without the need to close fully into their frame. Alternatives to non-functioning elevators depend on the occupancy/use of the structure but may include moving functions needed by less able individuals to the ground floor. More description of the effect of jammed egress doors, fire watches, and non-functioning elevators on recovery time can be found in Section CA.17. Further research into the fragility and consequence of these nonstructural systems may help reduce the need for such measures to be included in the functional recovery plan.

On-site backup requirements do not apply to structures assigned to Functional Recovery Categories other than A. Since it is likely that some electrical power, water, and sewage storage will be needed between the earthquake event and the target functional recovery time (i.e., 3 days for Functional Recovery Category B) for people performing cleanup or temporary repairs prior to the structure's return to use, a requirement is included in the functional recovery plan for structures assigned to Functional Recovery Category B. Where the external power, water and sewer supply is disrupted due to external factors, on-site power, water and sewage options may need to be provided by temporary means such as generators, bottled water, and pit toilets. Such temporary means are not expected to be integrated into the structure's design (except as required by Section A.16) but rather provided as part of the cleanup and temporary repair operations. Reference Section CA.2 for discussion of external factors.

Note that a structure's owner/occupant or a specific authority having jurisdiction may be interested in requesting that the individual(s) preparing the functional recovery plan address external factors for structures not only assigned to Functional Recovery Category A and/or not only those external factors addressed in Section A.16. One example is disruption to supply chains. It is known that earthquakes can have a critical impact on local production, as many warehouses and factories may be damaged or destroyed. This results in a shortage of supplies and a decrease in production capacity. Manufacturing problems may persist due to high costs, limited materials, and the challenge of maintaining a workforce as many workers are displaced. If essential functions within a building are dependent on regular delivery of supplies or materials, consideration should be given to storing surplus items to accommodate interruptions in deliveries. For example, California hospitals are required to maintain at least one week's supply of stable (non-perishable) foods and at least two days' supply of perishable foods. Other conditions that may warrant consideration include critical equipment that, if damaged, would cause an unacceptable loss of function. In such cases, spare parts for repairs, spare equipment for replacement, or providing off-site redundancy may be necessary.

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CA.16 ON-SITE BACKUP AND EXTERNAL HOOKUP REQUIREMENTS

CA.16.1 Scope

On-site backup for structures assigned to Functional Recovery Category A is required to address potential for short-term loss of external power, water and sewer (e.g., due to failure in an earthquake of the power, water and sewer networks leading to the building).

Note that 72 hours is used to match current state-of-the-practice requirements for critical-care facilities (e.g., hospitals in California). Seventy-two hours is not an estimate of the time to regain external power, water and sewer. Rather, the requirement for 72 hours is intended to provide a general buffer against these external factors. Therefore, it is possible for the on-site backups to “run out” before external power, water and sewer are re-established.

While full on-site backup is not required for structures assigned to Functional Recovery Category B, an electrical power external hookup is required. As discussed in Section CA.15.3, this aids in re-establishing function for structures assigned to Functional Recovery Category B.

CA.16.2 On-Site Backup Requirements for Functional Recovery Category A

CA.16.2.1 Electrical Power

The electrical power system providing the backup does not have to be the same system as the day-to-day electrical power system, although it can be.

Note that it may be possible to obtain fuel replenishment in order to reduce the amount of fuel required to be stored on site (e.g., as permitted by jurisdictions such as the California Department of Health Care Access and Information (HCAI)). However, this is not permitted in this section as fuel

replenishment may not be reliable for certain locations or occupancies after an earthquake. Where back-up power does not rely upon fuel (e.g., a photovoltaic panel and battery energy storage system in lieu of generators), the requirements for on-site supply of fuel do not apply.

CA.16.2.2 Potable Water

The water supply system providing the backup does not have to be the same system as the day-to-day water supply system, although it can be. For example, stored water on site or an on-site well could serve as the backup water supply so long as measures are taken to ensure the water from such sources remains potable after the earthquake (e.g., through appropriate treatment or filtration).

The on-site backup water supply referenced in these requirements is specifically for potable water. These requirements are not intended to cover water required for the fire suppression system (secondary or otherwise) which may be required by the applicable building code

Note that it may be possible to obtain water replenishment in order to reduce the amount of potable water required to be stored on site (e.g., as permitted by jurisdictions such as the California Department of Health Care Access and Information (HCAI)). However, this is not permitted in this section as water replenishment may not be reliable for certain locations or occupancies after an earthquake.

CA.16.2.3 Sewage Storage

The sewage storage system providing the backup does not have to be the same system as the day-to-day sewage system, although it can be. For example, a holding tank which can be pumped could serve as the backup sewage storage system. A means to remove sewage from the backup sewage storage system is required (e.g., access for pumping a holding tank).

Note that it may be possible to obtain sewage disposal in order to reduce the amount of sewage required to be stored on site (e.g., as permitted by jurisdictions such as the California Department of Health Care Access and Information (HCAI)). However, this is not permitted in this section as sewage disposal may not be reliable for certain locations or occupancies after an earthquake.

CA.16.3 Electrical Power External Hookup Requirements for Functional Recovery Category B The requirement for an electrical transfer switch provides capability for temporary electrical power to the structure in the event of external electrical power network disruption following an earthquake. However, unlike for structures assigned to Functional Recovery Category A which are required to have a full back-up, the requirements in this section for structures assigned to Functional Recovery Category B only include the transfer switch. A generator and associated fuel are not required to be provided.

CA.17 EXPLICIT FUNCTIONAL RECOVERY ASSESSMENT PROCEDURE

CA.17.1 Acceptable Procedures

Explicit functional recovery assessment procedures explicitly calculate functional recovery time.

Explicit functional recovery assessment procedures are intended to allow for more innovative design to achieve functional recovery than the prescriptive provisions by providing the user with an avenue to exercise more sophisticated functional recovery estimation methods. This section can be used for any structural system, and is not limited to the systems listed in Table A.8-1.

The term “explicit functional recovery assessment procedure” as opposed to the seemingly more straightforward choice of “performance-based procedure” was specifically selected to differentiate this procedure from the type of performance-based design practiced for life safety. While the requirements of this section may be used with a non-conforming seismic force-resisting system and nonlinear response history analysis (i.e., where performance-based design is commonly practiced), they may also be used at the user’s option with a conforming seismic force-resisting system and, under certain limitations, with linear analysis. Therefore, a clear distinction between these procedures and performance-based design was deemed necessary.

Rather than directly specifying a method, this section describes the requirements of an acceptable method. This permits the user to select their preferred explicit functional recovery assessment procedure so long as it satisfies the requirements of this section. Additionally, this approach allows new methods currently under development or developed in the future to be used, as long as they satisfy the requirements identified in Section A.17.1. An example of existing acceptable explicit functional recovery assessment procedures include the most updated FEMA P-58 Functional Recovery Module methodology developed in the ATC-138 project (FEMA, 2021; FEMA, 2025). Other explicit functional recovery assessment procedures for which independent review and industry consensus may not yet have occurred include: modifications to FEMA P-58 made outside of the FEMA P-58 series of projects, Resilience-based Earthquake Design Initiative for the Next Generation of Buildings (REDi) (Arup, 2013), the Tool for Recovery Estimation and Downtime Simulation of Buildings (TREADS) (Molina-Hutt et al., 2021; Molina-Hutt, 2025), and the Novel Framework for Probabilistic Evaluation of Functional Recovery of Building Systems (F-Rec Framework) (Terzic et al. 2021).

Appropriate fragility curves and repair time consequence functions for certain components, especially nonstructural components, may not be available or may be the subject of current research. For example, in the development of the prescriptive provisions for functional recovery (i.e., earlier sections in this appendix for designs not exercising the explicit functional recovery assessment procedure) using ATC-138 (FEMA, 2021), repair of elevators, fire break partitions, and egress doors were excluded from consideration for calculating functional recovery time (Blowes et al. 2026b). See Section CA.10.3 for more information. Ultimately, the decision of whether to exclude or modify fragility curves and repair time consequence functions for specific components will need to

be negotiated between the individual(s) performing the explicit functional recovery assessment and the independent reviewer(s).

Fragility curves are typically assumed to take the shape of a lognormal distribution. This shape generally fits experimental and experiential data fairly well for the majority of the distribution. However, the “tails” of the distribution (i.e., at the very high and low ends) may not be representative of reality. For this reason, it is often acceptable to truncate the distribution, particularly the lower “tail”, to remove sensitivity to the extremes of the distribution. For example, in the development of the prescriptive provisions for functional recovery using ATC-138 (FEMA, 2021), fragility curves were often truncated at -2 standard deviations. While truncations closer to the median than -2 standard deviations may be justifiable, particularly where more extensive quality assurance or inspection is exercised, such assumptions should be reviewed carefully by the independent reviewer(s). Appropriate methods for performing distribution truncation can be found in the literature (Cook et al. 2021).

CA.17.2 Structural Analysis Requirements

Explicit functional recovery assessment procedures require estimation of engineering demand parameters such as peak story drift ratio, residual story drift ratio, and peak floor acceleration. Where component fragilities are also based on local element deformations (e.g., wall hinge rotation), these also need to be determined from structural analysis. Estimation of engineering demand parameters using linear analysis becomes less accurate as the structural system ductility demand increases. Therefore, nonlinear analysis methods are generally required, specifically nonlinear response history analysis, unless a small enough functional recovery response modification coefficient is used in a linear design (i.e., Exception 1) or other rational analysis is performed that includes nonlinear response history effects (i.e., Exception 2).

Exception 2 permits rational analysis methods other than nonlinear response history analysis of three-dimensional models when such methods are judged acceptable by the independent reviewer(s) with respect to the conditions listed in the exception (e.g., ability to estimate engineering demand parameters, ability to capture nonlinear response, etc.). A rational analysis method would therefore need to (a) accurately capture the effect design strength and drift requirements have on the expected strength and stiffness of a structure, (b) account for deformation capacity of the structure, (c) include accurate estimation of elastic modal properties, and (d) reflect nonlinear behavior, all in order to appropriately determine engineering demand parameters. While three-dimensional analysis is not a requirement in Exception 2, it is expected that, for most projects, a three-dimensional elastic model of the structure would be used to quantify project-specific elastic modal properties (i.e., periods and mode shapes). And while project-specific nonlinear analysis is not a requirement in Exception 2, it is expected that, for most projects, determination of project-specific expected/ultimate system strength by adjusting from design strength would be needed (e.g., quantified through mechanism analysis or nonlinear static (pushover) analysis). In order to assess the acceptability of the rational analysis method, the independent reviewer(s) would need to be provided with the analysis details of the structure, background of the alternate structural analysis method, and documentation of any supporting or underlying data used in development of the

structural analysis method. Rational analysis methods in accordance with Exception 2 may be desired in order to allow for more efficient design iteration and are suitable when the nonlinear mode of the structure is well understood. Further restrictions on structural irregularities are also imposed for the rational analysis methods.

An example of a rational analysis which would invoke Exception 2 is the structural analysis method used in developing the functional recovery seismic performance factors for prescriptive structural design in Section A.8. See Section CA.8.4.5. This structural analysis method utilizes detailed two-dimensional nonlinear response history analyses to quantify inelastic damage localization and was applied to generally regular buildings. This structural analysis method was vetted through the technical development process when developing the prescriptive structural design procedure in Section A.8 (Blowes et al. 2026a). This structural analysis method was also used in the benchmarking of the FEMA P-58 Functional Recovery Module and was independently reviewed as part of the project (FEMA 2021).

It is important to note that the analysis requirements in this section are applicable to demonstration of functional recovery performance. They do not place any additional restrictions on when a Chapter 16 nonlinear response history analysis must be performed for the life safety/collapse performance objective. Linear design methods in Chapter 12 are often permitted for the life safety/collapse check in accordance with the requirements and limitations for analysis methods in Chapter 12.

CA.17.3 Acceptance Criteria

Reference Section CA.1.6 for a general description of target functional recovery times. The 80% of area supporting basic intended function limit recognizes that structures may satisfy their basic intended function when the majority, but not necessarily 100%, of that area has recovered. Particularly for structures where nonstructural systems only serve a portion of the structure (e.g., separate HVAC units for each tenant space in a residential building) or for larger buildings, permitting 80% instead of requiring 100% of the area supporting basic intended function allows for specific complications/difficulties in repair that only affect a small portion of the structure's area (e.g., damage to exterior façade in one corner). However, the 80% of area criteria should not be construed as 80% of the elements in each building system. For example, if only 80% of the structural gravity and lateral force-resisting system elements are repaired, that may still not resolve safety-critical structural damage nor structural elements with compromised vertical load-carrying capacity. Thus, the building would not be safe for entry and use. In that example, 0% of the building's area would be functional. As another example, if 80% of the electrical system is functioning but the main distribution panel is still down, the building does not have power and, again, 0% of the building's area would be functional. Note that the value of 80% is selected to achieve functional recovery in a "majority" of the structure based on committee judgment.

CA.17.4 Independent Review

CA.17.4.1 Review Scope and Reviewer Qualifications

The scope of the independent review in this section is focused on the aspects of the functional recovery analysis and design. The scope intends to avoid the independent review becoming a de facto peer review of the safety-based design.

The requirement in Item 1 instructs the independent reviewer(s) to evaluate the appropriateness of the functional recovery assessment procedure selected. In addition to the requirements in Section A.17.1, the independent reviewer(s) may consider whether, in its development, the functional recovery assessment procedure being proposed for use was adequately independently reviewed and has achieved general consensus in the resilience-based design community. If that cannot be demonstrated to the satisfaction of the independent reviewer(s), additional independent review and consensus building may need to occur for the procedure to be used on the project or a different procedure may need to be selected. Also see Commentary Section CA.17.1. The requirement in Item 3 instructs the independent reviewer(s) to evaluate the appropriateness of the structural analyses. It is noted that no prescriptive lower limit on structural requirements (e.g., R_{fr} and Δ_{afr}) exist in this section. However, since the requirements of Chapter 12 for the design earthquake must still be satisfied regardless of the requirements of this appendix, they provide a lower bound on structural strength and stiffness. In addition, the independent reviewer(s) may compare or may consider requesting that the structural engineer of record compare the structural strength and stiffness of the structural design implemented in accordance with this section against the requirements of Section A.8 (where R_{fr} and Δ_{afr} are available in Section A.8 for the structural system selected). Where the structural strength and stiffness of the design implemented in accordance with this section significantly differs from that which would result from implementing Section A.8, it is important to understand what aspects of the proposed design improve its performance compared to a Section A.8 design (e.g., lower seismic hazard than considered in Section A.8.4.6 for system qualification, better performing building height than that which controlled the system qualification in Section A.8.4.6, specific detailing of replaceable hinge zones or “fuses”, etc.).

The requirement in Item 5 instructs the independent reviewer(s) to have some qualifications and experience with nonstructural design. This is necessary for them to assess whether fragility adjustments in the explicit functional recovery assessment are commensurate with proposed design changes. For example, the designer may choose to implement an enhanced design of a critical piece of equipment’s anchorage (e.g., through introducing an anchorage design with a built-in ductile mechanism). They would then need to assign a suitable enhanced fragility for the enhanced anchorage detail. The independent reviewer(s) will need to assess whether the enhanced fragility the designer is proposing is commensurate with the enhanced design and detailing and not overly optimistic/aggressive. Additionally, a designer may choose to adjust established fragilities, particularly nonstructural, which they believe are outdated and/or no longer reflect current design and detailing practice. However, regardless of implementation in this section, it should be remembered that the nonstructural requirements of Section A.10 and A.11, without exception, still apply.

The requirement in Item 6 is necessary since, for many components, there are component damage states which may not trigger repair prior to regaining basic intended function (i.e., achieving functional recovery). An example is minor concrete cracking in structural walls or gypsum cracking in nonstructural partition walls, neither of which is expected to require repair prior to regaining basic intended function. On the other hand, vertical reinforcing bar buckling in a structural wall almost certainly requires repair prior to regaining basic intended function. While these examples are relatively straightforward, other conditions may not be as clear, which is why they should be reviewed by the independent reviewer. Guidance on what component damage states would trigger impact on functional recovery can be found in ATC-138 (FEMA, 2021).

The requirement in Item 7 is necessary to ensure that components are not assigned as rugged, and therefore excluded from the analysis, without adequate review.

The requirement in Item 8 is necessary since component repair time, repair sequencing, and long-lead time effects are critical to determining the whole building functional recovery time. Particular review attention may be necessary for the assumptions concerning long-lead components or equipment (i.e., components or equipment which may take several months between order and delivery to site). For example, electrical transformers are often considered long-lead equipment.

The requirement in Item 10 is necessary since temporary repairs and alternatives may be considered acceptable and efficient means of achieving faster functional recovery times. However, where such measures are relied upon in the explicit functional recovery assessment procedure, they should be a realistic reflection of what can be achieved under post-earthquake conditions and not simply an analytical tool to reduce calculated functional recovery times.

Quantities in Item 12 refer to counted number of, for example, beam-column connections, air handling units, etc. Especially for Functional Recovery Category A and B, a take-off of quantities from design or construction documents may be necessary in lieu of assumed normative quantities. However, a take-off of quantities may be difficult in certain scenarios (e.g., for projects with extensive nonstructural systems such as large critical-care hospitals, for projects in early stages of design, etc.). It should be noted that the explicit functional recovery assessment can be sensitive to the type and quantity of components used. It is therefore important to have a reasonable estimate of quantities even if a direct take-off is not practical or possible.

CA.17.4.2 Review Letter

No commentary proposed.

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CA.18 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

No commentary proposed.

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2026 NEHRP RECOMMENDED SEISMIC PROVISIONS FOR NEW BUILDINGS AND OTHER STRUCTURES

PROJECT PARTICIPANTS

The Building Seismic Safety Council acknowledges the individuals and organizations that contributed to the development of the 2026 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures.

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Seth Thomas	KPFF Engineers	Voting Member
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IT 1: Soil-foundation design

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Jakub Valigura	Voting Member
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Rahul Sharma	Voting Member
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Lachezar Handzhiyski	Corresponding Member
Zia Zafir	Corresponding Member
Koray Tureyen	Corresponding Member
David Teague	Corresponding Member
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IT 2: Interface between structural elements

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Jim Malley	Voting Member
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IT 3: Ground motion and Geotech issues

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Beatriz Arostegui	Voting Member
Gyimah Kasali	Voting Member
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Charlie Kircher	Corresponding Member
Hossein Mostafaei	Corresponding Member
Jon Heintz	Corresponding Member
Mahmoud Hachem	Corresponding Member
David Teague	Corresponding Member

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Sean McGowan	Corresponding Member
Jason Bock	Corresponding Member
Dustin Cook	Corresponding Member
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IT 4: Evaluation of structures in very high seismic regions

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Bob Hanson	Voting Member
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Seth Thomas	Corresponding Member
Alireza Haji-Soltani	Corresponding Member
Kari Klaboe	Corresponding Member
Ngai-Chi Chung	Corresponding Member
Jeff Berman	Corresponding Member
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IT-05: Drift limit evaluation

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IT 6: Analysis provisions for elastic design

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Koray Tureyen	Corresponding Member
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Name	Role
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IT 7: Non-linear analysis update

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IT 8: Unification of diaphragm design provisions

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IT 9: Building design as rocking system

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IT 11: Alignment of Seismic Isolation provisions with risk targets

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IT 12: Structural design for strength vs ductility

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IT 13: Nurture engineer creativity/innovation

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Future Needs Report for the 2026 NEHRP Provisions

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Organization	Name	Title
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American Iron and Steel Institute	Dan Snyder	Vice President, Construction
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American Society of Civil Engineers	Jeannette Torrents	SEI Technical Director
American Wood Council	Bradford Douglas	Vice President, Engineering
International Code Council	Lawrence C. Novak	Chief Structural Engineer
The Masonry Society	Anne Guiberson	Executive Director

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Organization	Representative
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