National Institute of BUILDING SCIENCES™

BSSC Project 17 Final Report

Development of the Next Generation of Seismic Design Value Maps for the 2020 NEHRP Provisions

December 2019

Innovative Solutions for the Built Environment

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Development of Next Generation of Seismic Design Value Maps for the 2020 NEHRP Provisions

by

National Institute of Building Sciences
Building Seismic Safety Council
Project 17 Committee (chair: Ron Hamburger)

Sponsored by

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The Building Seismic Safety Council (BSSC) was established as a Council of the National Institute of Building Sciences in 1979 for dealing with the complex technical, regulatory, social and economic issues involved in developing and promulgating building earthquake risk mitigation provisions that are national in scope. The BSSC brings together the needed expertise and relevant public and private interests to resolve issues related to the seismic safety of the built environment through authoritative guidance and assistance backed by a broad consensus.

BSSC's fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic planning, design, construction and regulation in the building community. To fulfill its mission, the BSSC: (1) evaluates research findings, practices and field investigations to develop seismic safety provisions; (2) encourages and promotes the adoption of provisions by the national standards and model building codes; (3) provides ongoing education for structural design professionals through training materials, webinars, workshops and colloquia; (4) provides education outreach on seismic design and construction to the non-technical building community and the general public; and (5) advises government bodies on their programs of research, development and implementation.

BSSC is an independent, voluntary membership body representing a wide variety of building community interests. Its activities are structured to provide all interested entities with the opportunity to participate. The BSSC is committed to assessment of advances in engineering knowledge based on design experience and evaluation of earthquake impacts, to lasting technical improvement of seismic design provisions that are implemented in design practice, and to the nation's model building codes and standards.

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For further information on Building Seismic Safety Council activities and products, see the council's website: https://www.nibs.org/page/bssc.

Executive Summary

During the period January 2015 through August 2018, a joint committee of U.S. Geological Survey (USGS) representatives and National Institute of Building Sciences Building Seismic Safety Council (BSSC) volunteers and staff formed a committee to conduct Project 17. The purpose of Project 17 was to formulate recommendations for the rules by which next-generation seismic design value maps, derived from USGS national seismic hazard models, will be developed for adoption by the *2020 National Earthquake Hazard Reduction Program Recommended Provisions for Seismic Regulations of New Buildings and Other Structures (NEHRP Provisions)*, *ASCE 7-22* and the *2024 International Building Code*. Two similar projects, Project 97 and Project 07, had been conducted in the past. Each of these projects established, for a period of approximately ten years, the rules by which design ground motion values referenced by the building codes would be developed both by the USGS and by private consultants engaged in site-specific studies. Project 17 was originally commissioned in response to issues identified in adopting the 2014 edition of the USGS national seismic hazard model and the design procedures that reference them for use, including the *NEHRP Provisions*, building codes and referenced standards. Specific issues included the engineering profession's discontent with the fluctuating design values portrayed by successive map editions; discovery that the standard spectral shape referenced by the design provisions did not adequately represent ground motion amplitude and spectral character on some sites; and a change in seismologic characterization of the possible size of earthquakes originating on various faults and source zones. Project 17 was funded by the Federal Emergency Management Agency (FEMA), and supported by the USGS with some collaborating experts.

An initial planning committee met throughout calendar year 2015 to identify key issues to be considered and to develop a work plan for addressing these as part of the *2020 NEHRP Provisions* update cycle. The planning committee recommended an effort of approximately 30-months duration during which the USGS would develop draft design maps based on the rules proposed, to allow evaluation and refinement of the recommendations.

A Project 17 Committee (P17C) was empaneled and four task subcommittees were formed, each charged with evaluating one of the key issues identified in the planning effort: Stabilizing mapped values; Definition of Acceptable Risk; Development of multi-period spectral parameter data; and, Definition of procedures for computing deterministic limits, should it be necessary to continue use of such caps in development of the maps. A fifth task subcommittee was formed in 2017 to look at ways to stabilize the seismic design category as an extended effort to stabilize mapped values. The P17C met three times per year throughout 2016, 2017 and 2018 to resolve these issues and develop recommendations for an updated technical basis and procedures to be followed in preparing next-generation seismic design value maps for inclusion in the *NEHRP Provisions*. The P17C documented these in the form of draft proposals for revision of the *NEHRP Provisions*. In August 2018, the P17C passed these recommendations to the Provisions Update Committee (PUC) for completion, development of consensus and adoption as appropriate.

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1. Introduction

1.1. PURPOSE

For earthquake engineers and others, this report documents the activities and recommendations of a joint U.S. Geological Survey (USGS) and Federal Emergency Management Agency (FEMA) project (Project 17) to develop a consensus basis for next-generation seismic design value maps and/or tools for adoption by the *2020 NEHRP Recommended Provisions for New Buildings and Other Structures (NEHRP Provisions)*, *ASCE 7-22 Minimum Design Loads Standard* and the 2024 series of I-Codes. These recommendations were prepared by a joint committee of volunteer engineers empaneled by the National Institute of Building Sciences Building Seismic Safety Council (BSSC) and USGS engineers and scientists. BSSC provided project management support for this joint committee and FEMA provided funding for the BSSC participants.

1.2. BACKGROUND

An important goal of the National Earthquake Hazards Reduction Program (*NEHRP*) is to promote the development, improvement, and adoption of reliable, nationally applicable, building code requirements for earthquake-resistant construction. In furtherance of this goal, FEMA has supported the BSSC's periodic development and update of the *NEHRP Recommended Provisions for Seismic Regulation of Buildings and Other Structures (NEHRP Provisions)*. Since 1992 the *NEHRP Provisions* have been the primary resource document for seismic design criteria contained in the *ASCE-7* standard and, more recently, the *International Building Code*. The *NEHRP Provisions* specify seismic design procedures in which required resistance to lateral and vertical forces is computed using spectral response acceleration parameters referenced to a series of national seismic design value maps produced by the USGS in cooperation with BSSC. The *NEHRP Provisions* also specify the procedures by which the values of these design ground motion parameters are determined, either to construct the maps, or based on site-specific seismic hazards study. In developing the *NEHRP Provisions*, BSSC volunteers have typically defined the rules by which the maps are produced (e.g. designation of parameters, hazard levels, etc.) while the USGS has applied these rules using its national seismic hazard model to produce the design maps.

The design procedures contained in the *NEHRP Provisions* are based on a framework first presented in the 1970s, in the landmark *ATC 3-06* document entitled *Tentative Provisions for the Development of Seismic Regulation of Buildings*. The ATC 3-06 document was developed with National Science Foundation funding after damage sustained by numerous modern code-conforming buildings during the 1971 San Fernando earthquake demonstrated that substantial improvement of the contemporary building codes was needed. Since 1985, the BSSC Provisions Update Committee, under FEMA funding, has maintained and updated these design procedures to take advantage of new lessons learned from investigation of the effects of earthquakes on buildings and other structures, and recent engineering and seismologic research. On a periodic basis BSSC publishes the most current recommended design procedures as the *NEHRP Recommended Provisions*. Editions have appeared in 1985, 1988, 1991, 1994, 1997, 2000, 2003,

2009, 2015, and the upcoming 2020. The nation's building code and standards committees rely heavily on the *NEHRP Provisions* as a resource for update and improvement of the nation's design standards and building codes.

Under the *NEHRP Provisions*, seismic design initiates with assignment of a building or other structure to a Seismic Design Category (SDC). A building's SDC is determined based on the intensity of shaking anticipated at the building site during a reference earthquake, termed design earthquake shaking, and the building's intended use or occupancy. The 2015 *NEHRP Provisions* specify six seismic design categories, designated A through F. SDC A encompasses buildings located on sites with such low design earthquake shaking intensity that significant damage is unlikely to occur. Structures assigned to SDC A need not be designed for seismic resistance but must meet certain design criteria intended to provide basic structural integrity under a variety of low intensity events including minor earthquakes, blasts, vehicle impacts, and similar rare occurrences. SDC B encompasses buildings of ordinary occupancy located on sites with anticipated design earthquake shaking that can cause significant damage only to structures with low earthquake resistance. Structures assigned to SDC B must be designed to resist seismic forces computed based on the amplitude and character of design earthquake shaking and consideration of the building's dynamic characteristics. SDC F includes buildings that fulfill an essential post-earthquake function, such as fire stations and hospitals, located on sites with very severe design earthquake shaking intensity, typically experienced only within a few kilometers of a fault that produces a large magnitude earthquake. Buildings assigned to SDC F must meet stringent requirements regarding configuration and detailing of their structural systems and must be designed to have substantial lateral strength and stiffness. In addition, nonstructural systems in these structures, including ceilings, cladding, partitions, mechanical and electrical equipment and utilities must be installed to resist the forces and deformations imparted by the buildings' response to design earthquake shaking. Many types of structures that have historically proven vulnerable to severe earthquake damage are prohibited in SDC F. SDCs C through E have progressively more severe criteria between those assigned to structures in SDC B or F, associated with the progressively more intense design earthquake shaking associated with those categories.

Earthquake shaking occurs in complex wave forms. Figure 1 is a record of such shaking recorded during the 1940 El Centro earthquake. The amplitude, duration and character of earthquake shaking experienced will be different on each site and in each earthquake. Many factors affect this including the earthquake magnitude, the type of faulting that caused the event, the direction of fault rupture either towards or away from the site, distance of the site from the fault rupture, and regional and local site conditions such as the depth and character of soils or rock materials. Many of these factors cannot be determined before an earthquake occurs. Engineering seismologists have developed relationships between these factors and the shaking that occurs at a site, although such relationships have a high degree of variability.

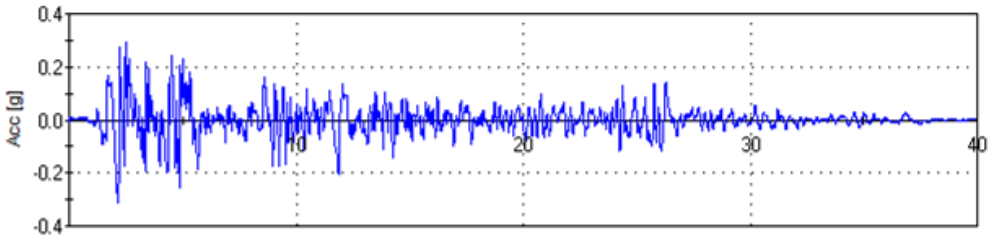

Figure 1: 1940 El Centro Accelerogram, 180° Component

The ATC 3-06 document designated design ground shaking as that intensity of shaking having 10% chance of exceedance in a 50-year period, based largely on then-current information as to the likely mean recurrence interval for large earthquakes in the western U.S. Such shaking has a 475-year mean return period. Shaking intensity was parameterized in the form of smoothed, 5%-damped, elastic acceleration response spectra. Acceleration response spectra are mathematical relationships that indicate the amount of acceleration that will be experienced by a simple structure when excited by an earthquake, as a function of the structure's natural period of vibration, *T*. The structure's natural period, *T*, is dependent on its mass and stiffness, and represents the amount of time, in seconds, it would take the structure to undergo one complete cycle of harmonic motion when displaced, and then released in free vibration. For a structure undergoing such motion, it is possible to relate the acceleration and velocity that the structure will experience to its displacement by the following relationships:

$$V = \frac{2\pi}{T} D \quad (1)$$

$$A = \frac{4\pi^2}{T^2} D \quad (2)$$

where *A* is the maximum acceleration the structure experiences, *V* is the maximum velocity and *D* is the maximum displacement. In the 1970s, N.M. Newmark observed that many earthquake acceleration response spectra obtained from real earthquake ground motion recordings could be enveloped, in the period range important to building design, by a mathematical relationship in which for small structural period, *T*, the maximum acceleration would be constant, over the range of structural periods; for intermediate period, the maximum velocity would be constant; and for large period, the maximum displacement would be constant.

The ATC 3-06 project adopted this approach to characterizing earthquake shaking character and amplitude and used so-called smoothed elastic acceleration response spectra for this purpose. The *NEHRP Provisions* have continued this practice, characterizing the design response spectrum by the following expressions:

$$S_a(T) = S_{DS} \quad (T \leq T_s) \tag{3}$$

$$S_a(T) = \frac{S_{D1}}{T} \quad (T_s < T \leq T_L) \tag{4}$$

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

In the above expressions, $S_a(T)$ is the design spectral acceleration for a structure having period T; S_{DS} is the "constant" design spectral response acceleration for buildings with periods less than a value T_s; T_s is the period at which the spectral shape changes from constant spectral acceleration to constant spectral velocity; and T_L is the period at which spectral shape changes from one of constant response velocity to constant response displacement, and is dependent on the magnitudes of earthquakes producing design shaking at the site. Figure 2 shows a plot of these relationships as contained in the *NEHRP Provisions*.

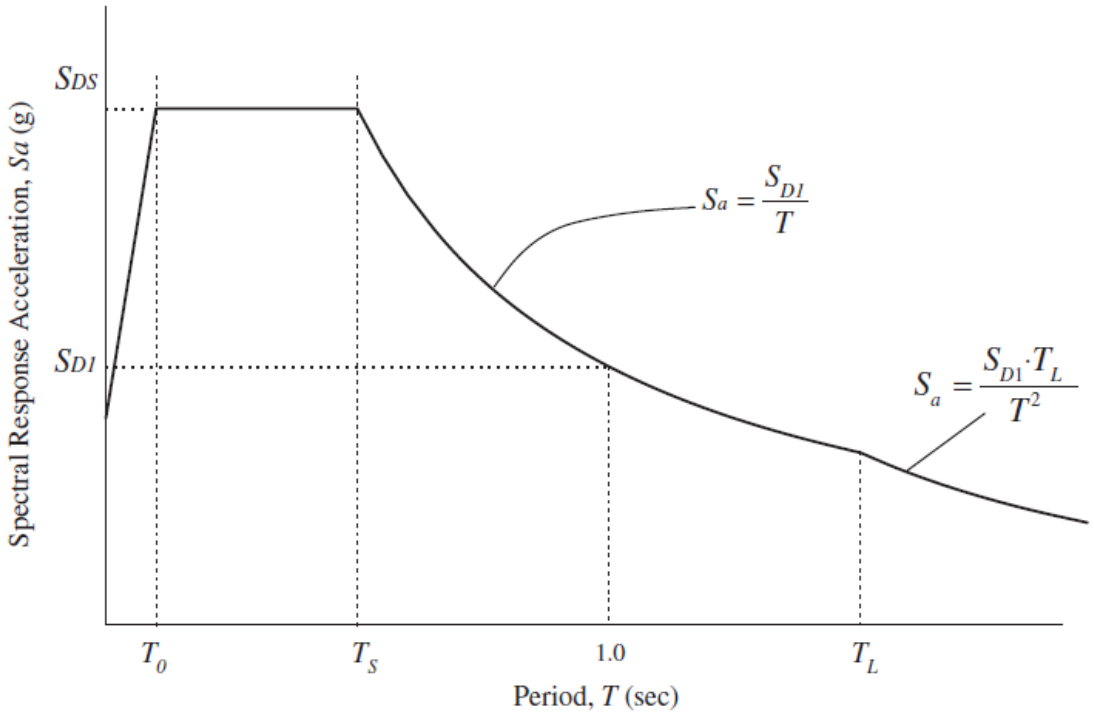

Figure 2: Standard NEHRP Provisions Spectral Shape

The USGS periodically produces national seismic hazard models and seismic design value maps that portray the values of S_{DS}, S_{D1} and T_L. The value of T_S can be determined from the other parameters.

The USGS periodically updates the national seismic design value maps in support of updates to the *NEHRP Provisions*. When developing the updated maps, USGS follows rules established by BSSC in the *NERHP Provisions*, but with updated scientific basis (fault locations, activity rates, ground motion prediction models, etc.) applied to produce more current values for the mapped parameters. Approximately one time each decade, BSSC and USGS have collaborated to re-examine the basis for the design maps and

the rules under which they are derived from USGS national seismic hazard models, resulting in major change to the basis and values contained on the design maps.

During the 1997 provisions update cycle, BSSC and USGS performed Project 97. Project 97 included a group of more than 30 leading engineers and scientists representing private practice and government research and regulatory agencies, who over a period of two years formed a series of subcommittees to explore a variety of topics associated with seismic design procedures and design seismic hazards. In conjunction with this evolution in the design value maps, BSSC made major revisions to the seismic design procedures contained in the *NEHRP Provisions*. As a result of the Project 97 recommendations, the *97 NEHRP Provisions* adopted a series of revisions to the seismic design procedures referenced by the building codes, which included the following:

- Definition of a Maximum Considered Earthquake (MCE) shaking level for which mapped values would be provided.
- Establishment of a 2%- in-50-year exceedance probability for MCE shaking, except in areas near major active faults, where deterministic limits were placed on mapped values.
- Establishment of MCE spectral response acceleration for a reference site class condition (S_S and S_1) as the mapped parameters.
- Establishment of rules for setting a deterministically derived limit on the mapped values of S_s and S_1.
- Establishment of site-adjusted design spectral acceleration values S_{DS} and S_{D1}, taken as 2/3 of the MCE values, following adjustment for Site Class effects, as the parameters used to determine required seismic strength.

The resulting design maps formed the basis for the 1997, 2000 and 2003 editions of the *NEHRP Provisions*, *ASCE 7-98*, *ASCE 7-02* and *ASCE 7-05* as well as the 2000, 2003, 2006 and 2009 editions of the *International Building Code* and *International Residential Code*.

During the 2009 *NEHRP Provisions* update cycle, BSSC and USGS collaborated in an effort known as Project 07, again resulting in substantive changes to the design basis underlying the *NEHRP Provisions* and the design value maps referenced by the provisions. Significant changes included the following:

- Establishment of probabilistic MCE shaking based on a uniform-risk, rather than a uniform-hazard basis. The redefined values are designated MCE_R for risk-targeted Maximum Considered Earthquake shaking.
- Selection of a notional 1%-in-50-year collapse risk as the primary design goal for ordinary occupancy structures located in regions where design seismic values are probabilistically rather than deterministically based
- Selection of maximum direction, as opposed to geomean values for mapped parameters.

During development of the 2015 NEHRP Provisions, the BSSC Provisions Update Committee (PUC) considered a proposal to adopt new design maps developed by the USGS. The USGS produced the new maps using the basic rules established previously by Project 07 efforts but incorporated updated models of source activity rates and segmentation as well as updated ground motion prediction equations. As

would be anticipated, mapped values in some locations increased and in others decreased, with the amplitude of change generally falling under 20%, but sometimes reversing directional trends observed in recent prior map revisions. Of particular note was the increase in a number of deterministic zones due to faults with low activity rate. After initial rejection of the maps, the PUC suggested revision of the deterministic zone definitions. The USGS revised the design maps, and the PUC adopted the revised maps. However, this adoption was not by unanimous vote and several PUC members expressed dissatisfaction with the process for developing the design maps and the lack of opportunity for the structural engineering community to provide input to design map development. This dissatisfaction carried over into the *ASCE-7* Main Committee, which initially rejected the new maps for inclusion in *ASCE 7-16*, though ultimately the maps were adopted. FEMA conceived of the concept for Project 17 to address these concerns and authorized an initial planning effort, conducted in 2015.

The Project 17 Planning Committee initially identified a list of important issues that could be considered in the Project 17 effort:

1. Timing for Updated Map Publication
2. Design Value Conveyance
3. Precision and Uncertainty
4. Acceptable Collapse Risk
5. Collapse Risk Definition
6. Maximum Direction Ground Motion Components
7. Multi-Period Spectral Values
8. Duration as a Mapped Parameter
9. Damping Levels
10. Vertical Motion Parameters
11. Use and Definition of Deterministic Parameters
12. Basin Effects
13. Use of 3-D Simulation to Develop Long Period Parameters

A complete description of these issues may be found in the Project 17 Planning Report, dated September 28, 2015. As noted in this report, the committee recommended that Project 17 focus on four primary issues:

1. Acceptable Risk
2. Deterministic Limits
3. Stabilizing Mapped Values
4. Multi-Period Spectral Values

This report presents the Project 17 recommendations for each of these issues.

1.3. PROJECT PARTICIPANTS

The initial Project 17 Planning Committee included a group of structural and geotechnical engineers who have been active in the BSSC Provisions Update process together with USGS engineers and earthquake scientists, FEMA representatives, and a secretary provided by BSSC. Table 1 below presents the project planning effort participants.

Table 1: Project 17 Planning Committee Participants

Name	Affiliation
David Bonneville	Degenkolb Engineers
C.B. Crouse	AECOM
Edward Field	U.S. Geological Survey
Arthur Frankel	U.S. Geological Survey
Ronald Hamburger	Simpson Gumpertz & Heger Inc.
Robert Hanson	University of Michigan (Emeritus)
James Harris	J.R. Harris and Associates
William Holmes	Rutherford & Chekene
John Hooper	Magnusson Klemencic Associates
Charles Kircher	Kircher & Associates
Nicolas Luco	U.S. Geological Survey
Morgan Moschetti	U.S. Geological Survey
Robert Pekelnicky	Degenkolb Engineers
Mark Petersen	U.S. Geological Survey
Peter Powers	U.S. Geological Survey
Sanaz Rezaeian	U.S. Geological Survey
Philip Schneider	Building Seismic Safety Council
Mai Tong	Federal Emergency Management Agency

In 2016, BSSC empaneled the Project 17 Committee to include the individuals shown in Table 2.

Table 2: Project 17 Committee Members

Name	Affiliation
Ronald Hamburger (Chair)	Simpson Gumpertz & Heger Inc.
David Bonneville	Degenkolb Engineers
C.B. Crouse	AECOM
James (Dan) Dolan	Washington State University
Ben Enfield	City of Seattle
Julie Furr	Rimkus Consulting Group, Inc.
Robert Hanson	University of Michigan (Emeritus)
James Harris	J.R. Harris and Associates
John Heintz	Applied Technology Council
William Holmes	Rutherford & Chekene
Jon Hooper	Magnusson Klemencic Associates
Charles Kircher	Kircher & Associates
Nicolas Luco	U.S. Geological Survey
Steven McCabe	National Institute of Standards and Technology
Robert Pekelnicky	Degenkolb Engineers
Sanaz Rezaeian	U.S. Geological Survey
Siamak Sattar	National Institute of Standards and Technology
Philip Schneider	Building Seismic Safety Council
Jon Siu	City of Seattle, Washington
Jonathan Stewart	University of California, Los Angeles
Mai Tong	Federal Emergency Management Agency
Jiqiu Yuan	Building Seismic Safety Council

The Project 17 Committee operated under consensus procedures and Roberts Rules of Order for small groups. Ronald Hamburger served as chair. Philip Schneider and Jiqiu Yuan acted as the project managers for the committee but did not vote. Mai Tong and Robert Hanson acted in an advisory role but did not vote. The USGS representatives performed substantial technical work to support the committee's activities and participated in meetings but did not vote. Steven McCabe and Siamak Satter served as government liaisons for National Institute of Standards and Technology but did not vote.

The committee held three meetings per year during 2016, 2017 and 2018, with many of these meetings timed to occur just prior to meetings of the Provisions Update Committee (PUC) so that interested PUC members could participate and to facilitate Project 17 reports to the PUC. The Project originally formed four working groups to perform detailed investigations associated within each of the four issue areas

identified above. A fifth working group on Seismic Design Category was formed to investigate the recommendations from the Stabilizing Mapped Values working group. The working group lead and team members are listed in Table 3. Ultimately, working groups 1 and 2 combined and produced joint recommendations.

Table 3: Project 17 Work Groups (WG)

WG 1: Acceptable Risk	WG 2: Deterministic Limits	WG 3: Stabilizing Mapped Values	WG 4: Multi-Period Spectral Values	WG 5: Seismic Design Category
Robert Pekelnicky (Chair)	C.B. Crouse (Chair)	Dan Dolan (Chair)	Charles Kircher (Chair)	Julie Furr(Chair)
Ibbi Amufti	Edward Field	Ben Enfield (co-chair)	David Bonneville	Bob Pekelnicky
Jack Baker	John Hooper	Jon Heintz	C.B. Crouse	Jim Harris
C.B. Crouse	Charles Kircher	Nicolas Luco	John Hooper	Bill Holmes
Jon Heintz	Nicolas Luco	Bob Pekelnicky	Nicolas Luco	Jon Siu
William Holmes	Sanaz Rezaeian		Sanaz Rezaeian	Dan Dolan
John Hooper	Jonathan Stewart		Jonathan Stewart	John Hooper
Charles Kircher				Paul Timko
Nicolas Luco				
Siamak Sattar				
Jonathan Stewart				

1.4. REPORT ORGANIZATION

The remaining sections of this report describe the issues addressed by each of the four working groups and the committee's recommended resolution for each. The committee prepared preliminary proposals, which were forwarded to the PUC for finalization and adoption into the 2020 *NEHRP Provisions*, and for use in forming its seismic design maps based on the 2018 USGS national seismic hazard model.

2. Acceptable Risk

2.1. ISSUE DESCRIPTION

The primary goal of seismic design provisions in U.S. building codes is to reduce the likelihood of life-threatening earthquake damage in buildings, considering the economic tradeoffs between enhanced protection and increased construction cost. Prior to publication of the 1997 *NEHRP Provisions*, U.S. building codes considered a single level of earthquake shaking having a notional 10% probability of exceedance in 50 years and sought to provide life safety protection for this shaking level. Such shaking has a mean recurrence interval of approximately 500 years. Based on the recommendations of Project 97, the 1997 edition of the *NEHRP Provisions* adopted design provisions that explicitly recognized two levels of earthquake shaking; Maximum Considered Earthquake (MCE) shaking and Design Earthquake (DE) shaking. Design provisions contained in the *NEHRP Provisions* were intended to minimize the potential for structural collapse under MCE shaking, and to minimize the potential for other life-threatening damage, such as failing nonstructural components, or loss of function in essential facilities for DE shaking, defined as having an intensity 2/3 that of MCE shaking.

The selection of this dual shaking level approach to design was influenced, in part, by a highly destructive earthquake that occurred in Armenia on December 7, 1988. This M6.8 earthquake occurred on a known, active fault near the cities of Spitak and Leninikan, both of which were heavily damaged by the earthquake. The earthquake caused collapse of hundreds of buildings and resulted in an estimated 25,000 to 50,000 deaths and perhaps more than 100,000 injuries. The Project 97 committee sought to develop design provisions that would protect American cities from similar disasters where rare, but foreseeable earthquakes, such as a repeat of the historic 1811-1812 New Madrid earthquakes or 1886 Charleston earthquake, would cause large numbers of building collapses and many fatalities. The shaking from such rare events was designated MCE shaking. The performance goal for MCE shaking was defined as avoiding large scale collapse of many buildings, a goal which admitted to substantially more damage than had been anticipated for buildings designed under earlier building codes, using more frequent earthquakes as design events. The Design Earthquake shaking level was developed as a means of continuing design for the levels of damage anticipated in earlier building codes, for more frequent events than the MCE. This also had the advantage of permitting continued use of other structural design parameters, such as the R factor, without major change. The 2/3 factor also permitted design ground motions in much of Coastal California, where seismic design had traditionally been a dominant design consideration, to remain at levels comparable to those in recent editions of the building code.

Determination of the shaking intensity expected at a site from an earthquake is a function of many factors including the earthquake magnitude, the characteristics of the fault rupture, the distance and direction of the rupture relative to the site, the geologic characteristics of the region and the nature of soils directly underlying the site. Before an earthquake occurs, it is impossible to determine many of these factors precisely, and even if these factors are known, the mathematical relationships used to compute shaking

intensity entail many uncertainties. Furthermore, how often earthquakes occur also affects the likelihood of life threatening damage, and is uncertain as well. Therefore, engineering seismologists use a process known as probabilistic seismic hazard analysis (PSHA) to estimate probabilities of future earthquake shaking intensities considering these various factors and uncertainties. PSHA starts with identification of the probable sources for future earthquakes in a region. These include known faults and areal sources. A recurrence model is developed for each potential source that indicates the probability of occurrence, over time, of earthquakes of different magnitudes. These recurrence relationships are based on past activity and the estimated or recorded magnitude, slip length and slip distance of past events on the fault, as well as understanding of the length or area of the fault rupture and the magnitude of crustal strain accumulation in the vicinity. Next, ground motion prediction equations (GMPEs) are used to estimate the intensity of shaking likely to occur at the site, given the occurrence of a specific magnitude earthquake at a specific location on a fault in the general vicinity of the site. GMPEs are formulated to capture the essential physics of wave propagation, with specific coefficients set based on regression analysis on ground motion records from past earthquakes and, where needed, computer simulations of earthquakes. GMPEs have functions related to the earthquake source, the effect of travel path as waves propagate through the Earth's crust towards a site, and the effects of local site conditions. The independent variables used in these functions include magnitude, fault type, rupture distance, sedimentary basin depth, and site characteristics as represented by an average shear wave velocity. GMPEs provide, for a given combination of these parameters, a distribution of potential ground shaking levels, which is described by a mean and standard deviation. Alternative credible GMPEs can give somewhat different results. In performing PSHA, USGS uses several GMPEs, weighting each according to its likely validity for the region and earthquake type. Finally, PSHA hazard curves are formed by summing over all earthquake sources, and all potential magnitudes and earthquake locations on these sources, the annualized frequency of exceeding ground shaking of a specified intensity at the site. By varying the intensity levels for which these calculations are performed, the probabilities of exceeding a range of ground motion amplitudes in a given time interval is obtained. Such relationships are commonly referred to as hazard curves. Such curves enable determination of the ground motion level for which the mean recurrence interval, in years (for example, 475 years), applies.

Levels of seismicity in the U.S. (i.e., how frequently earthquakes occur in time) vary considerably. In the Western U.S., strong earthquakes occur relatively frequently. A ground shaking return period of 500 years is significantly longer than the average time elapsed between earthquakes in such regions. Therefore, the feature of the ground motion that would cause its return period to be so large is that the amplitude is unusually strong for the size and location of the events. Because this ground motion is relatively rare, it can provide a safe level of shaking upon which to base the required strength of structures. Strong earthquakes in the Eastern U.S. occur less frequently. When Project 97 was conducted, earthquake scientists estimated that to adequately capture the shaking likely to be experienced by recurrence of the 1811-1812 New Madrid earthquakes, it was necessary to use return periods of more than 2,000 years. However, in the Western U.S., the intensity of shaking predicted by PSHA at return periods of 2,000 years or more can be very large, reflecting shaking intensities with very low probabilities of exceedance given the occurrence of events with considerable magnitudes. Engineers in the Western U.S. did not believe it was reasonable or

economically justifiable to design for such rare, and large, ground motions. As a result, there was conflict between the central/eastern and western parts of the US regarding the appropriate return period to use for the specification of design motions, with approximately 2000 and 500 years being considered appropriate, respectively, for these two regions. For the design motion maps, Project 97 considered it essential that a common return period be used nationwide, for reasons of fairness and practicality of implementation.

As a compromise, Project 97 recommended construction of design seismic value maps using PSHA and a 2%-50 year exceedance probability (return period of 2,475 years), but that where such shaking exceeded 150% of the levels used for design in the most seismically active regions of the U.S. under the then current building code, the values would be limited to a deterministically derived shaking level computed as a conservative estimate (84^{th} percentile, approximated as 1.5 times the median) of the shaking expected from a large magnitude earthquake event, termed a characteristic earthquake, on any known active fault in the region. As a result, the hazard levels associated with design seismic value maps in different part of the U.S. were no longer consistent, with much lower hazard levels in the western U.S. The rationale for this change was that construction in both the east and west would be designed for ground motion that had relatively small probability of being exceeded, should any considerable earthquake scenario occur.

Project 97 defined Design Earthquake (DE) shaking as having 2/3 the intensity of MCE shaking. The basis for this was belief that the design requirements in the then current building code incorporated an inherent margin, or factor of safety of about 1.5. That is, engineers believed that typical buildings designed to conform to the then current code requirements should be able to withstand ground shaking 150% more intense than that specified as a basis of design in those codes, with minimal risk of collapse. Selection of DE shaking at this level provided engineers confidence that reasonably foreseeable earthquakes would not result in collapse of large numbers of buildings while maintaining design force levels in the Western U.S. at levels comparable to those traditionally and successfully used for design in the past. As noted previously, use of the 2/3 value also provided desirable continuity in structural design parameters in active regions such as California.

In the years following Project 97, both earthquake science and structural engineering practice advanced. Structural reliability methods were developed that enabled computation of the theoretical probability a structure would collapse at different levels of ground shaking intensity. Evaluation of the collapse fragility of structural archetypes designed to modern code requirements suggested that typical well-designed structures would have a probability of collapse not exceeding about 10% when subjected to MCE shaking. For structures located on sites with MCE shaking having a 2% chance of exceedance in 50 years, this resulted in approximately a 1% chance of collapse in 50 years.

As part of the 2009 *NEHRP Provisions* Update Cycle, FEMA commissioned Project 07 to re-evaluate the basis for the national seismic design maps. Project 07 focused on two major issues, one being that earthquake scientists had recently developed a new generation of GMPEs, that significantly changed the values of ground motions at many sites, and the second being that some cities in the southeastern U.S.

refused to adopt the 2%-in-50-year MCE maps. The primary reason given for not adopting the 2%-in-50-year MCE maps was that these maps portrayed ground motions in the Southeast U.S. comparable to those shown in California, where the risk from damaging earthquakes was believed to be much higher, and that this essentially forced the southeastern U.S. to design for lower risk than was required for California.

In response to these concerns, Project 07 recommended a revised basis for the MCE maps consisting of ground motions that would result in a computed 1% probability of collapse in 50 years for buildings having typical fragility with 10% probability of collapse given the occurrence of MCE motion and not located on sites where the ground motion is deterministically limited. This enabled design ground motion values in the Western U.S. to remain at about the same levels as in prior building codes, while providing some reduction in design ground motions in the southeast and offering design for uniform collapse risk across the nation, except in places where MCE motion is deterministically defined.

Since the 1997 *NEHRP Provisions* were published, earthquake scientists have developed different understandings of expected recurrence intervals for large magnitude earthquakes in the New Madrid seismic zone. This meant that it might be possible to adopt MCE maps based on shaking corresponding to higher probability of exceedance (reduced return period) or collapse risk, while still maintaining building safety at an acceptable level. A principal advantage of such an approach is that it might permit the deterministic limits on calculated MCE motion at some locations in the Western U.S. to be eliminated, resulting in a more uniform risk basis for the entire nation.

Consideration was also given as to whether adjustment of the mapped values to obtain uniform collapse risk is appropriate. As described above, Project 07 recommended this, in part, to moderate the values of design ground motions in the eastern U.S., something which may not be desirable or necessary if an alternative return period is selected. Advantages of retaining the uniform collapse risk definition are that this would provide a measure of stability in the code-specified procedures. However, return to a uniform-hazard definition would simplify both the design shaking calculation procedures and engineers' ability to explain the ground motion basis to other stakeholders.

2.2. 2014 RISK BASIS

The national seismic design value maps adopted by the 2010 and 2015 *NEHRP Provisions* attempted to produce more uniform risk of collapse by adjusting the return period of mapped ground motion parameters to produce a 1% probability of collapse in 50 years, except at those sites, near major active faults, where the probabilistic motion was subjected to deterministic limits. While these maps produced more uniform risk of collapse, the deterministic limits and collapse-probability adjustments resulted in mapped ground motion values having widely different return periods around the U.S. Figure 3, produced by USGS, shows the return period of mapped MCE_R motion under the 2014 maps. As can be seen, MCE_R motion has return periods ranging from a few hundred years, at some sites in California, to more than 3,000 years at some cites in the central U.S.

The low return period of MCE_R motion at some sites in California is a product of the deterministic limits of ground motions near major active faults. Hazard and risk levels in all other portions of the US consider both the frequency of earthquakes in time and the different levels of ground motion that those earthquakes can produce. When an earthquake occurs frequently in time, it is a rare realization of the ground motion that would control the long return period (or low risk level) generally selected as the MCE target. As described previously, it was decided to not follow this process near active faults in California, instead assuming a particular percentile level of ground motion when a characteristic earthquake on the fault occurs. Because ground motions can be much larger than suggested by the selected percentile over long time horizons, national return period or risk maps (as in Figure 3) show strong discontinuities in California.

It is important to recognize that while the return period for MCE_R motion at some sites in California is quite low, these motions are computed so that there is a 16% chance they will be exceeded if any of the faults located near these sites produces a characteristic earthquake. The only reason the return periods for this shaking is low is that faults near these sites produce such characteristic earthquakes frequently. It is also important to note that while the return periods for MCE_R motion in other regions are significantly longer, on the order of a few thousand years, at most of these sites there is significantly more than a 16% chance that MCE_R motion will be exceeded, if the earthquakes that contribute most to their hazard occur.

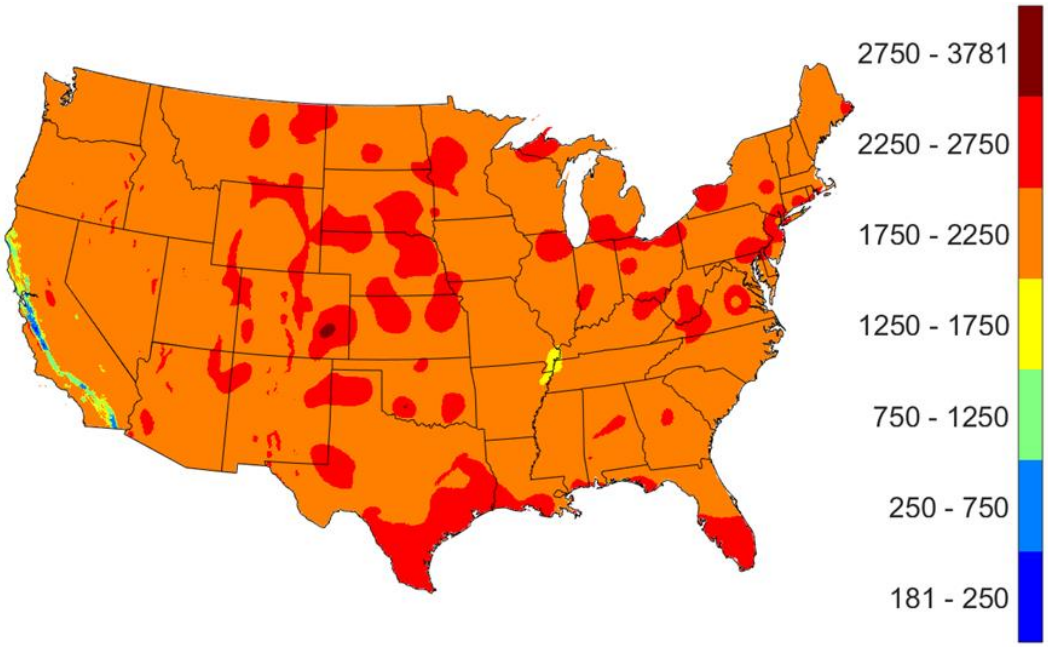

Figure 3: Return period in years of MCE_R shaking under 2014 national seismic design value maps

2.3. ALTERNATIVE RISK BASIS

The Project 17 team explored the potential use of alternative risk bases by evaluating the probabilities of collapse produced by selecting alternative probabilities of exceedance for MCE motion, and eliminating the deterministic limits currently imposed at near-fault sites in California. Preliminary study suggested that by selecting a return period of approximately 1250 years for MCE motion it would be possible to achieve an approximately uniform collapse risk of 1.5% in 50 years, without imposing deterministic limits on near-fault sites.

In reducing the return period for MCE shaking, or increasing probability of collapse in 50 years, the level of ground motion provided at the MCE level would be reduced. The committee was concerned that such reductions could leave some cities in the central and eastern U.S. vulnerable to large numbers of collapses should a characteristic earthquake occur on any of the sources that dominate the hazard in those regions. To moderate this concern and maintain stability in design ground motions with a redefined MCE, an alternative was discussed whereby the 2/3 factor would be modified to some larger value, with that value selected through committee deliberation.

Figure 4 shows the locations of major source zones in the central and eastern U.S. that generally dominate the hazard of eastern cities, and Table 4 presents their effective (across alternative models of each source) characteristic magnitudes and corresponding return periods. For several cities in the central and eastern U.S., USGS computed the probability that shaking from a major event on such sources would exceed MCE motion defined at different return periods. This was done by disaggregating the seismic hazard at each return period (using https://earthquake.usgs.gov/hazards/interactive/) and obtaining the so-called mean epsilon value across all events. Figure 5 presents a plot of the value for S_s, the spectral acceleration at a period of 0.2 seconds on a site with reference site class conditions, for Charleston, SC; Chicago, IL; Memphis, TN; New York, NY and St. Louis, IL. Figure 6 presents a similar plot for the same cities for S_1, the spectral acceleration at a period of 1 second on a site with reference site class conditions. In these plots, the horizontal axis is the return period of MCE shaking for the value of S_s or S_1. The vertical axis along the left-hand side of the plots is the mean epsilon from hazard disaggregation. Along the right-hand side of the plots is the corresponding probability of non-exceedance of the motion, given the controlling earthquake. As an example, in Figure 5, at a return period of 2,475 years, the value of S_s for Memphis is about a 70th-percentile value, meaning that there would be a 30% chance that a repeat of the 1811-1812 New Madrid series of events would produce ground motion exceeding the 2,475-year value.

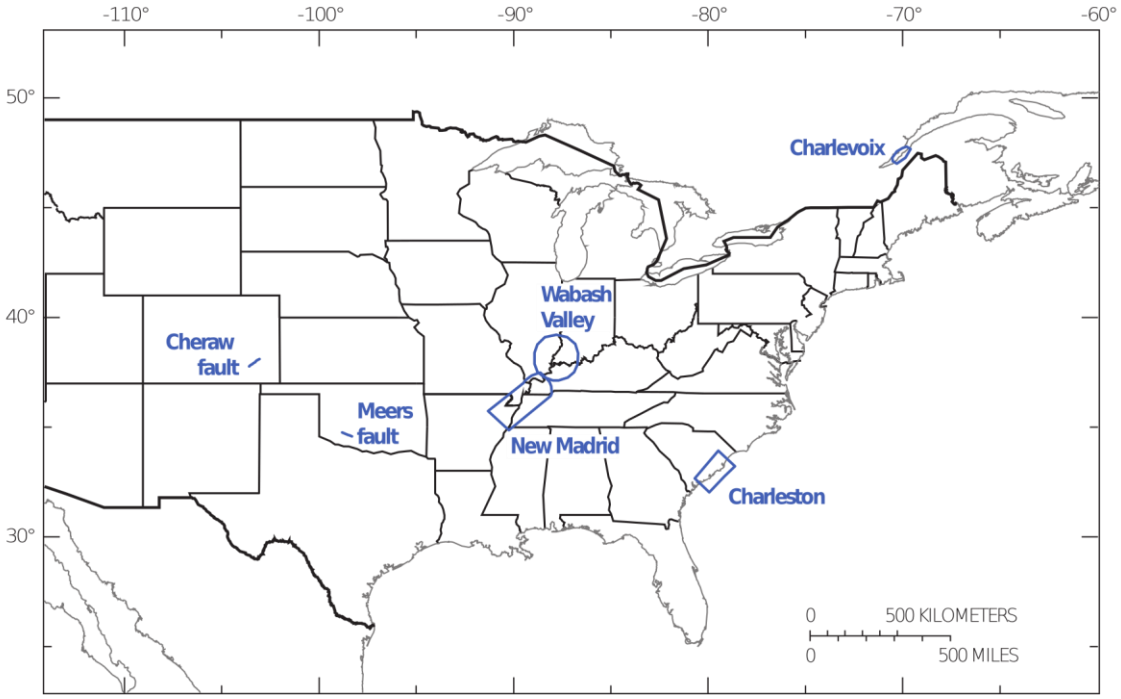

Figure 4: Major seismic source zones in the central and eastern U.S.

Table 4: Characteristic Magnitudes and Return Periods for Central and Eastern Sources

Source	Effective M_{max}	Effective Return Period
Cheraw	7	9500 yrs.
Meers	6.9	2100 yrs.
New Madrid	7.5	~500 yrs.
Wabash	7.5	5900 yrs.
Charleston	7.1	530 yrs.
Charlevoix	7	730 yrs.

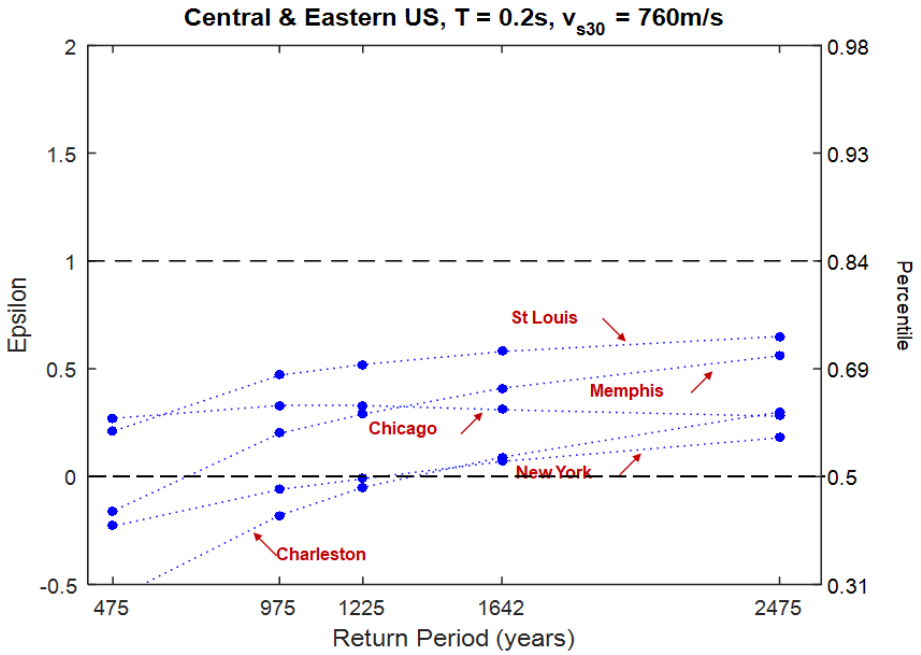

Figure 5: Probability of Non-Exceedance of MCE S_S values Given Controlling Earthquakes from Hazard Disaggregation, based on Mean Epsilon across all Events

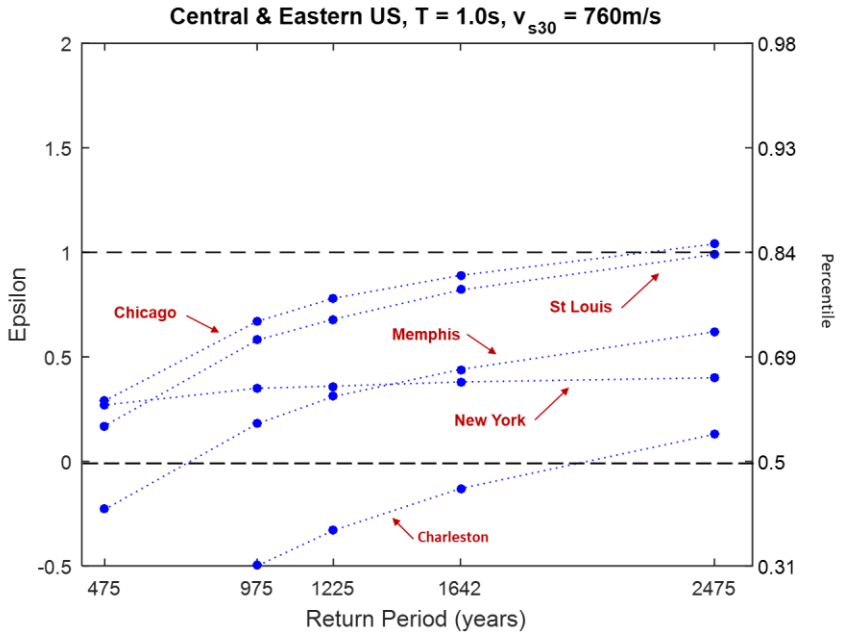

Figure 6: Probability of Non-Exceedance of MCE S_1 values Given Controlling Earthquakes from Hazard Disaggregation, based on Mean Epsilon across all Events

Examining Figures 5 and 6, the committee observed that with an MCE defined as having a return period of approximately 1,200 years, most sites in the central and eastern U.S. would have a relatively low probability (approximately 30% or less) of MCE ground motions being exceeded if the controlling earthquake

occurred. However, it is possible to identify a few locations where this is not the case, including Charleston, SC and New York, NY. For those locations, there is approximately a 50% chance that short period motion, represented by S_S, would exceed MCE motions if the controlling event occurred. For long period motion, shown in Figure 6, sites in Charleston would have greater than a 50% chance of having their ground motion exceeded for a return period less than about 2,000 years. Given this, the committee determined that a reduction in return period for MCE motion would result in an unacceptable increase in risk to some central and eastern American cities and should not be undertaken. This determination was made despite the possibility of maintaining general consistency in design ground motions through adjustment of the 2/3 factor as described above.

2.4. PROJECT 17 WORKSHOP INPUT

A Project 17 Workshop on Seismic Hazard Mapping was conducted on April 11, 2017 to solicit input on this issue from structural engineers, building officials, and members of earthquake community. The workshop attendees, in groups balanced by expertise and background, discussed and opined on the following questions. The 62 P17 Workshop participants seemed to be willing to make changes if there was a good reason to do so, but otherwise would stay with uniform risk with deterministic limits. Detailed workshop discussion and participants can be found at the *Project 17 Workshop Proceedings* published at the BSSC website[1].

Q1: Is the community willing to accept a major change in the mapped values?			
Group Vote	Yes	No	Not Voting
Group 1	11 (only if the change can be justified)	5	6
Group 2	2	12	8
Group 3	8 (major change is acceptable with compelling reasons)	9	1

Q2: Is it desirable to eliminate the "deterministic caps" and place the entire country at the same risk level?			
Group Vote	Yes	No	Not Voting
Group 1	17 (eliminate caps)	1	5
Group 2	6 (eliminate caps)	10	5

[1] *Project 17 Workshop Proceedings*

https://cdn.ymaws.com/www.nibs.org/resource/resmgr/bssc2/20170411P17WorkshopProceedin.pdf

Q2: Is it desirable to eliminate the "deterministic caps" and place the entire country at the same risk level?			
Group 3	9 (see the need to eliminate, but the change could create additional problems)	3	6

Q3: Uniform risk of collapse or uniform hazard?			
Group Vote	Uniform Hazard	Uniform Risk	Not Voting
Group 1	14 (easy to understand and explain)	2	7
Group 2	2	12 (no change to the current approach)	7
Group 3	0	8	10

Q4: If uniform risk of collapse is to be maintained, can this be done approximately, while maintaining uniform hazard?			
Group Vote	Yes	No	Not Voting
Group 1	14	2	7
Group 2	3	10	8
Group 3	18	0	0

Q4: If ground motions are reduced in the mid-south and east (because big earthquakes happen more often than previously estimated) is this acceptable?			
Group Vote	Yes	No	Not Voting
Group 1	17 (but need to provide a rationale)	0	6
Group 2	2	12	7
Group 3	0	18	The Rest

2.5. RESOLUTION

On the basis of the above, the Project 17 Committee recommends that national seismic design value maps continue to be developed on the basis developed by Project 07, as being ground motion that produces a 1% risk of collapse in 50 years for structures having 10% conditional probability of collapse given the occurrence of MCE_R shaking, except at those sites where such motion exceeds the deterministic lower limit, as defined in the 2015 *NEHRP Provisions*. At sites where the 1%-in-50-year collapse risk motion exceeds the deterministic limit, the motion should be limited to the maximum of the deterministic limit or

the 84th-percentile motion resulting from a defined deterministic event on nearby active faults. Selection of the deterministic events is discussed in Section 3.

It is worth note that the decisions described above did not have the unanimous support of the committee. Some committee members preferred alternative approaches including adopting reduced return periods for MCE shaking and grading the acceptable collapse risk on sites near major active faults as an alternative means of controlling the intensity of design shaking near these active sources. While specific proposals for these alternative approaches were not balloted, the proponents for these alternative approaches were provided opportunity to present the concepts, the concepts were balloted and did not obtain consensus. Part 3 White Papers on these concepts are proposed for inclusion in the *NEHRP Provisions*, to document the ideas and permit potential future reconsideration.

3. Deterministic Limits

3.1. ISSUE DESCRIPTION

Under the 1997 and later editions of the *NEHRP Provisions*, ground motions in most locations are determined using PSHA with the key parameters S_s and S_1 determined at either a defined hazard level (2,475-year return period for 1997 through 2003 editions) or collapse risk level (1%-in-50 years for 2009 and 2015 *Provisions*). Rare hazard levels, on the order of 2,475 years were initially selected to assure that ground motion values specified for sites subject to large earthquakes on faults that rupture infrequently, such as the source near Charleston, S.C., would be adequately captured. However, at sites where the hazard is controlled by faults that tend to produce earthquakes at or near their maximum magnitudes at intervals on the order of a few hundred years, the resulting ground motions tend to have large amplitude, driven primarily by the uncertainty inherent in the ground motion for a given earthquake.

Figure 7 illustrates the uncertainty in ground motion prediction for a given earthquake. The figure is a plot of median-component peak horizontal acceleration obtained from recordings of M7 earthquakes in active tectonic regions, including the western U.S., as function of distance of the ground motion recording instrument from the fault rupture surface. In this plot, at 10 km, recorded data ranges from 0.15 to 1.0 g. On the plot, the predictions of a ground motion model are shown that provides a median value of ground motion as a function of distance, as well as plus and minus 1 sigma (σ) functions, representing 16^{th}- and 84^{th}-percentile estimates. The significant scatter shown in this plot is because only two parameters, magnitude and distance are represented in this regression analysis. A portion of the residual scatter is due to other parameters that have not been taken into account, including site class at the recording instrument and fault type. Modern GMPEs used to develop the national seismic design value maps account for these and other factors, but still have significant uncertainties.

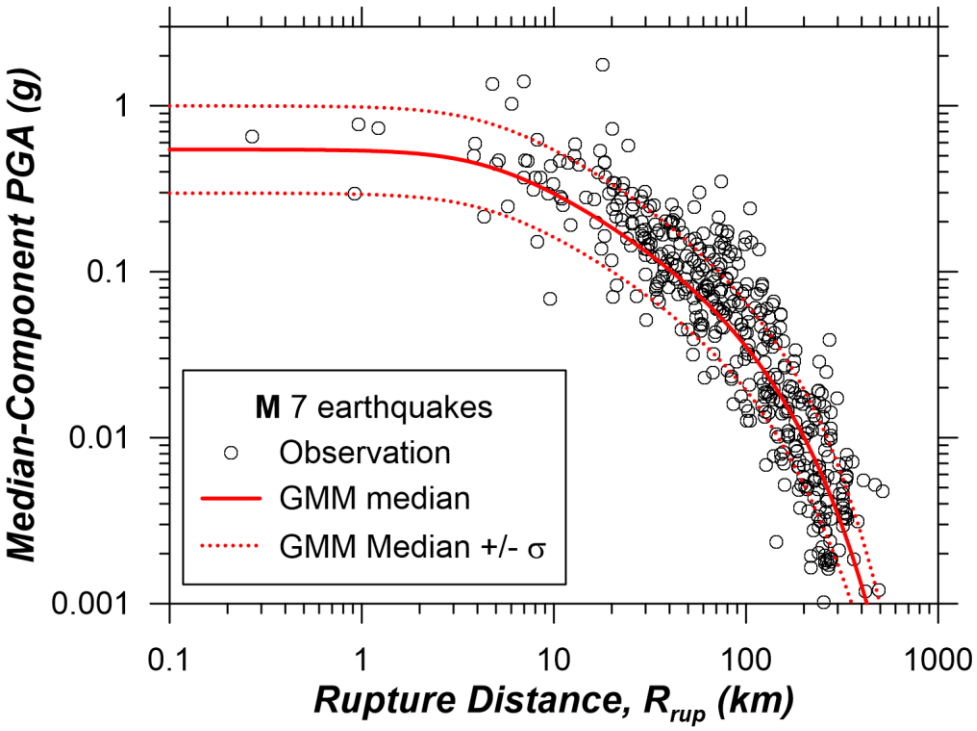

Figure 7: Peak ground accelerations (PGA) from magnitude 6.9-7.1 earthquakes in active tectonic regions from observations and as predicted by ground motion model (GMM) (Boore et al. 2014). Data from eight earthquakes from California, Japan, New Zealand, Italy, and Yugoslavia are presented (obtained from NGA-West2 database).

Equation 6 presents an approximate relationship between the return period for a motion (RP_{GM}), the return period of the earthquake causing that motion (RP_E), and the percentile of the ground motion given that earthquake (GMP).

$$GMP = 1 - \frac{RP_E}{RP_{GM}} \quad (6)$$

This equation illustrates that if ground motion with a 2,500-year return period is calculated for a site that experiences this motion because of a large magnitude earthquake that has a return period of 250 years, the ground motion represents a 90th-percentile estimate of the motion from the earthquake event. This is representative of the conditions associated with sites at some active faults in California. While a 90th-percentile motion does not sound like it is inordinately conservative, it does represent approximately 1.3 standard deviations above the mean, which given the large value of the standard deviation, is a large number, and often, ground motion values that are much larger than those that have traditionally been used for design.

Members of the Project 97 team felt that it was unreasonable to design for such large ground motions and further, that the large uncertainties in ground motion from the GMPEs of the day were due to the few parameters included in the data regression, rather than real potential for such very large values. Therefore, Project 97 crafted rules that limited the ground motion parameters determined by PSHA, when these

values became very large, to the value determined for 1 standard deviation above the median (84th percentile) for a characteristic earthquake on any major active fault in the region. The characteristic earthquake represented the best estimate of magnitude seismologists assigned to the fault, based on its characteristics and past earthquake history.

The USGS uses consensus-based models on fault magnitude recurrence models developed by regional working groups. In 2014, the regional working group that develops the consensus models for California published the third edition of its Uniform California Earthquake Fault Rupture model (UCERF3). Unlike prior editions, this model did not include the concept of characteristic earthquakes with limiting magnitudes on faults, and instead adopted a model that admitted to very large magnitude earthquakes on faults, albeit at low probability, resulting from simultaneous rupture in combination with other regional faults. Lacking a consensus value for characteristic earthquakes on faults the USGS could not compute deterministic limiting values. The purpose of this issue (deterministic limits) was to provide USGS with alternative rules for establishing limits on ground motions values near major active faults.

3.2. ALTERNATIVE APPROACHES

The Project 17 team explored several alternatives to resolve the issues associated with loss of characteristic earthquake definitions. These included: selection of an alternative acceptable risk value that would preclude the need for deterministically limited ground motions; use of a graduated risk model near major active faults; and selection of a characteristic earthquake magnitude through examination of the hazard disaggregation. The first of these approaches is discussed in the previous chapter. As described in that chapter, the project team elected to retain the 1%-in-50-year collapse risk model for determination of mapped motions at sites other than those near major active faults.

The concept of graduated risk was offered as an alternative approach to using deterministic estimates of ground motion near major active faults. The premise behind this approach is that the effect of limiting mapped ground motion values to deterministically computed values is to increase the accepted risk of collapse at sites where this is done, as compared with typical sites. This is because the effect of establishing limits on the ground motion at these sites is to reduce the return period of the motion at such sites, and consequently, accept greater annual probability of collapse than that used as the basis for ground motions at typical sites. Figure 8, prepared by USGS illustrates the computed probability of collapse in 50 years for sites in the conterminous U.S. under the 2014 edition of the national seismic design value maps. As can be seen, the collapse risk is a uniform 1%-in-50 years throughout the conterminous U.S. except in a band generally associated with the Pacific Plate and North American Plate boundaries, defined by the San Andreas fault system in California. At sites near this plate boundary, where mapped values of motion parameters are deterministically limited in the 2014 maps, the computed collapse risk reaches values nearly 10 times the target level in the deterministic zones (9.45% in 50 years).

Proponents of this approach noted that lacking consensus seismologic opinion as to the characteristic magnitudes of earthquakes on these major active faults, selection of a characteristic magnitude is arbitrary

and really an engineering decision rather than scientific one. Since the effect of this engineering decision is to accept an increased risk of collapse relative to that which is accepted on typical sites, the arbitrary engineering selection should be on the acceptable risk side. Under this approach, the rules would be crafted such that where PSHA adjusted to a 1%-in-50 year collapse risk produced values judged as excessive, a higher collapse risk would be used, and that this higher collapse risk could be graduated from 1%-in-50 years at the margins of the limit zone, remote from the fault, to the higher acceptable value adjacent to the fault.

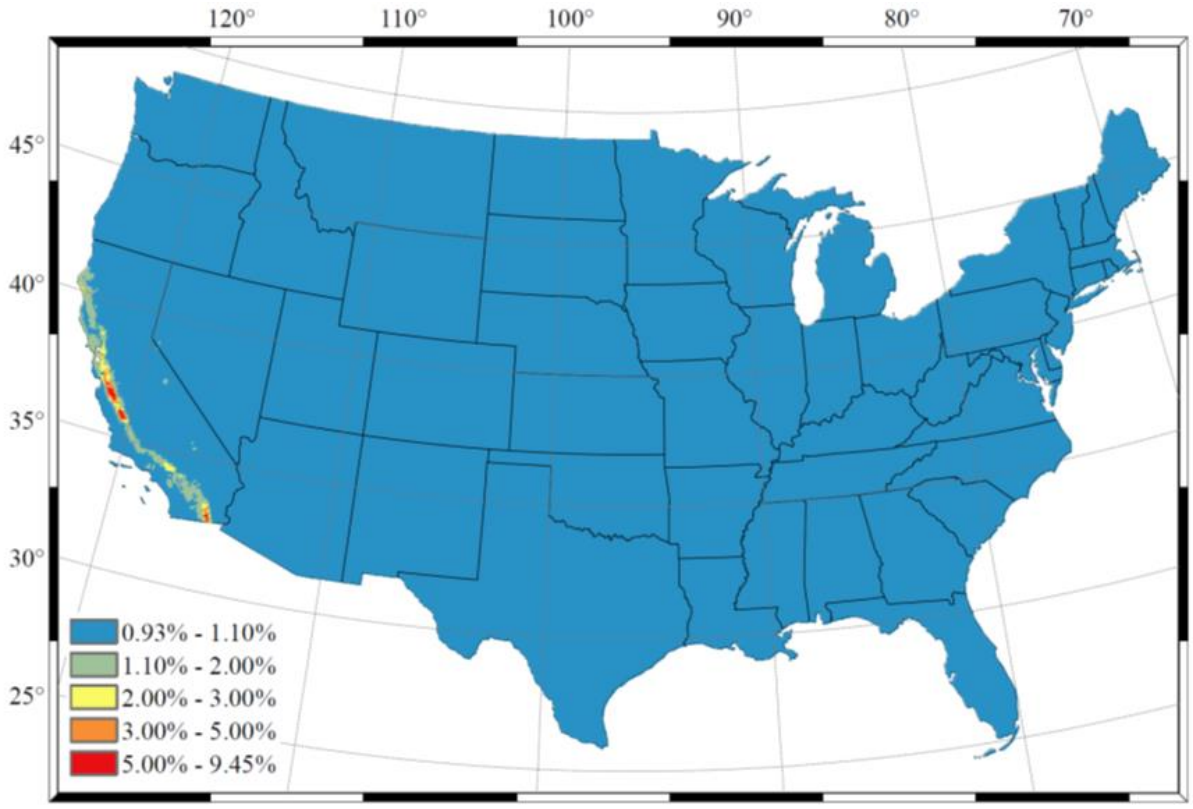

Figure 8: Collapse risk probability in 50 years for 2014 national seismic design value maps

Ultimately, this approach was not accepted by the Project 17 committee because the committee was not willing to specify higher risk targets near active faults. The committee prefers that the risk near active faults be an essentially uncontrolled outcome of a magnitude selection process, which is a continuation of past practice.

As a result, the committee defaulted to a modest adjustment of established practice, whereby limiting magnitudes are specified on these faults by using disaggregation of the hazard at return periods associated with MCE_R motion. Some members of the committee are of the opinion that the resulting magnitude definition would be useful in discussing the level of protection offered by the *NEHRP Provisions* with nontechnical audiences.

3.3. RESOLUTION

The Project 17 team recommends the following procedure for limiting ground motion at sites where computed ground motion values that target a 1%-in-50-year collapse risk exceed the deterministic lower limit as defined in *ASCE 7-16* Section 21.2 (e.g., values of 1.5 and 0.6g at periods of 0.2 and 1.0 seconds, respectively, for Site Class B): At those sites, the deterministic limit should be taken as the maximum of:

1. The deterministic lower limit value.
2. The maximum 84th-percentile value of the ground motion parameter amongst all scenarios (earthquake faults and corresponding mean magnitudes) from disaggregation at the return period of the risk-targeted value.

This procedure is essentially identical to that in *ASCE 7-16*, except that it replaces the characteristic earthquakes with scenario earthquakes from disaggregation. The USGS computed this recommended deterministic cap for several locations controlled by deterministic limits in the *ASCE 7-16* maps and found that the resulting values using the new procedure are mostly similar to those computed using the prior procedure, and do not require the selection of arbitrary characteristic earthquakes.

The Project 17 committee also recommends that USGS publish a catalog listing the scenario events that are used to cap ground motion on the maps to enable communication as to the protection provided by the Provisions.

4. Stabilizing Mapped Values

4.1. ISSUE DESCRIPTION

Prior to 1997, building codes regulated seismic design through reference to seismic zonation maps. These maps divided the nation into distinct seismic zones related to the frequency and intensity of earthquake shaking experienced in the regions in the past and anticipated to occur in the future. Based on the seismic zone a structure was assigned, it was determined whether seismic design was required, the types of structural systems that would be permitted, the required detailing, and, the required strength. The zones were generally based on political boundaries as well as considerations of past seismicity and remained relatively stable over time with few changes made to the boundaries between zones from one edition of the building code to the next.

A primary advantage of this system of seismic zonation was that, because the seismic zones were generally stable and covered broad, contiguous regions, the zone-related construction requirements became regularized, with engineers, contractor and building owners all understanding the level of seismic protection required and how to achieve it. A principal disadvantage of the zonation system was that the zones were geographically large and incorporated sites with widely variant fault distances and soil conditions, resulting in more onerous design requirements than necessary on some sites, and potentially less protection than appropriate for other structures. The zones also produced discontinuous design values across zone boundaries.

With the adoption by building codes of seismic requirements based on the 1997 and later editions of the *NEHRP Provisions*, the function of seismic zones were replaced by assignment of structures to Seismic Design Categories, determined based on computed spectral response values considering both site class and occupancy. This had several important effects. First, while seismic zones provide similar design and construction practices over broad regions, under the *NEHRP Provisions*, communities could encompass structures assigned to multiple Seismic Design Categories, depending on the variability of site class conditions within the community as well as the intended occupancy of the structures. Communities which traditionally had designed structures to the moderate requirements specified for zone 2 now had to deal with some structures requiring the design and construction practices associated with zone 3. This created problems for designers and contractors who found it difficult to become familiar with and properly execute the more rigorous seismic requirements. This is counterbalanced by the fact that the Seismic Design Category approach did allow structures of low occupancy category located on firm sites to be designed and constructed more economically than structures assigned to higher occupancy category and located on soft soil sites, effectively, requiring additional seismic protection where the intensity of future shaking justified it.

A second problem associated with the use of Seismic Design Categories is that whereas the boundaries of seismic zones remained relatively static from one code edition to the next, the values of mapped ground motion parameters tended to change from one code edition to the next, based on updates to the maps.

The updated maps would incorporate latest scientific understanding as to the locations and activity rates of seismic sources as well as the GMPEs used to compute ground motion parameters, resulting in modest changes in values on almost all sites, and significant changes at some sites, with changes in one map edition increasing the values while the next edition reduced these again. At sites located near the boundaries of Seismic Design Category assignment for common soil conditions in a region, this would result in changes of Seismic Design Category from one code edition to the next. These conditions were unsatisfying to engineers, building officials and building owners alike because it created the impression that code requirements were not well founded.

In addition to the public perception of poorly founded requirements, fluctuating Seismic Design Category classifications along boundaries necessitated that both engineers and contractors relearn varying code requirements between successive code versions, in lieu of building on already established knowledge and experience. Jurisdictions frequently skipped adoption of successive code versions to avoid the increase in seismic code requirements where possible, fully expecting the following code version would revert back to lower seismic requirements. Both the unfamiliarity with newer seismic detailing and the ability to pick and choose code versions based on variability in seismic detailing requirements along boundaries, raises the specter of significant structural life-safety concerns where important key technical changes are willfully avoided. These varying conditions also tended to make code enforcement more difficult as code officials work to understand new code requirements.

Finally, the economic impact of varying Seismic Design Category boundaries becomes a highly compelling incentive for jurisdictions to find ways to avoid more restrictive code requirements. From developers opting to relocate to neighboring jurisdictions with more seismic friendly local amendments to an increase in project fees if a code change is anticipated before permitting, development decisions are highly influenced by the governing code documents.

Although the fluctuation of Seismic Design Category boundaries occurs over a small geographical area within the United States, impacted jurisdictions encompass large population centers such as Memphis, St. Louis, and Charleston. When such regional population centers amend and reduce seismic code requirements for other than technical reasons, they provide justification for smaller jurisdictions to follow suit. The end result is less than national model building code level performance across significant regions of the country.

Under this issue, Project 17 sought to find means to stabilize the values of specified ground motions and design requirements.

4.2. ALTERNATIVES EVALUATED

The project team evaluated two primary means of stabilizing the design requirements: (1) using a weighted average of mapped values over several recent map editions; and (2) assigning Seismic Design Categories using separate seismic zonation-like maps.

Under the weighted average approach, the values published on new editions of the maps would use an averaged value obtained from the most current seismic design maps (before averaging) together with those of prior maps. This approach was conceived in recognition of the fact that fluctuations in the values of mapped parameters from one edition to the next are sometimes the result of the rapidly advancing scientific understanding of earthquake sources and ground motions, and in some cases, as new data and understanding are developed, they counter effects of prior developments. As an example, the 2009 *NEHRP Provisions* used (via USGS) the then-recently developed PEER NGA GMPEs to develop ground motion parameters in the Western U.S. Project 07 members were surprised that some ground motion parameters derived using these new GMPEs were markedly lower than those previously portrayed on the maps. This circumstance led directly to the adoption of the maximum horizontal component of ground motion as the basis for seismic design, which increased design ground motions back to the approximate levels that had existed in earlier versions of the Provisions. In a later update cycle, new GMPEs were used that increased ground motions again in some areas where motions had been reduced with the previous set of GMPEs. Because the maximum component definition remained in effect, this had the effect of increasing design ground motions substantially in some areas.

Using the weighted average approach would tend to dampen out the fluctuations in mapped values from one edition to the next. If a substantial increase in design motion values were justified in a region, for example, due to discovery of a previously unknown fault, the values would only increase slightly in the first new edition of the maps, since the model with this fault present would be averaged with models that did not include the fault. Over a period of several cycles, the full increase in design values associated with the new fault would be achieved. For other regions where fluctuations in mapped values are due to factors that are later countered by additional scientific understanding, the averaging approach would damp out changes that tend to contravene each other over several cycles.

The committee generally thought that this was a valid approach, although some members believed that it would be inappropriate to continue to use models that were known to be invalid, due to the discovery of new data, as a basis for the maps. Ultimately this approach was not adopted because the adoption of multi-point spectra, discussed in the next chapter of this report rendered the approach impractical for use in this cycle. It may have applicability in future cycles however, as a means of providing stability.

The second alternative reviewed in detail is to decouple the assignment of Seismic Design Category for a structure based on the computed values of ground motion for a site and instead, assign Seismic Design Category based on a zonation map that would be constructed using the procedures specified by the current edition of the *NEHRP Provisions* assuming a default site class condition. Figure 9 is such a map, applicable to Risk Categories I, II and III, produced by USGS, using the 2014 national seismic hazard model. A similar map would also be produced for Risk Category IV structures, including SDC F.

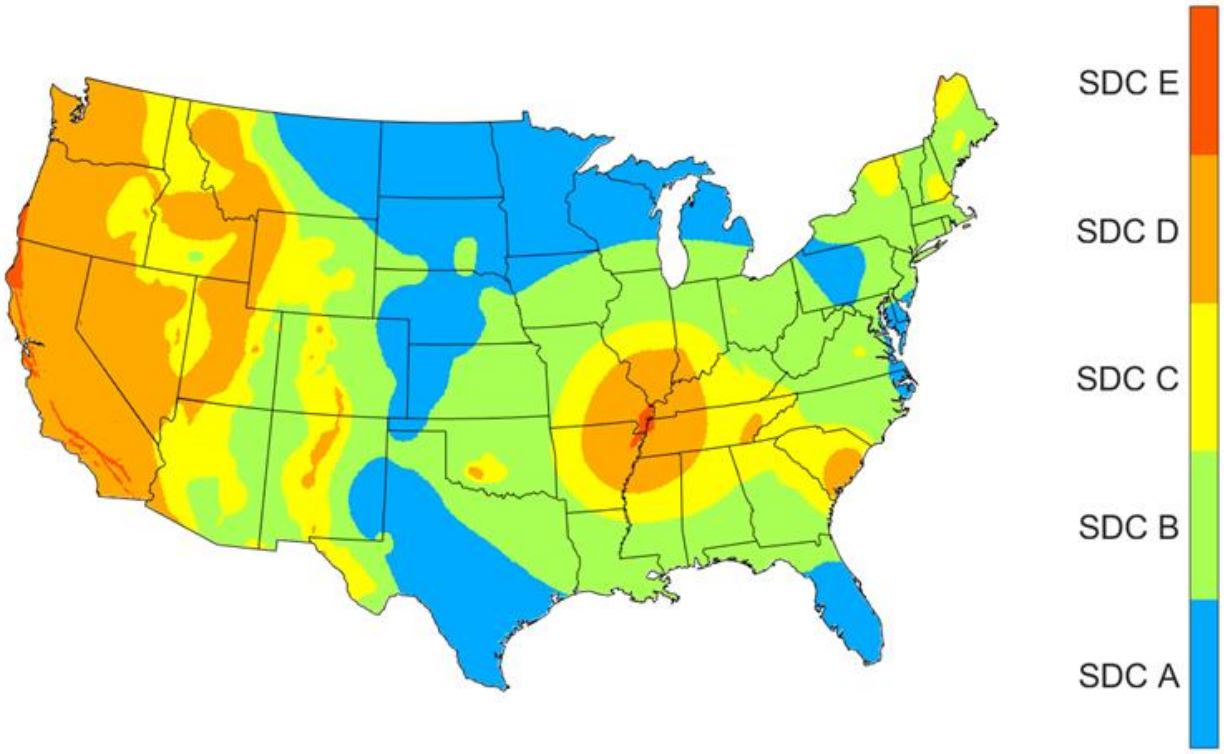

Figure 9: Seismic Design Category (SDC) Map for Risk Category I, II and III structures, 2014 seismic hazard model and default site class conditions

When future editions of the map are published, code developers would be able to compare the locations of Design Category boundaries and explore the reasons why the boundary had shifted, in consultation with the USGS. If the reason for shifting for a boundary was judged to be within the uncertainty of seismic hazard modeling and design mapping, and thus potentially subject to future change, the code adoption committee (e.g., the BSSC PUC) could elect to leave the boundary in the same location, providing a measure of stability. While Seismic Design Category assignments would be subject to review by the code adoption committee, actual ground motion values would still fluctuate as deemed necessary by USGS in response to updated scientific methodologies.

The principal advantages of assignment of design category based on a map such as that shown in Figure 9 is that it permits uniformity of practice within broad geographic regions; and it enables future code developers to preserve design and construction practices in a region, where it is believed the evidence for change is not compelling. The principal disadvantage is that by not allowing consideration of actual site class when determining Seismic Design Category, provisions based on such a map will require some structures to be designed more conservatively, for a higher design category than would otherwise be permitted. Similarly, for structures on sites with Site Class E or F soils, it is possible that the structure would be assigned to a lower Seismic Design Category than would otherwise have been possible.

4.3. RESOLUTION

The Project 17 committee recommends that future editions of the *NEHRP Provisions* assign Seismic Design Category through reference to a Seismic Design Category Map constructed by the USGS using the procedures for category assignment contained in the then current *NEHRP Provisions*, but assuming a default Site Class. If future Provisions adopt alternative procedures for Design Category assignment, these would form the basis for the SDC maps. Prior to adopting the SDC maps based on the latest USGS seismic hazard model, the Provisions Update Committee should review the design category boundaries to assure they are adequately stable and are appropriate for design.

5. Multi-Period Spectral Values

5.1. ISSUE DESCRIPTION

During the closing months of the 2015 PUC cycle, a study was undertaken of the compatibility of current Site Class coefficients, F_a and F_v with the NGA GMPEs used by USGS to produce the seismic design value maps. During this study it was discovered that the standard spectral shape derived from the S_{DS}, S_{D1}, and T_L parameters does not adequately represent the spectra of real ground motions on soft soil sites (Site Class D, E, or F) produced by large magnitude events. As shown in Figure 2, the standard spectral shape includes (1) a domain of constant response acceleration (S_{DS}) that extends from periods of about 0.2 seconds through period T_s, which typically has a value less than 1 second; and (2) a domain of constant response velocity, which takes the form of a hyperbolic function of the form $S_{DS}(T) = S_{D1}/T$, extending to period T_L which has values ranging from 6 seconds to 12 seconds, depending on the controlling fault magnitude in a region. As an example, Figure 10, shows 84th-percentile acceleration response spectra derived using NGA West2 GMPEs for a site distance of 5 km from an M8.0 earthquake on a strike slip fault for site classes A through E. These spectra are typical of MCE spectra for sites on the San Francisco Peninsula and other locations along the San Andreas Fault. As can be seen the spectra for site class A, B and C conform reasonably well to the standard NERHP spectral shape. However, the spectrum for Site Class D soils does not exhibit a hyperbolic relationship at periods of 1 second and greater and the spectrum for Site Class E soils does not reach the peak spectral value until periods substantially more than 1 second. The extent to which the spectra will vary from the standard NEHRP shape is dependent on earthquake magnitude, Site Class, fault distance and epsilon (the number of standard deviations above or below the man of the ground shaking).

As an interim solution to this problem, the 2015 *NEHRP Provisions* required site-specific seismic hazards study for design of structures with periods exceeding 1 second located on sites classified as Site Class D or E, with an exception that permitted the use of conservatively amplified (by a factor of 2) spectra in some cases. Under this issue, Project 17 was to develop an approach for derivation of spectra, and values for the S_{DS} and S_{D1} parameters for structures on such sites, that will not require site-specific seismic hazards study, or excessively conservative modifications of the NEHRP spectrum.

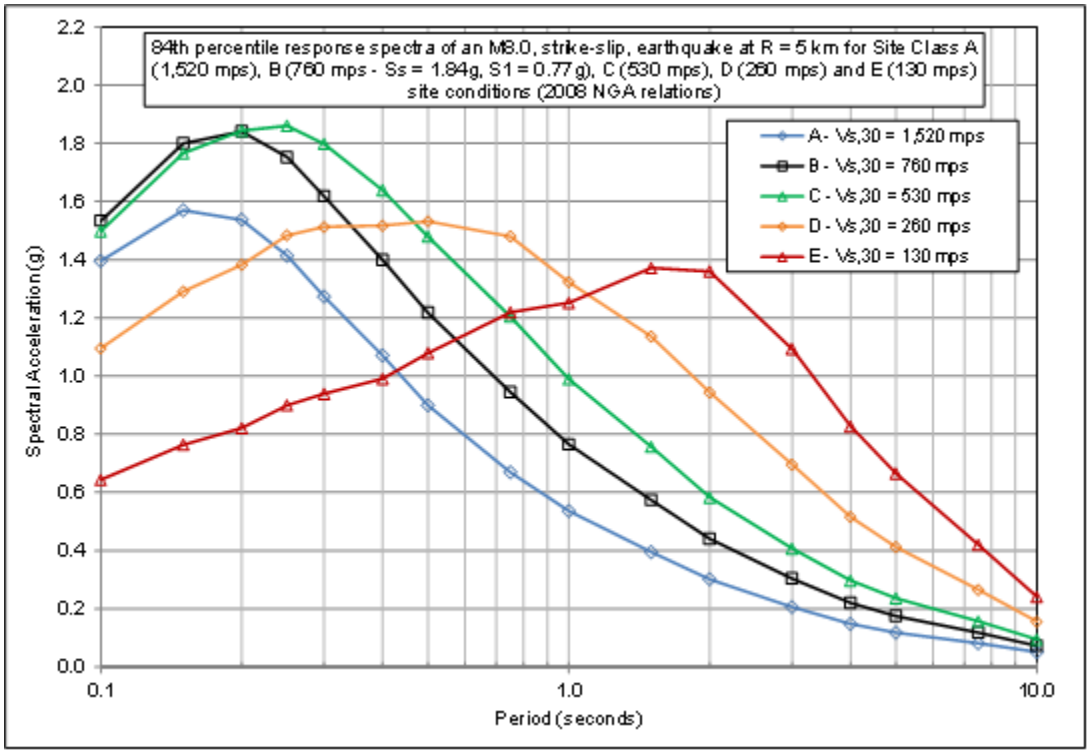

Figure 10: 84th-Percentile spectra shape for M8 earthquake on various Site Classes (Kircher)

5.2. RESOLUTION

The original NEHRP spectral shape was developed in an era when the available GMPEs were based on a relatively small set of ground motion recordings. These early GMPEs, commonly termed attenuation equations, did not directly account for site class effects and often did not provide values of spectral response acceleration at periods longer than 1 second or so. Therefore, to provide response spectra for seismic design, it was necessary to adjust a standard spectral shape with site class coefficients. However, over the past 15 years, the Pacific Earthquake Engineering Research Center, in partnership with the USGS, SCEC and other research organizations, has developed several sets of next-generation GMPEs that directly account for Site Class, through a shear wave velocity term, and permit computation of spectral acceleration parameters at a range of periods, extending to long periods.

Given this enhanced capability, Project 17 recommends the following:

1. Rather than producing maps of the value of the S_s and S_1 ground motion parameters, as has been done since the 1997 *NEHRP Provisions*, for each of the gridded data points from which the USGS constructs the maps, USGS should instead create a database of MCE_R spectral acceleration values at periods ranging from 0.2 to 10 seconds for each Site Class. Spectral values for sites other than the gridded data points will be obtained by geographic interpolation between the nearest gridded values.

2. Because the difference in spectral shape on sites having site class B, C, D and E are very substantial, the number of site classes referenced by the NERHP Provisions should be expanded to include site classes that are intermediate to these, creating new Site Classes BC, CD and DE.
3. For determination of response spectra, for use in modal analysis, or response history analysis, engineers will pull down the multi-period spectral values, through a web-based application, that will geo-reference the data base with input of latitude, longitude and Site Class. Alternatively, site-specific seismic hazard analysis can be performed as currently permitted. Limits on the values obtained from site-specific seismic hazard analysis, as a fraction of the USGS derived values, will be maintained.
4. The Site Class Coefficients, F_a and F_v will be deleted from the Provisions and will not be used any longer.
5. The *NEHRP Provisions* will continue to incorporate an Equivalent Lateral Force procedure tied to spectral parameters S_{DS} and S_{D1}. These will continue to be taken, respectively, as 2/3 of the MCE values of parameters S_{MS} and S_{M1}, where S_{MS} will be taken as 90% of the maximum value of MCE_R spectral response acceleration between periods of 0.2 to 5 seconds inclusive; and, S_{M1} will be taken as follows:
   a. For sites with values of $\boldsymbol{v_{s,30}}$ greater than 1,200 ft/sec (366 m/s) S_{M1} shall be taken as the maximum value of the quantity $T{\cdot}S_a$ for periods ranging from 1 sec to 2 sec, where S_a is the computed MCE_R spectral acceleration at these periods.
   b. For sites with values $\boldsymbol{v_{s,30}}$ less than 1,200 ft/sec (366 m/s) S_{M1} shall be taken as the maximum value of the quantity $T{\cdot}S_a$ for periods ranging from 1 sec to 5 sec
6. The site-specific procedures of Chapter 21 should be revised to require direct consideration of Site Class characteristics in determination of the spectral values, with the limitations noted in 3 above.
7. Where Site Class is not determined, as currently permitted by the Provisions, it shall be permitted to use the values defined for a "Default Site Class." The Default Site Class values shall be taken as having the maximum value (at each period) of the spectra for site class C, CD, and D, respectively.

The procedures described above can be applied using different sets of currently available GMPEs and related site terms for active tectonic regions in the western U.S. and in the central and eastern U.S. Currently available GMPEs in regions outside of the conterminous U.S. do not fully provide the capability to produce spectral values at all of the desired periods and site classes described above. In these and other locations, general rules will be produced for approximating spectral values at these periods and site classes. At the time of Project 17's conclusion, these rules were still under development.

6. Conclusions

The U.S. Geological Survey (USGS), under funding provided through the National Earthquake Hazards Reduction Program (NEHRP), develops national seismic design value maps for adoption in standards and building codes. The USGS develops these maps in a cooperative manner with the National Institute of Building Sciences' Building Seismic Safety Council's (BSSC) Provisions Update Committee (PUC). On a periodic basis, the PUC, acting under funding provided by the Federal Emergency Management Agency (FEMA), develops the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (*NEHRP Provisions*) for publication by FEMA as a resource document for standards and building codes and also as a standard for seismic design of new federally funded construction. The development of seismic design value maps is a significant part of this process.

FEMA, BSSC and USGS act cooperatively to develop the seismic maps. The current maps have undergone an evolutionary process over the past 30 years, with major innovations introduced approximately once every 10 years through efforts known as Project 97, Project 07, and Project 17. Project 17 is the latest such effort to formulate recommendations for the rules by which next-generation seismic design value maps will be developed for adoption in the *2020 NEHRP Provisions*, *ASCE 7-22* and the *2024 International Building Code*. This report summarizes the findings and recommendations by the Project 17 Committee and its five Working Groups, which include:

- Acceptable Risk: selecting an appropriate risk basis for the maps;
- Precision and Uncertainty: stabilizing the mapped values and associated design requirements over successive building code editions;
- Multi-Period Spectral Parameters: more properly representing site class effects, on soft sites where hazards are dominated by large magnitude earthquakes;
- Deterministic Maps: specifying the deterministic event on which seismic hazards are based at sites close to major active faults; and
- Seismic Design Category: minimizing the fluctuations that impact design requirements, specifically, with the objective of decoupling SDC from mapped ground motions.

The Project 17 effort concluded in September 2018 and the recommended resolutions documented in this report will be sent to PUC as *NEHRP Provisions* change proposals for review and ballot. Following the PUC ballot, the proposals will then be balloted again by BSSC Member Organizations (MOs), which represent the broader engineering and construction community. The final proposals that pass both the PUC and MO ballots and are approved by the BSSC Board of Direction will be included in *NEHRP Provisions*. Using the rules set by the PUC and included in the NEHRP Provisions, the USGS, with input from the earth science community and applying latest seismic hazard models, will develop the seismic design value maps, which will be presented for adoption in the *ASCE 7 Standard* and the building codes.

The detailed proposals balloted by the Project 17 Committee, including comments and resolutions, are provided in the Appendix; the final recommendations by the PUC on the related proposals will be published in future FEMA/BSSC reports once complete.

Appendix A. P17 Balloted Proposals

Proposals | Files | Admin

JiQiu Yuan

National Institute of Building Sciences

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P17 - Project 17 ballot on Multi-Period Spectra Proposals

Status

Finalized

Scope

Please vote yes or no to the following:

The Project 17 Committee approves forwarding proposals:

- CH11-MP P17-Ballot 01
- CH20-MP-P17-Ballot 01
- CH21 MP-P17 Ballot 01
- CH22 MP-P17 Ballot 01

to the Provisions Update Committee for finalization and approval, with the following instruction. The PUC is to be aware that to fully implement these proposals, the USGS requires approved Ground Motion Prediction Equations (GMPEs) that directly incorporate site class parameters and are able to generate spectral response parameters over the range of periods indicated. Presently, the U.S.G.S. has such GMPEs only for those portions of the Western U.S. where subduction hazards are negligible. U.S.G.S. anticipates that the NGA East and NGA Subduction GMPEs will be approved for use within a time frame that will permit them to fully implement these proposals across the continental United States. Even with approval of these GMPEs, the U.S.G.S. will require direction on how to generate the mapped spectral parameters in those locations where the available GMPEs are not directly applicable. Project 17 anticipates that PUC will charge an Issue Team with development of these rules prior to final approval of these proposals. The Issue Team will also need to develop commentary for these proposals.

Proponent

Charles Kircher

Voting Period

01-30-2018 3:00 PM – 02-12-2018 11:59 PM

Supporting Files

Ch11_-_MP_P17-Ballot_01.docx 2018-01-30 15:16:21
Ch20_MP_P17-Ballot_01.docx 2018-01-30 15:16:21
Ch21_MP_P17-Ballot_01.docx 2018-01-30 15:16:21
Ch22__MP_P17-Ballot01.docx 2018-01-30 15:16:21

Proposal Updates

Created: 01-30-2018 10:05 AM
Updated: 05-08-2018 1:49 PM

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Vote Summary for P17 - Project 17 ballot on Multi-Period Spectra Proposals

Results Report

Total Voting	Voted	50% Rule	Yes	Yes with Reservations	No	Not Voting	Online Ballot	PUC Meeting Final
15	13	Meet	12	1	0	0	Pass 'null' is not an object	

Vote and Comment Summary

Download Votes

Vote Key

Y Yes YR Yes with reservations
N No NV Not Voting

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	PUC Meeting
Bonneville	Y						
Crouse	Y						
Enfield	Y						
Furr	Y						
Hamburger	Y						
Harris	Y						
Heintz	Y						

Holmes	Y						
Hooper	Y						
Kircher	Y						
Pekelnicky	Y						
Siu	YR	General	General	My understanding is several details remain to be worked out at PUC. Conceptually, this is OK, but I have specific reservations about changing the thresholds for SDCs E and F by using SM1 instead of S1 in Section 11.6 without also changing the acceleration threshold. Doing so would have the effect of drawing more buildings into SDCs E and F that are currently designated as SDC D. We have not seen the justification for doing so.			
Stewart	Y						

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CHAPTER 11 SEISMIC DESIGN CRITERIA

11.1 GENERAL

11.1.1 Purpose.

Chapter 11 presents criteria for the design and construction of buildings and other structures subject to earthquake ground motions. The specified earthquake loads are based upon postelastic energy dissipation in the structure. Because of this fact, the requirements for design, detailing, and construction shall be satisfied, even for structures and members for which load combinations that do not include earthquake loads indicate larger demands than combinations that include earthquake loads.

11.1.2 Scope.

Every structure and portion thereof, including nonstructural components, shall be designed and constructed to resist the effects of earthquake motions as prescribed by the seismic requirements of this standard. Certain nonbuilding structures, as described in Chapter 15, are also within the scope and shall be designed and constructed in accordance with the requirements of Chapter 15. Requirements concerning alterations, additions, and change of use are set forth in Appendix 11B. Existing structures and alterations to existing structures need only comply with the seismic requirements of this standard where required by Appendix 11B. The following structures are exempt from the seismic requirements of this standard:

1. Detached one- and two-family dwellings that are located where the mapped, short period, spectral response acceleration parameter, S_S, is less than 0.4 or where the Seismic Design Category determined in accordance with Section 11.6 is A, B, or C.
2. Detached one- and two-family wood-frame dwellings not included in Exemption 1 with not more than two stories above grade plane, satisfying the limitations of and constructed in accordance with the IRC.
3. Agricultural storage structures that are intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed in Chapter 15 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances, and nuclear reactors.
5. Piers and wharves that are not accessible to the general public.

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:

a. Buildings: Chapter 12;
b. Nonbuilding Structures: Chapter 15;
c. Nonstructural Components: Chapter 13;

d. Seismically Isolated Structures: Chapter 17; and
e. Structures with Damping Systems: Chapter 18.

Buildings 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 classified as nonbuilding structures designed and detailed in accordance with Section 15.5 of this standard.

11.1.4 Alternate Materials and Methods of Construction.

Alternate materials and methods of construction to those prescribed in the seismic requirements of this standard shall not be used unless approved by the Authority Having Jurisdiction. Substantiating evidence shall be submitted demonstrating that the proposed alternate will be at least equal in strength, durability, and seismic resistance for the purpose intended.

11.1.5 Quality Assurance.

Quality assurance for seismic force-resisting systems and other designated seismic systems defined in Section 13.2.2 shall be provided in accordance with the requirements of the Authority Having Jurisdiction.

Where the Authority Having Jurisdiction has not adopted quality assurance requirements, or where the adopted requirements are not applicable to the seismic force-resisting system or designated seismic systems as described in Section 13.2.2, the registered design professional in responsible charge of designing the seismic force-resisting system or other designated seismic systems shall submit a quality assurance plan to the Authority Having Jurisdiction for approval. The quality assurance plan shall specify the quality assurance program elements to be implemented.

11.2 DEFINITIONS

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

ACTIVE FAULT: A fault determined to be active by the Authority Having Jurisdiction from properly substantiated data (e.g., most recent mapping of active faults by the U.S. Geological Survey).

ADDITION: An increase in building area, aggregate floor area, height, or number of stories of a structure.

ALTERATION: Any construction or renovation to an existing structure other than an addition.

APPENDAGE: An architectural component such as a canopy, marquee, ornamental balcony, or statuary.

APPROVAL: The written acceptance by the Authority Having Jurisdiction of documentation that establishes the qualification of a material, system, component, procedure, or person to fulfill the requirements of this standard for the intended use.

ATTACHMENTS: Means by which nonstructural components or supports of nonstructural components are secured or connected to the seismic force-resisting system of the structure. Such attachments include anchor bolts, welded connections, and mechanical fasteners.

BASE: The level at which the horizontal seismic ground motions are considered to be imparted to the structure.
BASE SHEAR: Total design lateral force or shear at the base.
BOUNDARY ELEMENTS: Portions along wall and diaphragm edges for transferring or resisting forces. Boundary elements include chords and collectors at diaphragm and shear wall perimeters, edges of openings, discontinuities, and reentrant corners.
BUILDING: Any structure whose intended use includes shelter of human occupants.
CANTILEVERED COLUMN SYSTEM: A seismic force-resisting system in which lateral forces are resisted entirely by columns acting as cantilevers from the base.
CHARACTERISTIC EARTHQUAKE: An earthquake assessed for an active fault having a magnitude equal to the best estimate of the maximum magnitude capable of occurring on the fault but not less than the largest magnitude that has occurred historically on the fault.
COLLECTOR (DRAG STRUT, TIE, DIAPHRAGM STRUT): A diaphragm or shear wall boundary element parallel to the applied load that collects and transfers diaphragm shear forces to the vertical elements of the seismic force-resisting system or distributes forces within the diaphragm or shear wall.
COMPONENT: A part of an architectural, electrical, or mechanical system.
Component, Flexible: Nonstructural component that has a fundamental period greater than 0.06 s.
Component, Nonstructural: A part of an architectural, mechanical, or electrical system within or without a building or nonbuilding structure.
Component, Rigid: Nonstructural component that has a fundamental period less than or equal to 0.06 s.
Component, Rugged: A nonstructural component that has been shown to consistently function after design earthquake level or greater seismic events based on past earthquake experience data or past seismic testing when adequately anchored or supported. The classification of a nonstructural component as rugged shall be based on a comparison of the specific component with components of similar strength and stiffness. Common examples of rugged components include AC motors, compressors, and base-mounted horizontal pumps.
CONCRETE:
Plain Concrete: Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in ACI 318 for reinforced concrete.
Reinforced Concrete: Concrete reinforced with no less reinforcement than the minimum amount required by ACI 318 prestressed or nonprestressed and designed on the assumption that the two materials act together in resisting forces.
CONSTRUCTION DOCUMENTS: The written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project required to verify compliance with this standard.
COUPLING BEAM: A beam that is used to connect adjacent concrete wall elements to make them act together as a unit to resist lateral loads.
DEFORMABILITY: The ratio of the ultimate deformation to the limit deformation.
High-Deformability Element: An element whose deformability is not less than 3.5 where subjected to four fully reversed cycles at the limit deformation.
Limited-Deformability Element: An element that is neither a low-deformability nor a high-deformability element.
Low-Deformability Element: An element whose deformability is 1.5 or less.

DEFORMATION:
Limit Deformation: Two times the initial deformation that occurs at a load equal to 40% of the maximum strength.
Ultimate Deformation: The deformation at which failure occurs and that shall be deemed to occur if the sustainable load reduces to 80% or less of the maximum strength.
DESIGN EARTHQUAKE: The earthquake effects that are two-thirds of the corresponding risk-targeted maximum considered earthquake (MCE_R) effects.
DESIGN EARTHQUAKE GROUND MOTION: The earthquake ground motions that are two-thirds of the corresponding MCE_R ground motions.
DESIGNATED SEISMIC SYSTEMS: Those nonstructural components that require design in accordance with Chapter 13 and for which the component Importance Factor, I_p, is greater than 1.0.
DIAPHRAGM: Roof, floor, or other membrane or bracing system acting to transfer the lateral forces to the vertical resisting elements.
Flexure-Controlled Diaphragm: Diaphragm with a flexural yielding mechanism, which limits the maximum forces that develop in the diaphragm, and having a design shear strength or factored nominal shear capacity greater than the shear corresponding to the nominal flexural strength.
Shear-Controlled Diaphragm: Diaphragm that does not meet the requirements of a flexure-controlled diaphragm.
Transfer Forces, Diaphragm: Forces that occur in a diaphragm caused by transfer of seismic forces from the vertical seismic force-resisting elements above the diaphragm to other vertical seismic force-resisting elements below the diaphragm because of offsets in the placement of the vertical elements or changes in relative lateral stiffnesses of the vertical elements.
Vertical Diaphragm: See WALL, Shear Wall.
DIAPHRAGM BOUNDARY: A location where shear is transferred into or out of the diaphragm element. Transfer is either to a boundary element or to another force-resisting element.
DIAPHRAGM CHORD: A diaphragm boundary element perpendicular to the applied load that is assumed to take axial stresses caused by the diaphragm moment.
DISTRIBUTION SYSTEM: An interconnected system of piping, tubing, conduit, raceway, or duct. Distribution systems include in-line components such as valves, in-line suspended pumps, and mixing boxes.
ELEMENT ACTION: Element axial, shear, or flexural behavior.
Critical Action: An action, failure of which would result in the collapse of multiple bays or multiple stories of the building or would result in a significant reduction in the structure's seismic resistance.
Deformation-Controlled Action: Element actions for which reliable inelastic deformation capacity is achievable without critical strength decay.
Force-Controlled Action: Any element actions modeled with linear properties and element actions not classified as deformation-controlled.
Noncritical Actions: An action, failure of which would not result in either collapse or significant loss of the structure's seismic resistance.

Ordinary Action: An action, failure of which would result in only local collapse, comprising not more than one bay in a single story, and would not result in a significant reduction of the structure's seismic resistance.
ENCLOSURE: An interior space surrounded by walls.
EQUIPMENT SUPPORT: Those structural members or assemblies of members or manufactured elements, including braces, frames, legs, lugs, snuggers, hangers, or saddles, that transmit gravity loads and operating loads between the equipment and the structure.
FLEXIBLE CONNECTIONS: Those connections between equipment components that permit rotational and/or translational movement without degradation of performance. Examples include universal joints, bellows expansion joints, and flexible metal hose.
FOUNDATION GEOTECHNICAL CAPACITY: The maximum pressure or strength design capacity of a foundation based upon the supporting soil, rock, or controlled low-strength material.
FOUNDATION STRUCTURAL CAPACITY: The design strength of foundations or foundation components as provided by adopted material standards and as altered by the requirements of this standard.
FRAME:
Braced Frame: An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a building frame system or dual system to resist seismic forces.
Concentrically Braced Frame (CBF): A braced frame in which the members are subjected primarily to axial forces. CBFs are categorized as ordinary concentrically braced frames (OCBFs) or special concentrically braced frames (SCBFs).
Eccentrically Braced Frame (EBF): A diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column or from another diagonal brace.
Moment Frame: A frame in which members and joints resist lateral forces by flexure and along the axis of the members. Moment frames are categorized as intermediate moment frames (IMFs), ordinary moment frames (OMFs), and special moment frames (SMFs).
Structural System:
Building Frame System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.
Dual System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment-resisting frames and shear walls or braced frames as prescribed in Section 12.2.5.1.
Shear Wall–Frame Interactive System: A structural system that uses combinations of ordinary reinforced concrete shear walls and ordinary reinforced concrete moment frames designed to resist lateral forces in proportion to their rigidities considering interaction between shear walls and frames on all levels.
Space Frame System: A 3-D structural system composed of interconnected members, other than bearing walls, that is capable of supporting vertical loads and, where designed for such an application, is capable of providing resistance to seismic forces.
FRICTION CLIP: A device that relies on friction to resist applied loads in one or more directions to anchor a nonstructural component. Friction is provided mechanically and is not due to gravity loads.

GLAZED CURTAIN WALL: A nonbearing wall that extends beyond the edges of building floor slabs and includes a glazing material installed in the curtain wall framing.
GLAZED STOREFRONT: A nonbearing wall that is installed between floor slabs, typically including entrances, and includes a glazing material installed in the storefront framing.
GRADE PLANE: A horizontal reference plane representing the average of finished ground level adjoining the structure at all exterior walls. Where the finished ground level slopes away from the exterior walls, the grade plane is established by the lowest points within the area between the structure and the property line or, where the property line is more than 6 ft (1,829 mm) from the structure, between the structure and points 6 ft (1,829 mm) from the structure.
HEATING, VENTILATING, AIR-CONDITIONING, AND REFRIGERATION (HVACR): The equipment, distribution systems, and terminals, excluding interconnecting piping and ductwork that provide, either collectively or individually, the processes of heating, ventilating, air-conditioning, or refrigeration to a building or portion of a building.
INSPECTION, SPECIAL: The observation of the work by a special inspector to determine compliance with the approved construction documents and these standards in accordance with the quality assurance plan.
Continuous Special Inspection: The full-time observation of the work by a special inspector who is present in the area where work is being performed.
Periodic Special Inspection: The part-time or intermittent observation of the work by a special inspector who is present in the area where work has been or is being performed.
INSPECTOR, SPECIAL: A person approved by the Authority Having Jurisdiction to perform special inspection, and who shall be identified as the owner's inspector.
INVERTED PENDULUM-TYPE STRUCTURES: Structures in which more than 50% of the structure's mass is concentrated at the top of a slender, cantilevered structure and in which stability of the mass at the top of the structure relies on rotational restraint to the top of the cantilevered element.
JOINT: The geometric volume common to intersecting members.
LIGHT-FRAME CONSTRUCTION: A method of construction where the structural assemblies (e.g., walls, floors, ceilings, and roofs) are primarily formed by a system of repetitive wood or cold-formed steel framing members or subassemblies of these members (e.g., trusses).
LONGITUDINAL REINFORCEMENT RATIO: Area of longitudinal reinforcement divided by the cross-sectional area of the concrete.
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION: The most severe earthquake effects considered by this standard, more specifically defined in the following two terms:
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 liquefaction, lateral spreading, seismic settlements, and other soil-related issues. 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 11.4.34; site-specific procedures are provided in Sections 21.1 and 21.2.

MECHANICALLY ANCHORED TANKS OR VESSELS: Tanks or vessels provided with mechanical anchors to resist overturning moments.

NONBUILDING STRUCTURE: A structure, other than a building, constructed of a type included in Chapter 15 and within the limits of Section 15.1.1.

NONBUILDING STRUCTURE SIMILAR TO A BUILDING: A nonbuilding structure that is designed and constructed in a manner similar to buildings, responds to strong ground motion in a fashion similar to buildings, and has a basic lateral and vertical seismic force-resisting system conforming to one of the types indicated in Tables 12.2-1 or 15.4-1.

OPEN-TOP TANK: A tank without a fixed roof or cover, floating cover, gas holder cover, or dome.

ORTHOGONAL: In two horizontal directions, at 90° to each other.

OWNER: Any person, agent, firm, or corporation that has a legal or equitable interest in a property.

P-DELTA EFFECT: The secondary effect on shears and moments of structural members caused by the action of the vertical loads induced by horizontal displacement of the structure resulting from various loading conditions.

PARTITION: A nonstructural interior wall that spans horizontally or vertically from support to support. The supports may be the basic building frame, subsidiary structural members, or other portions of the partition system.

PILE: Deep foundation element, which includes piers, caissons, and piles.

PILE CAP: Foundation elements to which piles are connected, including grade beams and mats.

PREMANUFACTURED MODULAR MECHANICAL AND ELECTRICAL SYSTEM: A prebuilt, fully or partially enclosed assembly of mechanical and electrical components.

REGISTERED DESIGN PROFESSIONAL: An architect or engineer registered or licensed to practice professional architecture or engineering, as defined by the statutory requirements of the professional registration laws of the state in which the project is to be constructed.

SEISMIC DESIGN CATEGORY: A classification assigned to a structure based on its Risk Category and the severity of the design earthquake ground motion at the site, as defined in Section 11.4.

SEISMIC FORCE-RESISTING SYSTEM: That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed herein.

SEISMIC FORCES: The assumed forces prescribed herein, related to the response of the structure to earthquake motions, to be used in the design of the structure and its components.

SELF-ANCHORED TANKS OR VESSELS: Tanks or vessels that are stable under design overturning moment without the need for mechanical anchors to resist uplift.

SHEAR PANEL: A floor, roof, or wall element sheathed to act as a shear wall or diaphragm.

SITE CLASS: A classification assigned to a site based on the types of soils present and their engineering properties, as defined in Chapter 20.

STORAGE RACKS, STEEL: A framework or assemblage, comprised of cold-formed or hot-rolled steel structural members, intended for storage of materials, including, but not limited to, pallet storage racks, selective racks, movable-shelf racks, rack-supported systems, automated

storage and retrieval systems (stacker racks), push-back racks, pallet-flow racks, case-flow racks, pick modules, and rack-supported platforms. Other types of racks, such as drive-in or drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel, are not considered steel storage racks for the purpose of this standard.

STORAGE RACKS, STEEL CANTILEVERED: A framework or assemblage comprised of cold-formed or hot-rolled steel structural members, primarily in the form of vertical columns, extended bases, horizontal arms projecting from the faces of the columns, and longitudinal (down-aisle) bracing between columns. There may be shelf beams between the arms, depending on the products being stored; this definition does not include other types of racks such as pallet storage racks, drive-in racks, drive-through racks, or racks made of materials other than steel.

STORY: The portion of a structure between the tops of two successive floor surfaces and, for the topmost story, from the top of the floor surface to the top of the roof surface.

STORY ABOVE GRADE PLANE: A story in which the floor or roof surface at the top of the story is more than 6 ft (1,828 mm) above grade plane or is more than 12 ft (3,658 mm) above the finished ground level at any point on the perimeter of the structure.

STORY DRIFT: The horizontal deflection at the top of the story relative to the bottom of the story as determined in Section 12.8.6.

STORY DRIFT RATIO: The story drift, as determined in Section 12.8.6, divided by the story height, h_{sx}.

STORY SHEAR: The summation of design lateral seismic forces at levels above the story under consideration.

STRENGTH:

Design Strength: Nominal strength multiplied by a strength reduction factor, ϕ.

Nominal Strength: Strength of a member or cross section calculated in accordance with the requirements and assumptions of the strength design methods of this standard (or the reference documents) before application of any strength-reduction factors.

Required Strength: Strength of a member, cross section, or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by this standard.

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.

STRUCTURAL OBSERVATIONS: The visual observations to determine that the seismic force-resisting system is constructed in general conformance with the construction documents.

STRUCTURE: That which is built or constructed and limited to buildings and nonbuilding structures as defined herein.

SUBDIAPHRAGM: A portion of a diaphragm used to transfer wall anchorage forces to diaphragm crossties.

SUPPORTS: Those members, assemblies of members, or manufactured elements, including braces, frames, legs, lugs, snubbers, hangers, saddles, or struts, and associated fasteners that transmit loads between nonstructural components and their attachments to the structure.

TESTING AGENCY: A company or corporation that provides testing and/or inspection services.

VENEERS: Facings or ornamentation of brick, concrete, stone, tile, or similar materials attached to a backing.

WALL: A component that has a slope of 60 deg or greater with the horizontal plane used to enclose or divide space.
Bearing Wall: Any wall meeting either of the following classifications:

1. Any metal or wood stud wall that supports more than 100 lb/linear ft (1,459 N / m) of vertical load in addition to its own weight.
2. Any concrete or masonry wall that supports more than 200 lb/linear ft (2,919 N / m) of vertical load in addition to its own weight.

Light Frame Wall: A wall with wood or steel studs.
Light Frame Wood Shear Wall: A wall constructed with wood studs and sheathed with material rated for shear resistance.
Nonbearing Wall: Any wall that is not a bearing wall.
Nonstructural Wall: A wall other than a bearing wall or shear wall.
Shear Wall (Vertical Diaphragm): A wall, bearing or nonbearing, designed to resist lateral forces acting in the plane of the wall (sometimes referred to as a "vertical diaphragm").
Structural Wall: A wall that meets the definition for bearing wall or shear wall.
WALL SYSTEM, BEARING: A structural system with bearing walls providing support for all or major portions of the vertical loads. Shear walls or braced frames provide seismic force resistance.
WOOD STRUCTURAL PANEL: A wood-based panel product that meets the requirements of DOC PS1 or DOC PS2 and is bonded with a waterproof adhesive. Included under this designation are plywood, oriented strand board, and composite panels.

11.3 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 seismic provisions of Chapters 11 through 22 in this standard.

A_0 = area of the load-carrying foundation [ft^2 (m^2)]

A_{ch} = cross-sectional area [$in.^2$ (mm^2)] of a structural member measured out-to-out of transverse reinforcement

A_{sh} = total cross-sectional area of hoop reinforcement [$in.^2$ (mm^2)], including supplementary crossties, having a spacing of s_h and crossing a section with a core dimension of h_c

A_{vd} = required area of leg [$in.^2$ (mm^2)] of diagonal reinforcement

A_x = torsional amplification factor (Section 12.8.4.3)

a_i = the acceleration at level i obtained from a modal analysis (Section 13.3.1)

a_p = the amplification factor related to the response of a system or component as affected by the type of seismic attachment, determined in Section 13.3.1

b_p = the width of the rectangular glass panel

C_d = deflection amplification factor as given in Tables 12.2-1, 15.4-1, or 15.4-2

C_{dX} = deflection amplification factor in the X direction (Section 12.9.2.5)

C_{dY} = deflection amplification factor in the Y direction (Section 12.9.2.5)

C_{p0} = diaphragm design acceleration coefficient at the structure base (Section 12.10.3.2.1)

C_{pi} = diaphragm design acceleration coefficient at 80% of the structural height above the base, h_n (Section 12.10.3.2.1)

C_{pn} = diaphragm design acceleration coefficient at the structural height, h_n (Section 12.10.3.2.1)

C_{px} = diaphragm design acceleration coefficient at level x (Section 12.10.3.2.1)

C_R = site-specific risk coefficient at any period (Section 21.2.1.1)

C_{R1} = mapped value of the risk coefficient at a period of 1 s as given by Fig. 22-19

C_{RS} = mapped value of the risk coefficient at short periods as given by Fig. 22-18

C_s = seismic response coefficient determined in Section 12.8.1.1 or 19.3.1 (dimensionless)

C_{s2} = higher mode seismic response coefficient (Section 12.10.3.2.1)

C_t = building period coefficient (Section 12.8.2.1)

C_{vx} = vertical distribution factor as determined (Section 12.8.3)

c = distance from the neutral axis of a flexural member to the fiber of maximum compressive strain [in. (mm)]

D = the effect of dead load

D_{clear} = relative horizontal (drift) displacement, measured over the height of the glass panel under consideration, which causes initial glass-to-frame contact. For rectangular glass panels within a rectangular wall frame, D_{clear} is set forth in Section 13.5.9.1

D_{pI} = seismic relative displacement; see Section 13.3.2

D_s = the total depth of stratum in Eq. (19.3-4) [ft (m)]

d_c = the total thickness of cohesive soil layers in the top 100 ft (30 m); see Section 20.4.3 [ft (m)]

d_i = the thickness of any soil or rock layer i [between 0 and 100 ft (between 0 and 30 m)]; see Section 20.4.1 [ft (m)]

d_S = the total thickness of cohesionless soil layers in the top 100 ft (30 m); see Section 20.4.2 [ft (m)]

E = effect of horizontal and vertical earthquake-induced forces (Section 12.4)

E_{cl} = The capacity-limited horizontal seismic load effect, equal to the maximum force that can develop in the element as determined by a rational, plastic mechanism analysis

~~F_a = short-period site coefficient (at 0.2-s period); see Section 11.4.4~~

F_i, F_n, F_x = portion of the seismic base shear, V, induced at level i, n, or x, respectively, as determined in Section 12.8.3

F_p = the seismic force acting on a component of a structure as determined in Sections 12.11.1 and 13.3.1

F_{px} = diaphragm seismic design force at Level x

~~F_{PGA} = site coefficient for peak ground acceleration (PGA); see Section 11.8.3~~

~~F_v = long-period site coefficient (at 1.0-s period); see Section 11.4.4~~

f_c' = specified compressive strength of concrete used in design

f_s' = ultimate tensile strength [psi (MPa)] of the bolt, stud, or insert leg wires. For ASTM A307 bolts or ASTM A108 studs, it is permitted to be assumed to be 60,000 psi (415 MPa)

f_y = specified yield strength of reinforcement [psi (MPa)]

f_{yh} = specified yield strength of the special lateral reinforcement [psi (kPa)]

$G = \gamma \upsilon_s^2 / g$ = the average shear modulus for the soils beneath the foundation at large strain levels [psf (Pa)]

$G_0 = \gamma \upsilon_{s0}^2 / g$ = the average shear modulus for the soils beneath the foundation at small strain levels [psf (Pa)]

g = acceleration due to gravity

H = thickness of soil

h = height of a shear wall measured as the maximum clear height from top of foundation to bottom of diaphragm framing above, or the maximum clear height from top of diaphragm to bottom of diaphragm framing above

h = average roof height of structure with respect to the base; see Chapter 13

h^* = effective height of the building as determined in Chapter 19 [ft (m)]

h_c = core dimension of a component measured to the outside of the special lateral reinforcement [in. (mm)]

h_i, h_x = the height above the base to level i or x, respectively

h_n = structural height as defined in Section 11.2

h_p = the height of the rectangular glass panel

h_{sx} = the story height below level $x = (h_x - h_{x-1})$

I_e = the Importance Factor as prescribed in Section 11.5.1

I_p = the component importance factor as prescribed in Section 13.3.1

i = the building level referred to by the subscript i; $i = 1$ designates the first level above the base

K_p = the stiffness of the component or attachment (Section 13.3.3)

K_{xx}, K_{rr} = rotational foundation stiffness [Eqs. (19.3-9) and (19.3-19) [ft-lb / degree (N-m / rad)]

K_y, K_r = translational foundational stiffness [Eqs. (19.3-8) and (19.3-18)] [lb / in. (N / m)]

KL / r = the lateral slenderness ratio of a compression member measured in terms of its effective length, KL, and the least radius of gyration of the member cross section, r

k = distribution exponent given in Section 12.8.3

k_a = coefficient defined in Sections 12.11.2.1 and 12.14.7.5

L = overall length of the building (ft or m) at the base in the direction being analyzed

M_t = torsional moment resulting from eccentricity between the locations of center of mass and the center of rigidity (Section 12.8.4.1)

M_{ta} = accidental torsional moment as determined in Section 12.8.4.2

m = a subscript denoting the mode of vibration under consideration; that is, $m = 1$ for the fundamental mode

N = standard penetration resistance, ASTM D1586

N = number of stories above the base (Section 12.8.2.1)

$\overline{N}$ = average field standard penetration resistance for the top 100 ft (30 m); see Sections 20.3.3 and 20.4.2

$\overline{N}_{ch}$ = average standard penetration resistance for cohesionless soil layers for the top 100 ft (30 m); see Sections 20.3.3 and 20.4.2

N_i = standard penetration resistance of any soil or rock layer i [between 0 and 100 ft (between 0 and 30 m)]; see Section 20.4.2

n = designation for the level that is uppermost in the main portion of the building

~~PGA = mapped MCE_G peak ground acceleration shown in Figs. 22-9 through 22-13~~

PGA_M = mapped MCE_G peak ground acceleration adjusted for site class effects; see Section 11.8.3

PI = plasticity index, ASTM D4318

P_x = total unfactored vertical design load at and above level x, for use in Section 12.8.7

Q_E = effect of horizontal seismic (earthquake-induced) forces

R = response modification coefficient as given in Tables 12.2-1, 12.14-1, 15.4-1, and 15.4-2

R_p = component response modification factor as defined in Section 13.3.1

R_s = diaphragm design force reduction factor (Section 12.10.3.5)

R_X = response modification coefficient in the X direction (Section 12.9.2.5)

R_Y = response modification coefficient in the Y direction (Section 12.9.2.5)

~~S_1 = mapped MCE_R, 5% damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.2~~

S_{aM} = the site-specific MCE_R spectral response acceleration parameter at any period

S_{D1} = design, 5% damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.~~5~~

S_{DS} = design, 5% damped, spectral response acceleration parameter at short periods as defined in Section 11.4.~~5~~

S_{M1} = mapped~~the~~ MCE_R, 5% damped, spectral response acceleration parameter at a period of 1 s adjusted for site class effects as defined in Section 11.4.34

S_{MS} = mapped~~the~~ MCE_R, 5% damped, spectral response acceleration parameter at short periods adjusted for site class effects as defined in Section 11.4.34.

~~S_S = mapped MCE_R, 5% damped, spectral response acceleration parameter at short periods as defined in Sections 11.4.2, 11.4.4~~

s_h = spacing of special lateral reinforcement [in. (mm)]

s_u = undrained shear strength; see Section 20.4.3

$\overline{s}_u$ = average undrained shear strength in top 100 ft (30 m); see Sections 20.3.3 and 20.4.3, ASTM D2166, or ASTM D2850

s_{ui} = undrained shear strength of any cohesive soil layer i [between 0 and 100 ft (0 and 30 m)]; see Section 20.4.3

T = the fundamental period of the building

$T_0 = 0.2S_{D1} / S_{DS}$

$\tilde{T}$ = the fundamental period as determined in Chapter 19

T_a = approximate fundamental period of the building as determined in Section 12.8.2

T_L = long-period transition period as defined in Section 11.4.5~~6~~

T_{lower} = period of vibration at which 90% of the actual mass has been recovered in each of the two orthogonal directions of response (Section 12.9.2). The mathematical model used to compute T_{lower} shall not include accidental torsion and shall include P-delta effects.

T_p = fundamental period of the component and its attachment (Section 13.3.3)

$T_S = S_{D1} / S_{DS}$

T_{upper} = the larger of the two orthogonal fundamental periods of vibration (Section 12.9.2). The mathematical model used to compute T_{upper} shall not include accidental torsion and shall include P-delta effects

V = total design lateral force or shear at the base

V_{EX} = maximum absolute value of elastic base shear computed in the X direction among all three analyses performed in that direction (Section 12.9.2.5)

V_{EY} = maximum absolute value of elastic base shear computed in the Y direction among all three analyses performed in that direction (Section 12.9.2.5)

V_{IX} = inelastic base shear in the X direction (Section 12.9.2.5)

V_{IY} = inelastic base shear in the Y direction (Section 12.9.2.5)

V_t = design value of the seismic base shear as determined in Section 12.9.1.4.1

V_X = ELF base shear in the X direction (Section 12.9.2.5)

V_x = seismic design shear in story x as determined in Section 12.8.4

V_Y = ELF base shear in the Y direction (Section 12.9.2.5)

$\tilde{V}$ = reduced base shear accounting for the effects of soil structure interaction as determined in Section 19.3.1

$\tilde{V}_1$ = portion of the reduced base shear, $\tilde{V}_1$ contributed by the fundamental mode, Section 19.3, in kip (kN)

ΔV = reduction in V as determined in Section 19.3.1, in kip (kN)

ΔV_1 = reduction in V_1 as determined in Section 19.3.1, in kip (kN)

v_s = shear wave velocity at small shear strains (greater than 10^{-3}% strain); see Section 19.2.1, in ft/s (m/s)

$\overline{v}_s$ = average shear wave velocity at small shear strains in top 100 ft (30 m); see Sections 20.3.3 and 20.4.1

v_{si} = the shear wave velocity of any soil or rock layer i (between 0 and 100 ft (between 0 and 30 m)); see Section 20.4.1

v_{so} = average shear wave velocity for the soils beneath the foundation at small strain levels, Section 19.2.1.1 in ft/s (m/s)

W = effective seismic weight of the building as defined in Section 12.7.2. For calculation of seismic-isolated building period, W is the total effective seismic weight of the building as defined in Sections 19.2 and 19.3, in kip (kN)

W = effective seismic weight of the building as defined in Sections 19.2 and 19.3, in kip (kN)

W_c = gravity load of a component of the building

W_P = component operating weight, in lb (N)

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

w = moisture content (in percent), ASTM D2216

w_i, w_n, w_x = portion of W that is located at or assigned to level i, n, or x, respectively

x = level under consideration, 1 designates the first level above the base

z = height in structure of point of attachment of component with respect to the base; see Section 13.3.1

z_s = mode shape factor, Section 12.10.3.2.1

β = ratio of shear demand to shear capacity for the story between levels x and $x-1$

$\overline{\beta}$ = fraction of critical damping for the coupled structure–foundation system, determined in Section 19.2.1

β_0 = foundation damping factor as specified in Section 19.2.1.2

Γ_{m1}, Γ_{m2} = first and higher modal contribution factors, respectively, Section 12.10.3.2.1

γ = average unit weight of soil, in lb/ft^3 (N/m^3)

Δ = design story drift as determined in Section 12.8.6

Δ_{fallout} = the relative seismic displacement (drift) at which glass fallout from the curtain wall, storefront, or partition occurs

Δ_a = allowable story drift as specified in Section 12.12.1

Δ_{ADVE} = average drift of adjoining vertical elements of the seismic force-resisting system over the story below the diaphragm under consideration, under tributary lateral load equivalent to that used in the computation of δ_{MDD} Fig. 12.3-1, in in. (mm)

δ_{MDD} = computed maximum in-plane deflection of the diaphragm under lateral load, Fig. 12.3-1, in in. (mm)

δ_{max} = maximum displacement at level x, considering torsion, Section 12.8.4.3

δ_M = maximum inelastic response displacement, considering torsion, Section 12.12.3

δ_{MT} = total separation distance between adjacent structures on the same property, Section 12.12.3

δ_{avg} = the average of the displacements at the extreme points of the structure at level x, Section 12.8.4.3

δ_x = deflection of level x at the center of the mass at and above level x, Eq. (12.8-15)

δ_{xc} = deflection of level x at the center of the mass at and above level x determined by an elastic analysis, Section 12.8.6

δ_{xm} = modal deflection of level x at the center of the mass at and above level x as determined by Section 19.3.2

$\overline{\delta}_x, \overline{\delta}_{x1}$ = deflection of level x at the center of the mass at and above level x, Eqs. (19.2-13) and (19.3-3), in in. (mm)

θ = stability coefficient for P-delta effects as determined in Section 12.8.7

ηx = Force scale factor in the *X* direction (12.9.2.5)

ηy = Force scale factor in the *Y* direction (12.9.2.5)

ρ = a redundancy factor based on the extent of structural redundancy present in a building as defined in Section 12.3.4

ρ_s = spiral reinforcement ratio for precast, prestressed piles in Section 14.2.3.2.6

λ = time effect factor

Ω_0 = overstrength factor as defined in Tables 12.2-1, 15.4.-1, and 15.4-2

Ω_v = Diaphragm shear overstrength factor (Section 14.2.4.1.3)

11.4 SEISMIC GROUND MOTION VALUES

11.4.1 Near-Fault Sites.

Sites satisfying either of the following conditions shall be classified as near fault:

1. 9.5 miles (15 km) of the surface projection of a known active fault capable of producing $M_w 7$ or larger events, or
2. 6.25 miles (10 km) of the surface projection of a known active fault capable of producing $M_w 6$ or larger events.

EXCEPTIONS:

1. Faults with estimated slip rate along the fault less than 0.04 in. (1 mm) per year shall not be considered.
2. The surface projection shall not include portions of the fault at depths of 6.25 mi (10 km) or greater.

~~11.4.2 Mapped Acceleration Parameters.~~

~~The parameters S_S and S_1 shall be determined from the 0.2- and 1-s spectral response accelerations shown in Figs. 22-1, 22-3, 22-5, 22-6, 22-7, and 22-8 for S_S and Figs. 22-2, 22-4, 22-5, 22-6, 22-7, and 22-8 for S_1. Where S_1 is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 11.7.~~

User Note: Electronic values of mapped acceleration parameters and other seismic design parameters are provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76.

11.4.3 Site Class.

Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with Chapter 20. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D, subject to the requirements of Section 11.4.4, shall be used unless the authority having jurisdiction or geotechnical data determine that Site Class E or F soils are present at the site.

For situations in which site investigations, performed in accordance with Chapter 20, reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, the site coefficients F_a, F_v and F_{PGA} shall be taken as unity (1.0).

11.4.4 Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters.

The MCE_R spectral response acceleration parameters for short periods (S_{MS}) and at 1 s (S_{M1}), adjusted for site class effects, shall be determined by Eqs. (11.4-1) and (11.4-2), respectively.

$$S_{MS} = F_a S_S \quad (11.4\text{-}1)$$

$$S_{M1} = F_v S_1 \quad (11.4\text{-}2)$$

where

S_S = the mapped MCE_R spectral response acceleration parameter at short periods as determined in accordance with Section 11.4.2, and

S_1 = the mapped MCE_R spectral response acceleration parameter at a period of 1 s as determined in accordance with Section 11.4.2

where site coefficients F_a and F_v are defined in Tables 11.4-1 and 11.4-2, respectively. Where Site Class D is selected as the default site class per Section 11.4.3, the value of F_a shall not be less than 1.2. Where the simplified design procedure of Section 12.14 is used, the value of F_a shall be determined in accordance with Section 12.14.8.1, and the values for F_v, S_{MS}, and S_{M1} need not be determined.

Table 11.4-1 Short-Period Site Coefficient, F_a

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
Site Class	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0

E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_s.

Table 11.4-2 Long-Period Site Coefficient, F_v

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
Site Class	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.

[a]Also, see requirements for site-specific ground motions in Section 11.4.8.

11.4.2 Site Class.

Based on the site soil properties, the site shall be classified as Site Class A, B, BC, C, CD, D, DE, E, or F in accordance with Chapter 20. For situations in which site investigation, performed in accordance with Chapter 20, reveal rock conditions consistent with Site Class B site conditions, but site-specific velocity measurements are not made, risk-targeted maximum considered earthquake (MCE_R) spectral response accelerations shall be based on the Site Class BC site conditions.

11.4.2.1 Default Site Class.

Where the soil properties are not known in sufficient detail to determine the site class, risk-targeted maximum considered earthquake (MCE_R) spectral response accelerations shall be based on the more critical spectral response acceleration of Site Class C, Site Class CD, Site Class D and Site Class DE site conditions, unless the authority having jurisdiction or geotechnical data determine that Site Class E or F soils are present at the site.

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_{MS} and S_{M1} shall be determined from the mapped values of these parameters

provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76 for the site class determined in accordance with the site class requirements of Section 11.4.2.

Where the soil properties are not known in sufficient detail to determine the site class and the default site class requirements of Section 11.4.2.1 apply, risk-targeted maximum considered earthquake (MCE_R) spectral response acceleration parameters S_{MS} and S_{M1} shall be determined from the mapped values of 0.2- and 1-s spectral response accelerations shown in Figs. 22-1, 22-3, 22-5, 22-6, 22-7, and 22-8 for S_{MS} and Figs. 22-2, 22-4, 22-5, 22-6, 22-7, and 22-8 for S_{M1}.

11.4.~~4~~5 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 Eqs. (11.4-1~~3~~) and (11.4-24), respectively. Where the alternate simplified 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.

$$S_{DS} = \frac{2}{3} S_{MS} \tag{11.4-1~~3~~}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{11.4-24}$$

where

S_{MS} = the mapped MCE_R spectral response acceleration parameter at short periods as determined in accordance with Section 11.4.3, and

S_{M1} = the mapped MCE_R spectral response acceleration parameter at a period of 1 s as determined in accordance with Section 11.4.3.

11.4.~~5~~6 Design Response Spectrum.

Where a design response spectrum is required by this standard and site-specific ground motion procedures are not used, the design response spectrum curve shall be developed as indicated in Fig. 11.4-1 and as follows:

1. For periods less than T_0, the design spectral response acceleration, S_a, shall be taken as given in Eq. (11.4-3~~5~~):

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \tag{11.4-3~~5~~}$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken as equal to S_{DS}.

3. For periods greater than T_S and less than or equal to T_L, the design spectral response acceleration, S_a, shall be taken as given in Eq. (11.4-~~46~~):

$$S_a = \frac{S_{D1}}{T} \qquad (11.4\text{-}\sout{46})$$

4. For periods greater than T_L, S_a shall be taken as given in Eq. (11.4-~~57~~):

$$S_a = \frac{S_{D1} T_L}{T^2} \qquad (11.4\text{-}\sout{57})$$

Where
S_{DS} = the design spectral response acceleration parameter at short periods
S_{D1} = the design spectral response acceleration parameter at a 1-s period
T = the fundamental period of the structure, s
$T_0 = 0.2(S_{D1} / S_{DS})$
$T_S = S_{D1} / S_{DS}$, and
T_L = long-period transition period(s) shown in Figs. 22-14 through 22-17.

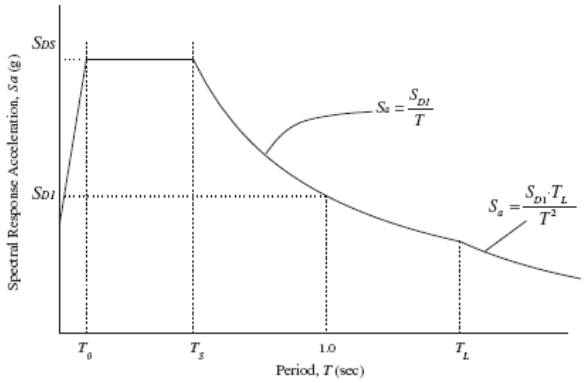


FIGURE 11.4-1 Design Response Spectrum

11.4.~~6~~7 Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum.

Where an MCE_R response spectrum is required, it shall be determined by multiplying the design response spectrum by 1.5.

11.4.~~7~~8 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. A ground motion hazard analysis shall be performed in accordance with Section 21.2 for ~~the following:~~ seismically

isolated structures and structures with damping systems on sites with S_{M1}~~S_1~~ greater than or equal to 0.6,

1. ~~structures on Site Class E sites with S_S greater than or equal to 1.0, and.~~
2. ~~structures on Site Class D and E sites with S_1 greater than or equal to 0.2.~~

~~**EXCEPTION:** A ground motion hazard analysis is not required for structures other than seismically isolated structures and structures with damping systems where:~~

1. ~~Structures on Site Class E sites with S_S greater than or equal to 1.0, provided the site coefficient F_a is taken as equal to that of Site Class C.~~
2. ~~Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T <= 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L >= T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$.~~
3. ~~Structures on Site Class E sites with S_1 greater than or equal to 0.2, provided that T is less than or equal to T_s and the equivalent static force procedure is used for design.~~

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_G peak ground acceleration parameter shall be determined in accordance with Section 21.5.

11.5 IMPORTANCE FACTOR AND RISK CATEGORY

11.5.1 Importance Factor.

An Importance Factor, I_e, shall be assigned to each structure in accordance with Table 1.5-2.

11.5.2 Protected Access for Risk Category IV.

Where operational access to a Risk Category IV structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for Risk Category IV structures. Where operational access is less than 10 ft (3.048 m) from an interior lot line or another structure on the same lot, protection from potential falling debris from adjacent structures shall be provided by the owner of the Risk Category IV structure.

11.6 SEISMIC DESIGN CATEGORY

Structures shall be assigned a Seismic Design Category in accordance with this section.

Risk Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_{M1}~~S_1~~, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Risk Category IV structures located where the mapped spectral response acceleration parameter at 1-s period, S_{M1}~~S_1~~, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a Seismic Design Category

based on their Risk Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in accordance with Section 11.4.~~4~~5. Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 11.6-1 or 11.6-2, irrespective of the fundamental period of vibration of the structure, T. The provisions in Chapter 19 shall not be used to modify the spectral response acceleration parameters for determining Seismic Design Category.

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
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

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
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$.	D	

Where $\underline{S_{M1}}$~~S_1~~ is less than 0.75, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone where all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, determined in accordance with Section 12.8.2.1 is less than $0.8T_s$, where T_s is determined in accordance with Section 11.4.~~5~~6.
2. In each of two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Eq. (12.8-2) is used to determine the seismic response coefficient C_s.
4. The diaphragms are rigid in accordance with Section 12.3; or, for diaphragms that are not rigid, the horizontal distance between vertical elements of the seismic force-resisting system does not exceed 40 ft (12.192 m).

Where the alternate simplified design procedure of Section 12.14 is used, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone, using the value of S_{DS} determined in Section 12.14.8.1, except that where $\underline{S_{M1}}$~~S_1~~ is greater than or equal to 0.75, the Seismic Design Category shall be E.

11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

Buildings and other structures assigned to Seismic Design Category A need only comply with the requirements of Section 1.4. Nonstructural components in SDC A are exempt from seismic design requirements. In addition, tanks assigned to Risk Category IV shall satisfy the freeboard requirement in Section 15.6.5.1.

11.8 Geologic Hazards and Geotechnical Investigation

11.8.1 Site Limitation for Seismic Design Categories E and F.

A structure assigned to Seismic Design Category E or F shall not be located where a known potential exists for an active fault to cause rupture of the ground surface at the structure.

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 faulting or 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.

EXCEPTION: Where approved by the authority having jurisdiction, a site-specific geotechnical report is not required where prior evaluations of nearby sites with similar soil conditions provide direction relative to the proposed construction.

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 all of the following, as applicable:

1. The determination of dynamic seismic lateral earth pressures on basement and retaining walls caused by design earthquake ground motions.
2. The potential for liquefaction and soil strength loss 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~~8~~ or (2) the mapped value of MCE_G peak ground acceleration adjused for site class effects, PGA_M, provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76 for the site class determined in accordance with the site class requirements of Section 11.4.2. Where the soil properties are not known in sufficient detail to determine the site class and the default site class requirements of Section 11.4.2.1 apply, peak ground acceleration shall be determined from the mapped values of PGA_M shown in Figs. 22-9 through 22-13.~~peak ground acceleration PGA_M, from Eq. (11.8-1).~~

~~$$PGA_M = F_{PGA} \cdot PGA \quad (11.8\text{-}1)$$~~

~~Where~~

~~PGA_M = MCE_G peak ground acceleration adjusted for site class effects.~~

~~PGA = Mapped MCE_G peak ground acceleration shown in Figs. 22-9 through 22-13.~~

~~F_{PGA} = Site coefficient from Table 11.8-1.~~

3. Assessment of potential consequences of liquefaction 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.
4. Discussion of mitigation measures such as, but not limited to, 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.

~~**TABLE 11.8-1 Site Coefficient F_{PGA}**~~

	~~Mapped Maximum Considered Geometric Mean (MCE_G) Peak Ground Acceleration, PGA~~					
~~Site Class~~	~~PGA ≤ 0.1~~	~~PGA = 0.2~~	~~PGA = 0.3~~	~~PGA = 0.4~~	~~PGA = 0.5~~	~~PGA ≥ 0.6~~
~~A~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~
~~B~~	~~0.9~~	~~0.9~~	~~0.9~~	~~0.9~~	~~0.9~~	~~0.9~~
~~C~~	~~1.3~~	~~1.2~~	~~1.2~~	~~1.2~~	~~1.2~~	~~1.2~~
~~D~~	~~1.6~~	~~1.4~~	~~1.3~~	~~1.2~~	~~1.1~~	~~1.1~~
~~E~~	~~2.4~~	~~1.9~~	~~1.6~~	~~1.4~~	~~1.2~~	~~1.1~~
~~F~~	~~See Section 11.4.8~~					

~~Note: Use straight-line interpolation for intermediate values of PGA.~~

11.9 VERTICAL GROUND MOTIONS FOR SEISMIC DESIGN

11.9.1 General.

If the option to incorporate the effects of vertical seismic ground motions is exercised in lieu of the requirements of Section 12.4.2.2, the requirements of this section are permitted to be used in the determination of the vertical design earthquake ground motions. The requirements of Section 11.9 shall only apply to structures in Seismic Design Categories C, D, E, and F.

11.9.2 MCE_R Vertical Response Spectrum.

Where a vertical response spectrum is required by this standard and site-specific procedures are not used, the MCE_R vertical response spectral acceleration, S_{aMv}, shall be developed as follows:

1. For vertical periods less than or equal to 0.025 s, S_{aMv} shall be determined in accordance with Eq. (11.9-1) as follows:

$$S_{aMv} = 0.3 C_v S_{MS} \tag{11.9-1}$$

2. For vertical periods greater than 0.025 s and less than or equal to 0.05 s, S_{aMv} shall be determined in accordance with Eq. (11.9-2) as follows:

$$S_{aMv} = 20 C_v S_{MS} (T_v - 0.025) + 0.3 C_v S_{MS} \tag{11.9-2}$$

3. For vertical periods greater than 0.05 s and less than or equal to 0.15 s, S_{aMv} shall be determined in accordance with Eq. (11.9-3) as follows:

$$S_{aMv} = 0.8 C_v S_{MS} \tag{11.9-3}$$

4. For vertical periods greater than 0.15 s and less than or equal to 2.0 s, S_{aMv} shall be determined in accordance with Eq. (11.9-4) as follows:

$$S_{aMv} = 0.8 C_v S_{MS} \left(\frac{0.15}{T_v} \right)^{0.75} \tag{11.9-4}$$

Where

C_v = is defined in terms of S_S in Table 11.9-1,

S_{MS} = the MCE_R spectral response acceleration parameter at short periods, and

T_v = the vertical period of vibration.

TABLE 11.9-1 Values of Vertical Coefficient C_v

Mapped MCE_R Spectral Response Parameter at Short Periods[a]	Site Class A, B	Site Class BC	Site Class C	Site Class CD	Site Class D, DE, E, F
$S_{MS} \geq 2.0$	0.9	1.0	1.3	1.4	1.5

$S_{MS} = 1.0$	0.9	1.0	1.1	1.2	1.3
$S_{MS} = 0.6$	0.8	0.9	1.0	1.05	1.1
$S_{MS} = 0.3$	0.75	0.8	0.9	0.95	1.0
$S_{MS} \leq 0.2$	0.7	0.75	0.8	0.85	0.9

[a]Use straight-line interpolation for intermediate values of S_{MS}.

~~**TABLE 11.9-1 Values of Vertical Coefficient C_v**~~

~~Mapped MCE$_R$ Spectral Response Parameter at Short Periods[a]~~	~~Site Class A, B~~	~~Site Class C~~	~~Site Class D, E, F~~
$S_S \geq 2.0$	~~0.9~~	~~1.3~~	~~1.5~~
$S_S = 1.0$	~~0.9~~	~~1.1~~	~~1.3~~
$S_S = 0.6$	~~0.9~~	~~1.0~~	~~1.1~~
$S_S = 0.3$	~~0.8~~	~~0.8~~	~~0.9~~
$S_S \leq 0.2$	~~0.7~~	~~0.7~~	~~0.7~~

~~[a]Use straight-line interpolation for intermediate values of S_S.~~

S_{aMv} shall not be less than one-half of the corresponding S_{aM} for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.

For vertical periods greater than 2.0 s, S_{aMv} shall be developed from a site-specific procedure; however, the resulting ordinate of S_{aMv} shall not be less than one-half of the corresponding S_a for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.

In lieu of using the above procedure, a site-specific study is permitted to be performed to obtain S_{aMv} at vertical periods less than or equal to 2.0 s, but the value so determined shall not be less than 80% of the S_{aMv} value determined from Eqs. (11.9-1) through (11.9-4).

11.9.3 Design Vertical Response Spectrum.

The design vertical response spectral acceleration, S_{av}, shall be taken as two-thirds of the value of S_{aMv} determined in Section 11.9.2.

11.10 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.

CHAPTER 20 SITE CLASSIFICATION PROCEDURE FOR SEISMIC DESIGN

20.1 SITE CLASSIFICATION

The site soil shall be classified in accordance with Table 20.3-1 and Section 20.3 based on the upper 100 ft (30 m) of the site profile. Where site-specific data are not available to a depth of 100 ft (30 m), appropriate soil properties are permitted to be estimated by the registered design professional preparing the soil investigation report based on known geologic conditions. Where the soil properties are not known in sufficient detail to determine the site class, the more critical site conditions of Site Class C, Site Class CD, Site Class D and Site Class DE Site~~Class D, subject to the requirements of Section 11.4.4~~, shall be used unless the Authority Having Jurisdiction or geotechnical data determine that Site Class E or F soils are present at the site. Site Classes A and B shall not be assigned to a site if there is more than 10 ft (3.1 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

Table 20.3-1 Site Classification

Site Class	$\overline{v}_s$.	$\overline{N}$ or $\overline{N}_{ch}$.	$\overline{s}_u$
A. Hard rock	$\geq$ 5,000 fps	NA	NA
B. Rock	3,000 to 5,000 ft/s	NA	NA
BC. Soft Rock	2,100 to 3,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,450 to 2,100 ft/s ~~1,200 to 2,500 ft/s~~	> 50 blows / ft	$> 2{,}000$ lb / ft^2
CD. Very Stiff Soil	1,000 to 1,450 ft/s	> 50 blows/ft	> 2,000 lb/ft^2
D. Stiff soil	700 to 1,000 ft/s ~~600 to 1,200 ft/s~~	15 to 50 blows/ft	1,000 to 2,000 lb/ft^2
DE. Soft Soil	500 to 700 ft/s	15 to 50 blows/ft	1,000 to 2,000 lb/ft^2
E. Soft clay soil	<500 ft/s ~~< 600 ft/s~~	15 blows / ft	1,000 lb / ft^2
	Any profile with more than 10 ft of soil that has the following characteristics: — Plasticity index $PI > 20$, — Moisture content $w \geq 40\%$, — Undrained shear strength $\overline{s}_u$ 500 lb / ft^2		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

Note: For SI: 1 ft = 0.3048 m ; 1 ft / s = 0.3048 m / s ; 1 lb / ft^2 = 0.0479 kN / m^2.

20.2 SITE RESPONSE ANALYSIS FOR SITE CLASS F SOIL

A site response analysis in accordance with Section 21.1 shall be provided for Site Class F soils, unless any of the exceptions to Section 20.3.1 are applicable.

20.3 SITE CLASS DEFINITIONS

Site class types shall be assigned in accordance with the definitions provided in Table 20.3-1 and this section.

20.3.1 Site Class F.

Where any of the following conditions is satisfied, the site shall be classified as Site Class F and a site response analysis in accordance with Section 21.1 shall be performed.

1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.
   EXCEPTION: For structures that have fundamental periods of vibration equal to or less than 0.5 s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 ~~and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2~~.

2. Peats and/or highly organic clays [$H > 10$ ft ($H > 3$ m)] of peat and/or highly organic clay where H = thickness of soil.
3. Very high plasticity clays [$H > 25$ ft ($H > 7.6$ m) with $PI > 75$] in a soil profile that would otherwise be classified as Site Class CD, D, DE or E.
   EXCEPTION: Site response analysis is not required for this clay category for Seismic Design Category A and B sites.~~provided that both of the following requirements are satisfied: (i) values of F_a and F_v are obtained from Tables 11.4-1 and 11.4-2 for Site Class D or E multiplied by a factor that varies linearly from 1.0 at $PI = 75$ to 1.3 for $PI = 125$ and is equal to 1.3 for $PI > 125$; and (ii) the resulting values of S_{DS} and S_{D1} obtained using the scaled factors F_a and F_v do not exceed the upper bound values for Seismic Design Category B given in Tables 11.6-1 and 11.6-2.~~

4. Very thick soft/medium stiff clays [$H > 120$ ft ($H > 37$ m)] with $s_u < 1{,}000$ psf ($s_u < 50$ kPa).
   EXCEPTION: Site response analysis is not required for this clay category for Seismic Design Category A and B sites.~~provided that both of the following requirements are satisfied: (i) values of F_a and F_v are obtained from Tables 11.4-1 and 11.4-2 for Site Class E; and (ii) the resulting values of S_{DS} and S_{D1} using the factors F_a and F_v do not exceed the upper bound values for Seismic Design Category B given in Tables 11.6-1 and 11.6-2.~~

20.3.2 Soft Clay Site Class DE~~E~~.

Where a site does not qualify under the criteria for Site Class F and there is a total thickness of soft clay greater than 10 ft (3 m) where a soft clay layer is defined by $s_u < 500$ psf ($s_u < 25$ kPa), $w \geq 40\%$, and $PI > 20$, it shall be classified as Site Class DE.

20.3.3 Site Classes C, CD, D, DE and E.

The existence of Site Class C, CD, D, DE and E soils shall be classified by using one of the following three methods with $\overline{v}_s$, $\overline{N}$, and $\overline{s}_u$ computed in all cases as specified in Section 20.4:

1. $\overline{v}_s$ for the top 100 ft (30 m) ($\overline{v}_s$ method).
2. $\overline{N}$ for the top 100 ft (30 m) ($\overline{N}$ method).
3. $\overline{N}_{ch}$ for cohesionless soil layers ($PI < 20$) in the top 100 ft (30 m) and $\overline{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft (30 m) ($\overline{s}_u$ method). Where the $\overline{N}_{ch}$ and $\overline{s}_u$ criteria differ, the site shall be assigned to the category with the softer soil.

20.3.4 Shear Wave Velocity for Site Classes B and BC.

The shear wave velocity for rock, Site Classes B and BC, shall be either measured on site or estimated by a geotechnical engineer, engineering geologist, or seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

20.3.5 Shear Wave Velocity for Site Class A.

The hard rock, Site Class A, category shall be supported by shear wave velocity measurement either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft (30 m), surficial shear wave velocity measurements are permitted to be extrapolated to assess $\overline{v}_s$.

20.4 DEFINITIONS OF SITE CLASS PARAMETERS

The definitions presented in this section shall apply to the upper 100 ft (30 m) of the site profile. Profiles containing distinct soil and rock layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft (30 m). Where some of the n layers are cohesive and others are not, k is the number of cohesive layers and m is the number of cohesionless layers. The symbol i refers to any one of the layers between 1 and n.

20.4.1 $\overline{v}_s$, Average Shear Wave Velocity.

$\overline{v}_s$ shall be determined in accordance with the following formula:

$$\overline{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \quad (20.4\text{-}1)$$

where

d_i = the thickness of any layer between 0 and 100 ft (30 m);

v_{si} = the shear wave velocity in ft/s (m/s); and

$\sum_{n}^{i=1} d_i$ = 100 ft (30 m).

20.4.2 $\overline{N}$, Average Field Standard Penetration Resistance and $\overline{N}_{ch}$, Average Standard Penetration Resistance for Cohesionless Soil Layers.

$\overline{N}$ and $\overline{N}_{ch}$ shall be determined in accordance with the following formulas:

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \quad (20.4\text{-}2)$$

where N_i and d_i in Eq. (20.4-2) are for cohesionless soil, cohesive soil, and rock layers.

$$\overline{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \quad (20.4\text{-}3)$$

where N_i and d_i in Eq. (20.4-3) are for cohesionless soil layers only and

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

where

d_s is the total thickness of cohesionless soil layers in the top 100 ft (30 m).

N_i is the standard penetration resistance (ASTM D1586) not to exceed 100 blows/ft (305 blows/m) as directly measured in the field without corrections.

Where refusal is met for a rock layer, N_i shall be taken as 100 blows / ft (305 blows / m).

20.4.3 $\overline{s}_u$, Average Undrained Shear Strength.

$\overline{s}_u$ shall be determined in accordance with the following formula:

$$\overline{s}_u = \frac{d_c}{\sum_{k}^{i=1} \frac{d_i}{s_{ui}}} \quad (20.4\text{-}4)$$

where

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

d_c = the total thickness of cohesive soil layers in the top 100 ft (30 m);

PI = the plasticity index as determined in accordance with ASTM D4318;

w = the moisture content in percent as determined in accordance with ASTM D2216; and

s_{ui} = the undrained shear strength in psf (kPa), not to exceed 5,000 psf (240 kPa) as determined in accordance with ASTM D2166 or ASTM D2850.

20.5 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.

CHAPTER 21 SITE-SPECIFIC GROUND MOTION PROCEDURES FOR SEISMIC DESIGN

21.1 SITE RESPONSE ANALYSIS

The requirements of Section 21.1 shall be satisfied where site response analysis is performed or required by Section 11.4.7. The analysis shall be documented in a report.

21.1.1 Base Ground Motions.

An MCE_R response spectrum shall be developed for bedrock, using the procedure of Sections 11.4.6 or 21.2. Unless a site-specific ground motion hazard analysis described in Section 21.2 is carried out, the MCE_R rock response spectrum shall be developed using the procedure of Section 11.4.6, assuming Site Class B. If bedrock consists of Site Class A, the spectrum shall be adjusted using the site coefficients in Section 11.4.3 unless other site coefficients can be justified. At least five recorded or simulated horizontal ground motion acceleration time histories shall be selected from events that have magnitudes and fault distances that are consistent with those that control the MCE_R ground motion. Each selected time history shall be scaled so that its response spectrum is, on average, approximately at the level of the MCE_R rock response spectrum over the period range of significance to structural response.

21.1.2 Site Condition Modeling.

A site response model based on low strain shear wave velocities, nonlinear or equivalent linear shear stress–strain relationships, and unit weights shall be developed. Low strain shear wave velocities shall be determined from field measurements at the site or from measurements from similar soils in the site vicinity. Nonlinear or equivalent linear shear stress–strain relationships and unit weights shall be selected on the basis of laboratory tests or published relationships for similar soils. The uncertainties in soil properties shall be estimated. Where very deep soil profiles make the development of a soil model to bedrock impractical, the model is permitted to be terminated where the soil stiffness is at least as great as the values used to define Site Class D in Chapter 20. In such cases, the MCE_R response spectrum and acceleration time histories of the base motion developed in Section 21.1.1 shall be adjusted upward using site coefficients in Section 11.4.3 consistent with the classification of the soils at the profile base.

21.1.3 Site Response Analysis and Computed Results.

Base ground motion time histories shall be input to the soil profile as outcropping motions. Using appropriate computational techniques that treat nonlinear soil properties in a nonlinear or equivalent-linear manner, the response of the soil profile shall be determined and surface ground motion time histories shall be calculated. Ratios of 5% damped response spectra of surface ground motions to input

base ground motions shall be calculated. The recommended surface MCE_R ground motion 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 (calculated period by period) obtained from the site response analyses. The recommended surface ground motions that result from the analysis shall reflect consideration of sensitivity of response to uncertainty in soil properties, depth of soil model, and input motions.

21.2 RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION HAZARD ANALYSIS

The requirements of Section 21.2 shall be satisfied where a ground motion hazard analysis is performed or required by Section 11.4.7. The ground motion hazard analysis shall account for the regional tectonic setting, geology, and seismicity; the expected recurrence rates and maximum magnitudes of earthquakes on known faults and source zones; the characteristics of ground motion attenuation near source effects, if any, on ground motions; and the effects of subsurface site conditions on ground motions. The characteristics of subsurface site conditions shall be considered either using attenuation relations that represent regional and local geology or in accordance with Section 21.1. The analysis shall incorporate current seismic interpretations, including uncertainties for models and parameter values for seismic sources and ground motions. If the spectral response accelerations predicted by the attenuation relations do not represent the maximum response in the horizontal plane, then the response spectral accelerations computed from the hazard analysis shall be scaled by factors to increase the motions to the maximum response. If the attenuation relations predict the geometric mean or similar metric of the two horizontal components, then the scale factors shall be 1.1 for periods less than or equal to 0.2 s, 1.3 for a period of 1.0 s, and 1.5 for periods greater than or equal to 5.0 s, unless it can be shown that other scale factors more closely represent the maximum response, in the horizontal plane, to the geometric mean of the horizontal components. Scale factors between these periods shall be obtained by linear interpolation. The analysis shall be documented in a report.

21.2.1 Probabilistic (MCE_R) Ground Motions.

The probabilistic spectral response accelerations shall be taken as the spectral response accelerations in the direction of maximum horizontal response represented by a 5% damped acceleration response spectrum that is expected to achieve a 1% probability of collapse within a 50-year period. ~~For the purpose of this standard, ordinates of the probabilistic ground motion response spectrum shall be determined by either Method 1 of Section 21.2.1.1 or Method 2 of Section 21.2.1.2.~~

~~21.2.1.1 Method 1.~~

~~At each spectral response period for which the acceleration is computed, ordinates of the probabilistic ground motion response spectrum shall be determined as the product of the risk coefficient, C_R, and the spectral response acceleration from a 5% damped acceleration response spectrum that has a 2%~~

~~probability of exceedance within a 50-year period. The value of the risk coefficient, C_R, shall be determined using values of C_{RS} and C_{R1} from Figs. 22-18 and 22-19, respectively. At spectral response periods less than or equal to 0.2 s, C_R shall be taken as equal to C_{RS}. At spectral response periods greater than or equal to 1.0 s, C_R shall be taken as equal to C_{R1}. At response spectral periods greater than 0.2 s and less than 1.0 s, C_R shall be based on linear interpolation of C_{RS} and C_{R1}.~~

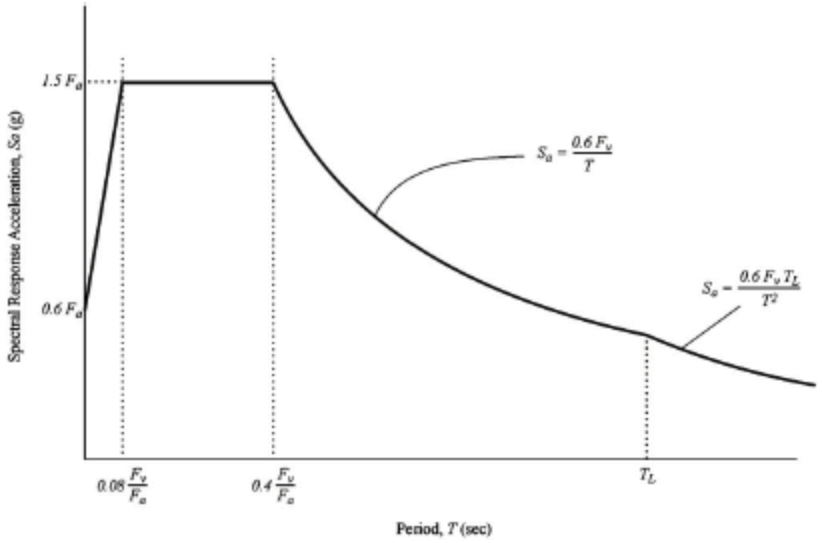
~~21.2.1.2 Method 2.~~

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 hazard curve with a lognormal probability density function representing the collapse fragility (i.e., probability of collapse as a function of spectral response acceleration). The ordinate of the probabilistic ground motion response spectrum at each period shall achieve a 1% probability of collapse within a 50-year period for a collapse fragility that has (1) a 10% probability of collapse at said ordinate of the probabilistic ground motion response spectrum and (2) a logarithmic standard deviation value of 0.6.

21.2.2 Deterministic (MCE_R) Ground Motions.

The deterministic spectral response acceleration at each period shall be calculated as an 84th-percentile 5% damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for the characteristic earthquakes on all known active faults within the region shall be used. If the largest spectral response acceleration of the resulting deterministic ground motion response spectrum is less than 1.67 g~~1.5F_a~~, then this response spectrum shall be scaled by a single factor such that the maximum response spectral acceleration equals 1.67 g.~~1.5F_a.~~ ~~For Site Classes A, B, C and D, F_a shall be determined using Table 11.4.1 with the value of S_s taken as 1.5; for Site Class E, F_a shall be taken as 1.0.The ordinates of the deterministic ground motion response spectrum shall not be taken as lower than the corresponding ordinates of the response spectrum determined in accordance with Fig. 21.2-1. For the purposes of calculating the ordinates~~

~~(i) for Site Classes A, B or C: F_a and F_v shall be determined using Tables 11.4-1 and 11.4-2, with the value of S_S taken as 1.5 and the value of S_1 taken as 0.6;~~
~~(ii) for Site Class D: F_a shall be taken as 1.0, and F_v shall be taken as 2.5; and~~
~~(iii)for Site Classes E and F: F_a shall be taken as 1.0, and F_v shall be taken as 4.0.~~



~~**FIGURE 21.2-1 Deterministic Lower Limit on** MCE_R **Response Spectrum**~~

21.2.3 Site-Specific MCE_R .

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 risk-targeted MCE_R response spectrum provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76 for the site location of interest and site class determined in accordance with the site class requirements of Section 11.4.2.

21.3 DESIGN RESPONSE SPECTRUM

The design spectral response acceleration at any period shall be determined from Eq. (21.3-1):

$$S_a = \frac{2}{3} S_{aM} \qquad (21.3\text{-}1)$$

where S_{aM} is the MCE spectral response acceleration obtained from Section 21.1 or 21.2.

The design spectral response acceleration at any period shall not be taken as less than 67 percent of the risk-targeted MCE_R response spectrum provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76 for the site location of interest and site class determined in accordance with the site class requirements of Section 11.4.2.~~80% of S_a determined in accordance with Section 11.4.6, where F_a and F_v are determined as follows:~~

~~(i) for Site Class A, B, and C: F_a and F_v are determined using Tables 11.4-1 and 11.4-2, respectively;~~

~~(ii) for Site Class D: F_a is determined using Table 11.4-1, and F_v is taken as 2.4 for $S_1 <$ 0.2 or 2.5 or $S_1 >=$ 0.2; and~~

~~(iii) for Site Class E: F_a is determined using Table 11.4-1 for $S_S <$ 1.0 or taken as 1.0 for $S_S >=$ 1.0, and F_v is taken as 4.2 for $S_1 <$ 0.1 or 4.0 for $S_1 >=$ 0.1.~~

For sites classified as Site Class F requiring site-specific analysis in accordance with Section 11.4.7, the design spectral response acceleration at any period shall not be less than 67 percent of the risk-targeted MCE_R response spectrum provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76 for the site location of interest and Site Class DE site conditions.~~80% of S_a determined for Site Class E in accordance with Section 11.4.5.~~

EXCEPTION: Where a different site class can be justified using the site-specific classification procedures in accordance with Section 20.3.3, a lower limit of 100~~80~~% of S_a for the justified site class shall be permitted to be used.

21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TS_a, for periods from 1 to 2 s for sites with $v_{s,30} > 1,200$ ft / s ($v_{s,30} > 365.76$ m / s) and for periods from 1 to 5 s for sites with $v_{s,30} \le 1,200$ ft / s ($v_{s,30} \le 365.76$ m / s). The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 100%~~80%~~ of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

For use with the equivalent lateral force procedure, the site-specific spectral acceleration, S_a, at T shall be permitted to replace S_{D1} / T in Eq. (12.8-3) and $S_{D1}T_L / T^2$ in Eq. (12.8-4). The parameter S_{DS} calculated per this section shall be permitted to be used in Eqs. (12.8-2), (12.8-5), (15.4-1), and (15.4-3). The mapped value of S_1 shall be used in Eqs. (12.8-6), (15.4-2), and (15.4-4).

21.5 MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN (MCE_G) PEAK GROUND ACCELERATION

21.5.1 ~~Probabilistic~~ MCE_G Peak Ground Acceleration.

The MCE_G ~~probabilistic geometric mean~~ peak ground acceleration parameter PGA_M shall be taken as the greater of the geometric mean peak ground acceleration with a 2% probability of exceedance within a 50-year period and

~~**21.5.2 Deterministic MCE_R Peak Ground Acceleration.**~~

~~The deterministic geometric mean peak ground acceleration shall be calculated as the largest 84th-percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region. The deterministic geometric mean peak ground acceleration shall not be taken as lower than $0.5F_{PGA}$, where F_{PGA} is determined using Table 11.8-1 with the value of PGA taken as 0.5g.~~

~~**21.5.23 Site-Specific MCE_R Peak Ground Acceleration.**~~

~~The site-specific MCE_R peak ground acceleration, PGA_M, shall be taken as the lesser of the probabilistic geometric mean peak ground acceleration of Section 21.5.1 and the deterministic geometric mean peak ground acceleration of Section 21.5.2. The site-specific MCE_R peak ground acceleration shall not be taken as less than~~ 100%~~80%~~ of the value of PGA_M required by Section 11.8.3~~determined from Eq. (11.8-1).~~

21.6 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.

CHAPTER 22 SEISMIC GROUND MOTION, LONG-PERIOD TRANSITION, AND RISK COEFFICIENT MAPS

Contained in this chapter are Figs. 22-1 through 22-8, which provide the risk-targeted maximum considered earthquake (MCE_R) ground motion parameters S_{MS} and S_{M1}, for default site conditions as defined in Section 11.4.2.1~~SS and S_1; Figs. 22-18 and 22-19, which provide the risk coefficients C_{RS} and C_{R1}~~; Figs. 22-9 through 22-13 which provide the peak ground acceleration parameter, PGA_M, for default site conditions as defined in Section 11.4.2.~~ and Figs. 22-14 through 22-17, which provide the long-period transition periods T_L for use in applying the seismic provisions of this standard~~. S_{MS}~~S_S~~ is the mapped MCE_R , 5% damped, spectral response acceleration parameter at short periods~~as defined in Section 11.4.2~~. S_{M1}~~S_1~~ is the mapped MCE_R , 5% damped, spectral response acceleration parameter at a period of 1 s.~~as defined in Section 11.4.2. C_{RS} is the mapped risk coefficient at short periods used in Section 21.2.1.1. C_{R1} is the mapped risk coefficient at a period of 1 s used in Section 21.2.1.1.~~ PGA_M is the mapped value of geometric mean (MCE_G) peak ground acceleration. Mapped values of S_{MS}, S_{M1} and PGA_M are provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76 for User-specified site location and site class determined in accordance with the site class requirements of Section 11.4.2.

Also contained in this chapter are Figs. 22-14 through 22-17, which provide the long-period transition periods T_L for use in applying the seismic provisions of this standard. Maps of the long-period transition periods, T_L, for Guam and the Northern Mariana Islands and for American Samoa are not provided because parameters have not yet been developed for those islands via the same deaggregation computations done for the other U.S. regions. Therefore, as in previous editions of this standard, the parameter. T_L shall be 12 s for those islands. T_L is the mapped long-period transition period used in Section 11.4.5~~6~~.

These maps were prepared by the United States Geological Survey (USGS) in collaboration with the Building Seismic Safety Council (BSSC) Provisions Update Committee and the American Society of Civil Engineers (ASCE) 7 Seismic Subcommittee and have been updated for this standard.

~~Maps of the long-period transition periods, T_L, for Guam and the Northern Mariana Islands and for American Samoa are not provided because parameters have not yet been developed for those islands via the same deaggregation computations done for the other U.S. regions. Therefore, as in previous editions of this standard, the parameter T_L shall be 12 s for those islands.~~

~~Also contained in this chapter are Figs. 22-9 through 22-13, which provide the maximum considered earthquake geometric mean (MCE_G) peak ground accelerations as a percentage of g~~

The following is a list of figures contained in this chapter:

Fig. 22-17 Mapped Long-Period Transition Period, T_L (s), for Puerto Rico and the United States Virgin Islands

~~Fig. 22-18 Mapped Risk Coefficient at 0.2-s Spectral Response Period, C_{RS}~~

~~Fig. 22-19 Mapped Risk Coefficient at 1.0-s Spectral Response Period, C_{R1}~~

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.

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JiQiu Yuan

National Institute of Building Sciences

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P17 - P17 ballot on SDC

Status

Finalized

Scope

Chapter 11: Update Seismic Design Category Definition, add Default Site Class Definition, and modify Sections 11.4 through 11.6 to refer to Seismic Design Category Maps in lieu of Tables 11.6-1 and 11.6-2. Insert Seismic Design Category Maps in Section 11.6 for use by the designer. The information currently provided in Sections 11.4 through 11.6 outlining the procedures to determine the SDC has been moved to the Commentary for reference.

Section 12.4.1.1 Simplified Design Procedure: update reference to Chapter 11 identifying how Seismic Design Category is to be determined to point to new Chapter 11 Seismic Design Category Maps. The existing commentary for this section discusses the rationale behind the Simplified Design Procedure without reference to Seismic Design Category so does not require modification.

11.6 Commentary: update commentary to include discussion on the need and development of Seismic Design Category maps in Chapter 11, Section 11.6. Commentary also includes recommended procedures for comparison of future SDC maps with currently published versions and parameters to consider when choosing to update SDCs or not.

Note: All changes are highlighted in red fond in the proposal .

Proponent

Julie Furr

Voting Period

04-16-2018 11:00 AM – 05-07-2018 3:00 AM

Proponent Comment Period

05-07-2018 9:00 AM – 05-21-2018 11:59 PM

Supporting Files

P17_No3_SDC_Proposal.docx 2018-04-16 12:13:17

Proposal Updates

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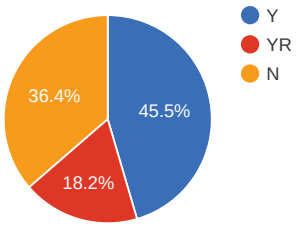
Vote Summary for P17 - P17 ballot on SDC

Results Report

Total Voting	Voted	50% Rule	Yes	Yes with Reservations	No	Not Voting	Online Ballot	PUC Meeting Final
15	11	Meet	5	2	4	0	Fail 	Revise and reballot.

Vote and Comment Summary

Download Votes
Download Votes and Responses

Vote Key

Yes — Yes with reservations
No — **NV** Not Voting

Response Key

Persuasive — Non-Persuasive
NR Non-Responsive — Editorial/Persuasive

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
Bonneville				My No vote is for several reasons: First, I continue to oppose the idea of assigning SDC independent of site class. In general, in seismically active areas this has not been a problem since the SDC concept was introduced. If simplicity is a goal in certain parts of the country, I would prefer that such locations adopt ordinances specifying that the either the controlling SDC for that region be used throughout the the region, or that a conservative site class be used throught that region.	Aside from the first item, where I will need to be found non-persuasive, I would change my vote to yes, if the terms of this proposal were consistent with those proposed in the MPS Chapter 11 concept proposal and and if sample SDC maps were shown based on ASCE 7-16 design maps.		

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
				Second, this proposal will conflict in many ways with Charlie Kircher's proposal on multi-period spectra when it is submitted. For example, the MPS proposal will remove the site coefficients and redefine the mapped spectral response parameters using S_{MS} and S_{M1}. Since both P17 and the PUC have given their approval in concept to the MPS proposal, I think it would be preferable to write this proposal in terms that will be consistent with the MPS Chapter 11 proposed revisions. Otherwise, it will need to be substantially rewritten later. Third, the proposal refers to SDC maps (Figures 11.6.1 and 11.6.2) that are not provided. I assume therefore that we're voting on a map *concept* described in the commentary and that the maps will be provided for the PUC vote (and the concept will be consistent with the MPS proposal). Nevertheless it seems odd for Project 17 members to be voting on SDC maps without see them. Fourth, the proposal refers to Tables C11.6-1 and 11.6-2, which are also not provided.			
Dolan							
Enfield							
Furr							
Hamburger							

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
Harris		General	General	While I do understand and sympathize with the frustration with changes in Seismic Design Category from one code to the next, I do not believe this is the proper solution. The need for seismic detailing should depend first on the amplitude of ground motion that we expect a structure to survive and second on the degree of confidence that we want in that expectation. This means that separating the effect of site specific amplification of ground motion from the decision is fundamentally wrong. The second feature I mention has been watered down over the years, such that it only affects Risk Category IV structures at higher levels of ground motion, but nonetheless, this proposal would also make that situation worse, in my opinion.	I would support a guideline document advising authorities having jurisdiction how they might simplify the application of the code by creating local adopting ordinances that reduce the options for Seismic Design Category in a jurisdiction where the range of site conditions is well understood.		
Holmes		16	24	I voted Yes because I am in favor of the mapping using a default site class. It is unclear to me why there are changes to section 11.4.4. Is this related to the default site classes used for the SDC maps? If it simply a change in the rules for poorly understood sites, it should not be in this proposal. However the use of a default site class to establish the first maps should be described in the commentary. The second paragraph of page 28 starting with line 14 must be rewritten in the past tense to indicated how the SDC maps were established, and what default site class was used.	See above		

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
Hooper		General	General	My "no" vote is based mainly on the following two issues: 1. I need to see the maps before approving this. Without them it is impossible to know how significant the impact will be. 2. The proposed changes will result in added conservatism to seismic design (since the basis for the zones will be Site Class D), merely to solve a problem that only impacts small areas of the country, which are generally on the SDC C/D boundary.	To change my "no" vote to "yes", I suggest the proposed change allow for a site-specific determination of SDC. The current proposal would remain the standard approach and would mitigate the yo-yo effect for many designers. However, the original provisions for determining SDC should be kept, but moved to a new section (in either Chapter 11 or 21). Then if an engineer wants to remove unneeded conservatism they can calculate their SDC based on the actual site class.		
Kircher				The SDC proposal contains changes to sections of Chapter 11 other than Section 11.6 (Seismic Design Category) that are contrdictory to changes to Chapter 11 already approved by P17 as part of the MPRS proposal (e.g., new site class defintions, new default site class defintion and elimination of site coefficient tables, etc.) I would reconsider my No vote provided that the SDC proposal was revised to be limited to Section 11.6 and be otherwise fully complient with changes to Chapter 11 already approved by P17 as part of th MPRS proposal.	See attached current draft copy of MPRS proposal (Chapter 11) now being developed for PUC ballot	FILE	

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
Pekelnicky				I support almost everything about this proposal. However, I do not agree with providing a suggestion of how to develop maps in the future. That should not be included in this proposal, as I do not believe it is appropriate commentary. Future consensus committees cannot be directed what to do. Further, the 10% number is somewhat arbitrary. Since it is "judgement" based, it would be better, in my opinion, to say nothing at all about how to adjust the maps and allow the future consensus committee to make the decision on how to do so.	Remove the final proposed portion of the commentary which begins with "As previously described, the..." and ends with "...should remain unchanged."		
Siu							

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FEMA

nehrp

PROPOSAL P17-3-Rev. 0-2018-04-11

Seismic Design Category Maps

Submitted by: Julie C. Furr, PE
901-426-6010
jfurr@rimkus.com

Original Proposal Submission date: 4/11/2018

SCOPE: Part 1 and Corresponding Part 2 Commentary

Chapter 11: Update Seismic Design Category Definition, add Default Site Class Definition, and modify Sections 11.4 through 11.6 to refer to Seismic Design Category Maps in lieu of Tables 11.6-1 and 11.6-2. Insert Seismic Design Category Maps in Section 11.6 for use by the designer. The information currently provided in Sections 11.4 through 11.6 outlining the procedures to determine the SDC has been moved to the Commentary for reference.

Section 12.4.1.1 Simplified Design Procedure: update reference to Chapter 11 identifying how Seismic Design Category is to be determined to point to new Chapter 11 Seismic Design Category Maps. The existing commentary for this section discusses the rationale behind the Simplified Design Procedure without reference to Seismic Design Category so does not require modification.

11.6 Commentary: update commentary to include discussion on the need and development of Seismic Design Category maps in Chapter 11, Section 11.6. Commentary also includes recommended procedures for comparison of future SDC maps with currently published versions and parameters to consider when choosing to update SDCs or not.

REASON FOR PROPOSAL:

Seismic parameters used by stakeholders have oscillated in response to updates in the National Seismic Hazard Maps (NSHM) and ASCE 7 provisions, creating what is commonly referred to as the yo-yo effect. Often these oscillations are within the margin of error for ground motion calculations and may simply be the result of tweaks in GMPEs or other modeling techniques. However, in some regions this change is enough to jump the numerical boundaries between different Seismic Design Categories (SDCs) as currently established in Tables 11.6-1 and 11.6-2. Because system detailing requirements and limitations are established through the designated SDC, these variations mean systems have been allowed/disallowed/allowed as new ASCE 7 versions are published. Aside from stakeholder frustration,

this has real consequential impacts from loss of public confidence in ASCE 7 seismic provisions to lack of established design experience by engineers due to changing design requirements.

SDC maps are proposed as a means to establish stability and minimize the practical effects of oscillations on the design community, without artificially constraining ground motion values established by science. SDC maps will identify geographical regions experiencing significant variations in revised NSHM ground motion mapped values, which would have otherwise resulted in changing SDCs. At these locations, a subjective review will be performed to determine if the underlying cause(s) of ground motion variation justifies changing the SDC in that region. If the change is a) within the reported margin of error, or b) due to tweaks in scientific methodology, the SDC will likely remain as previously published. Alternately, if the change is due to significant new scientific information or understanding, the SDC will likely be updated. The SDC map will provide a stable set of system detailing requirements and limitations, similar to the old zone maps, that stakeholders will grow to be familiar with. Where changes do occur, a justifiable rationale can be provided to stakeholders defending and explaining the change in that region.

IT Consensus/Notes:

The SDC Working group reached a consensus decision that there is a significant potential benefit in SDC maps, that site specific data should not be allowed to override mapped SDC values, and on the basic methodology to develop the SDC maps and identify changes with subsequent NSHM revisions. After discussions on how to incorporate Risk Categories, the working group determined two SDC maps were required, 1 each for Risk Categories I/II/III and Risk Category IV, reflecting the two existing columns in Tables 11.6-1 and 11.6-2. Although the group reached a consensus decision to use a “default site class” to develop the maps, several concerns continue to be voiced calling for a mechanism to allow site class variations in the SDC map. The working group was unable to develop such a mechanism and ultimately agreed to remain with a “default site class” as the conservative approach that may require more ductile systems in less seismically active border regions.

CHAPTER 11 SEISMIC DESIGN CRITERIA

11.1 GENERAL

11.1.1 Purpose. Chapter 11 presents criteria for the design and construction of buildings and other structures subject to earthquake ground motions. The specified earthquake loads are based upon postelastic energy dissipation in the structure. Because of this fact, the requirements for design, detailing, and construction shall be satisfied, even for structures and members for which load combinations that do not include earthquake loads indicate larger demands than combinations that include earthquake loads.

11.1.2 Scope. Every structure and portion thereof, including nonstructural components, shall be designed and constructed to resist the effects of earthquake motions as prescribed by the seismic requirements of this standard. Certain nonbuilding structures, as described in Chapter 15, are also within the scope and shall be designed and constructed in accordance with the requirements of Chapter 15. Requirements concerning alterations, additions, and change of use are set forth in Appendix 11B. Existing structures and alterations to existing structures need only comply with the seismic requirements of this standard where required by Appendix 11B. The following structures are exempt from the seismic requirements of this standard:

1. Detached one- and two-family dwellings that are located where the mapped, short period, spectral response acceleration parameter, S_S, is less than 0.4 or where the Seismic Design Category determined in accordance with Section 11.6 is A, B, or C.
2. Detached one- and two-family wood-frame dwellings not included in Exemption 1 with not more than two stories above grade plane, satisfying the limitations of and constructed in accordance with the IRC.
3. Agricultural storage structures that are intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed in Chapter 15 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances, and nuclear reactors.
5. Piers and wharves that are not accessible to the general public.

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:

a. Buildings: Chapter 12;
b. Nonbuilding Structures: Chapter 15;
c. Nonstructural Components: Chapter 13;
d. Seismically Isolated Structures: Chapter 17; and
e. Structures with Damping Systems: Chapter 18.

Buildings 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 classified as nonbuilding structures designed and detailed in accordance with Section 15.5 of this standard.

11.1.4 Alternate Materials and Methods of Construction. Alternate materials and methods of construction to those prescribed in the seismic requirements of this standard shall not be used unless approved by the Authority Having Jurisdiction. Substantiating evidence shall be submitted demonstrating that the proposed alternate will be at least equal in strength, durability, and seismic resistance for the purpose intended.

11.1.5 Quality Assurance. Quality assurance for seismic force-resisting systems and other designated seismic systems defined in Section 13.2.2 shall be provided in accordance with the requirements of the Authority Having Jurisdiction.

Where the Authority Having Jurisdiction has not adopted quality assurance requirements, or where the adopted requirements are not applicable to the seismic force-resisting system or designated seismic systems as described in Section 13.2.2, the registered design professional in responsible charge of designing the seismic force-resisting system or other designated seismic systems shall submit a quality assurance plan to the Authority Having Jurisdiction for approval. The quality assurance plan shall specify the quality assurance program elements to be implemented.

11.2 DEFINITIONS

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

ACTIVE FAULT: A fault determined to be active by the Authority Having Jurisdiction from properly substantiated data (e.g., most recent mapping of active faults by the U.S. Geological Survey).

ADDITION: An increase in building area, aggregate floor area, height, or number of stories of a structure.

ALTERATION: Any construction or renovation to an existing structure other than an addition.

APPENDAGE: An architectural component such as a canopy, marquee, ornamental balcony, or statuary.

APPROVAL: The written acceptance by the Authority Having Jurisdiction of documentation that establishes the qualification of a material, system, component, procedure, or person to fulfill the requirements of this standard for the intended use.

ATTACHMENTS: Means by which nonstructural components or supports of nonstructural components are secured or connected to the seismic force-resisting system of the structure. Such attachments include anchor bolts, welded connections, and mechanical fasteners.

BASE: The level at which the horizontal seismic ground motions are considered to be imparted to the structure.

BASE SHEAR: Total design lateral force or shear at the base.

BOUNDARY ELEMENTS: Portions along wall and diaphragm edges for transferring or resisting forces. Boundary elements include chords and collectors at diaphragm and shear wall perimeters, edges of openings, discontinuities, and reentrant corners.

BUILDING: Any structure whose intended use includes shelter

CANTILEVERED COLUMN SYSTEM: A seismic force-resisting system in which lateral forces are resisted entirely by columns acting as cantilevers from the base.

CHARACTERISTIC EARTHQUAKE: An earthquake assessed for an active fault having a magnitude equal to the best estimate of the maximum magnitude capable of occurring on the fault but not less than the largest magnitude that has occurred historically on the fault.

COLLECTOR (DRAG STRUT, TIE, DIAPHRAGM STRUT): A diaphragm or shear wall boundary element parallel to the applied load that collects and transfers diaphragm shear forces to the vertical elements of the seismic force-resisting system or distributes forces within the diaphragm or shear wall.

COMPONENT: A part of an architectural, electrical, or mechanical system.

Component, Flexible: Nonstructural component that has a fundamental period greater than 0.06 s.

Component, Nonstructural: A part of an architectural, mechanical, or electrical system within or without a building or nonbuilding structure.

Component, Rigid: Nonstructural component that has a fundamental period less than or equal to 0.06 s.

Component, Rugged: A nonstructural component that has been shown to consistently function after design earthquake level or greater seismic events based on past earthquake experience data or past seismic testing when adequately anchored or supported. The classification of a nonstructural component as rugged shall be based on a comparison of the specific component with components of similar strength and stiffness. Common examples of rugged components include AC motors, compressors, and base-mounted horizontal pumps.

CONCRETE:

Plain Concrete: Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in ACI 318 for reinforced concrete.

Reinforced Concrete: Concrete reinforced with no less reinforcement than the minimum amount required by ACI 318 prestressed or nonprestressed and designed on the assumption that the two materials act together in resisting forces.

CONSTRUCTION DOCUMENTS: The written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project required to verify compliance with this standard.

COUPLING BEAM: A beam that is used to connect adjacent concrete wall elements to make them act together as a unit to resist lateral loads.

DEFORMABILITY: The ratio of the ultimate deformation to the limit deformation.

High-Deformability Element: An element whose deform-ability is not less than 3.5 where subjected to four fully reversed cycles at the limit deformation.

Limited-Deformability Element: An element that is neither a low-deformability nor a high-deformability element.

Low-Deformability Element: An element whose deformability is 1.5 or less.

DEFORMATION:

Limit Deformation: Two times the initial deformation that occurs at a load equal to 40% of the maximum strength.

Ultimate Deformation: The deformation at which failure occurs and that shall be deemed to occur if the sustainable load reduces to 80% or less of the maximum strength.

Default Site Class Properties: Values of the site coefficients F_a, F_v, and F_{PGA} assigned when available data is insufficient to determine site-specific site class properties.

DESIGN EARTHQUAKE: The earthquake effects that are two-thirds of the corresponding risk-targeted maximum considered earthquake (MCE_R) effects.

DESIGN EARTHQUAKE GROUND MOTION: The earthquake ground motions that are two-thirds of the corresponding MCE_R ground motions.

DESIGNATED SEISMIC SYSTEMS: Those nonstructural components that require design in accordance with Chapter 13 and for which the component Importance Factor, I_p, is greater than 1.0.

DIAPHRAGM: Roof, floor, or other membrane or bracing system acting to transfer the lateral forces to the vertical resisting elements.

Flexure-Controlled Diaphragm: Diaphragm with a flexural yielding mechanism, which limits the maximum forces that develop in the diaphragm, and having a design shear strength or factored nominal shear capacity greater than the shear corresponding to the nominal flexural strength.

Shear-Controlled Diaphragm: Diaphragm that does not meet the requirements of a flexure-controlled diaphragm.

Transfer Forces, Diaphragm: Forces that occur in a diaphragm caused by transfer of seismic forces from the vertical seismic force-resisting elements above the diaphragm to other vertical seismic force-resisting elements below the diaphragm because of offsets in the placement of the vertical elements or changes in relative lateral stiffnesses of the vertical elements.

Vertical Diaphragm: See WALL, Shear Wall.

DIAPHRAGM BOUNDARY: A location where shear is transferred into or out of the diaphragm element. Transfer is either to a boundary element or to another force-resisting element.

DIAPHRAGM CHORD: A diaphragm boundary element perpendicular to the applied load that is assumed to take axial stresses caused by the diaphragm moment.

DISTRIBUTION SYSTEM: An interconnected system of piping, tubing, conduit, raceway, or duct. Distribution systems include in-line components such as valves, in-line suspended pumps, and mixing boxes.

ELEMENT ACTION: Element axial, shear, or flexural behavior.

Critical Action: An action, failure of which would result in the collapse of multiple bays or multiple stories of the building or would result in a significant reduction in the structure's seismic resistance.

Deformation-Controlled Action: Element actions for which reliable inelastic deformation capacity is achievable without critical strength decay.

Force-Controlled Action: Any element actions modeled with linear properties and element actions not classified as deformation-controlled.

Noncritical Actions: An action, failure of which would not result in either collapse or significant loss of the structure's seismic resistance.

Ordinary Action: An action, failure of which would result in only local collapse, comprising not more than one bay in a single story, and would not result in a significant reduction of the structure's seismic resistance.

ENCLOSURE: An interior space surrounded by walls.

EQUIPMENT SUPPORT: Those structural members or assemblies of members or manufactured elements, including braces, frames, legs, lugs, snuggers, hangers, or saddles, that transmit gravity loads and operating loads between the equipment and the structure.

FLEXIBLE CONNECTIONS: Those connections between equipment components that permit rotational and/or translational movement without degradation of performance. Examples include universal joints, bellows expansion joints, and flexible metal hose.

FOUNDATION GEOTECHNICAL CAPACITY: The maximum pressure or strength design capacity of a foundation based upon the supporting soil, rock, or controlled low-strength material.

FOUNDATION STRUCTURAL CAPACITY: The design strength of foundations or foundation components as provided by adopted material standards and as altered by the requirements of this standard.

FRAME:

Braced Frame: An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a building frame system or dual system to resist seismic forces.

Concentrically Braced Frame (CBF): A braced frame in which the members are subjected primarily to axial forces. CBFs are categorized as ordinary concentrically braced frames (OCBFs) or special concentrically braced frames (SCBFs).

Eccentrically Braced Frame (EBF): A diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column or from another diagonal brace.

Moment Frame: A frame in which members and joints resist lateral forces by flexure and along the axis of the members. Moment frames are categorized as intermediate moment frames (IMFs), ordinary moment frames (OMFs), and special moment frames (SMFs).

Structural System:

Building Frame System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.

Dual System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment-resisting frames and shear walls or braced frames as prescribed in Section 12.2.5.1.

Shear Wall–Frame Interactive System: A structural system that uses combinations of ordinary reinforced concrete shear walls and ordinary reinforced concrete moment frames designed to resist lateral forces in proportion to their rigidities considering interaction between shear walls and frames on all levels.

Space Frame System: A 3-D structural system composed of interconnected members, other than bearing walls, that is capable of supporting vertical loads and, where designed for such an application, is capable of providing resistance to seismic forces.

FRICTION CLIP: A device that relies on friction to resist applied loads in one or more directions to anchor a nonstructural

component. Friction is provided mechanically and is not due to gravity loads.

GLAZED CURTAIN WALL: A nonbearing wall that extends beyond the edges of building floor slabs and includes a glazing material installed in the curtain wall framing.

GLAZED STOREFRONT: A nonbearing wall that is installed between floor slabs, typically including entrances, and includes a glazing material installed in the storefront framing.

GRADE PLANE: A horizontal reference plane representing the average of finished ground level adjoining the structure at all exterior walls. Where the finished ground level slopes away from the exterior walls, the grade plane is established by the lowest points within the area between the structure and the property line or, where the property line is more than 6 ft (1,829 mm) from the structure, between the structure and points 6 ft (1,829 mm) from the structure.

HEATING, VENTILATING, AIR-CONDITIONING, AND REFRIGERATION (HVACR): The equipment, distribution systems, and terminals, excluding interconnecting piping and ductwork that provide, either collectively or individually, the processes of heating, ventilating, air-conditioning, or refrigeration to a building or portion of a building.

INSPECTION, SPECIAL: The observation of the work by a special inspector to determine compliance with the approved construction documents and these standards in accordance with the quality assurance plan.

Continuous Special Inspection: The full-time observation of the work by a special inspector who is present in the area where work is being performed.

Periodic Special Inspection: The part-time or intermittent observation of the work by a special inspector who is present in the area where work has been or is being performed.

INSPECTOR, SPECIAL: A person approved by the Authority Having Jurisdiction to perform special inspection, and who shall be identified as the owner's inspector.

INVERTED PENDULUM-TYPE STRUCTURES: Structures in which more than 50% of the structure's mass is concentrated at the top of a slender, cantilevered structure and in which stability of the mass at the top of the structure relies on rotational restraint to the top of the cantilevered element.

JOINT: The geometric volume common to intersecting members.

LIGHT-FRAME CONSTRUCTION: A method of construction where the structural assemblies (e.g., walls, floors, ceilings, and roofs) are primarily formed by a system of repetitive wood or cold-formed steel framing members or subassemblies of these members (e.g., trusses).

LONGITUDINAL REINFORCEMENT RATIO: Area of longitudinal reinforcement divided by the cross-sectional area of the concrete.

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

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 liquefaction, lateral spreading, seismic settlements, and other soil-related issues. 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.

MECHANICALLY ANCHORED TANKS OR VESSELS: Tanks or vessels provided with mechanical anchors to resist overturning moments.

NONBUILDING STRUCTURE: A structure, other than a building, constructed of a type included in Chapter 15 and within the limits of Section 15.1.1.

NONBUILDING STRUCTURE SIMILAR TO A BUILDING: A nonbuilding structure that is designed and constructed in a manner similar to buildings, responds to strong ground motion in a fashion similar to buildings, and has a basic lateral and vertical seismic force-resisting system conforming to one of the types indicated in Tables 12.2-1 or 15.4-1.

OPEN-TOP TANK: A tank without a fixed roof or cover, floating cover, gas holder cover, or dome.

ORTHOGONAL: In two horizontal directions, at 90° to each other.

OWNER: Any person, agent, firm, or corporation that has a legal or equitable interest in a property.

P-DELTA EFFECT: The secondary effect on shears and moments of structural members caused by the action of the vertical loads induced by horizontal displacement of the structure resulting from various loading conditions.

PARTITION: A nonstructural interior wall that spans horizontally or vertically from support to support. The supports may be the basic building frame, subsidiary structural members, or other portions of the partition system.

PILE: Deep foundation element, which includes piers, caissons, and piles.

PILE CAP: Foundation elements to which piles are connected, including grade beams and mats.

PREMANUFACTURED MODULAR MECHANICAL AND ELECTRICAL SYSTEM: A prebuilt, fully or partially enclosed assembly of mechanical and electrical components.

REGISTERED DESIGN PROFESSIONAL: An architect or engineer registered or licensed to practice professional architecture or engineering, as defined by the statutory requirements of the professional registration laws of the state in which the project is to be constructed.

SEISMIC DESIGN CATEGORY: A classification assigned to a structure based on its Risk Category, default site class properties, and the severity of the design earthquake ground motion at the site, as defined in Section 11.4.

SEISMIC FORCE-RESISTING SYSTEM: That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed herein.

SEISMIC FORCES: The assumed forces prescribed herein, related to the response of the structure to earthquake motions, to be used in the design of the structure and its components.

SELF-ANCHORED TANKS OR VESSELS: Tanks or vessels that are stable under design overturning moment without the need for mechanical anchors to resist uplift.

SHEAR PANEL: A floor, roof, or wall element sheathed to act as a shear wall or diaphragm.

SITE CLASS: A classification assigned to a site based on the types of soils present and their engineering properties, as defined in Chapter 20.

STORAGE RACKS, STEEL: A framework or assemblage, comprised of cold-formed or hot-rolled steel structural members, intended for storage of materials, including, but not limited to, pallet storage racks, selective racks, movable-shelf racks, rack-supported systems, automated storage and retrieval systems (stacker racks), push-back racks, pallet-flow racks, case-flow racks, pick modules, and rack-supported platforms. Other types of racks, such as drive-in or drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel, are not considered steel storage racks for the purpose of this standard.

STORAGE RACKS, STEEL CANTILEVERED: A framework or assemblage comprised of cold-formed or hot-rolled steel structural members, primarily in the form of vertical columns, extended bases, horizontal arms projecting from the faces of the columns, and longitudinal (down-aisle) bracing between columns. There may be shelf beams between the arms, depending on the products being stored; this definition does not include other types of racks such as pallet storage racks, drive-in racks, drive-through racks, or racks made of materials other than steel.

STORY: The portion of a structure between the tops of two successive floor surfaces and, for the topmost story, from the top of the floor surface to the top of the roof surface.

STORY ABOVE GRADE PLANE: A story in which the floor or roof surface at the top of the story is more than 6 ft (1,828 mm) above grade plane or is more than 12 ft (3,658 mm) above the finished ground level at any point on the perimeter of the structure.

STORY DRIFT: The horizontal deflection at the top of the story relative to the bottom of the story as determined in Section 12.8.6.

STORY DRIFT RATIO: The story drift, as determined in Section 12.8.6, divided by the story height, h_{sx}.

STORY SHEAR: The summation of design lateral seismic forces at levels above the story under consideration.

STRENGTH:

Design Strength: Nominal strength multiplied by a strength reduction factor, ϕ.

Nominal Strength: Strength of a member or cross section calculated in accordance with the requirements and assumptions of the strength design methods of this standard (or the reference documents) before application of any strength-reduction factors.

Required Strength: Strength of a member, cross section, or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by this standard.

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.

STRUCTURAL OBSERVATIONS: The visual observations to determine that the seismic force-resisting system is constructed in general conformance with the construction documents.

STRUCTURE: That which is built or constructed and limited to buildings and nonbuilding structures as defined herein.

SUBDIAPHRAGM: A portion of a diaphragm used to transfer wall anchorage forces to diaphragm crossties.

SUPPORTS: Those members, assemblies of members, or manufactured elements, including braces, frames, legs, lugs, snubbers, hangers, saddles, or struts, and associated fasteners that transmit loads between nonstructural components and their attachments to the structure.

TESTING AGENCY: A company or corporation that provides testing and/or inspection services.

VENEERS: Facings or ornamentation of brick, concrete, stone, tile, or similar materials attached to a backing.

WALL: A component that has a slope of 60 deg or greater with the horizontal plane used to enclose or divide space.

Bearing Wall: Any wall meeting either of the following classifications:

1. Any metal or wood stud wall that supports more than 100 lb/linear ft (1,459 N/m) of vertical load in addition to its own weight.
2. Any concrete or masonry wall that supports more than 200 lb/linear ft (2,919 N/m) of vertical load in addition to its own weight.

Light Frame Wall: A wall with wood or steel studs.

Light Frame Wood Shear Wall: A wall constructed with wood studs and sheathed with material rated for shear resistance. **Nonbearing Wall:** Any wall that is not a bearing wall.

Nonstructural Wall: A wall other than a bearing wall or shear wall.

Shear Wall (Vertical Diaphragm): A wall, bearing or nonbearing, designed to resist lateral forces acting in the plane of the wall (sometimes referred to as a "vertical diaphragm").

Structural Wall: A wall that meets the definition for bearing wall or shear wall.

WALL SYSTEM, BEARING: A structural system with bearing walls providing support for all or major portions of the vertical loads. Shear walls or braced frames provide seismic force resistance.

WOOD STRUCTURAL PANEL: A wood-based panel product that meets the requirements of DOC PS1 or DOC PS2 and is bonded with a waterproof adhesive. Included under this designation are plywood, oriented strand board, and composite panels.

11.3 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 seismic provisions of Chapters 11 through 22 in this standard.

A_0 = area of the load-carrying foundation [ft^2 (m^2)]

A_{ch} = cross-sectional area [in.^{2} (mm^2)] of a structural member measured out-to-out of transverse reinforcement

A_{sh} = total cross-sectional area of hoop reinforcement [in.^{2} (mm^2)], including supplementary crossties, having a spacing of s_h and crossing a section with a core dimension of h_c

A_{vd} = required area of leg [in.^{2} (mm^2)] of diagonal reinforcement

A_x = torsional amplification factor (Section 12.8.4.3)

a_i = the acceleration at level i obtained from a modal analysis (Section 13.3.1) a

a_p = the amplification factor related to the response of a system or component as affected by the type of seismic attachment, determined in Section 13.3.1 b_p= the width of the rectangular glass panel

C_d = deflection amplification factor as given in Tables 12.2-1, 15.4-1, or 15.4-2

C_{dX} = deflection amplification factor in the X direction (Section 12.9.2.5)

C_{dY} = deflection amplification factor in the Y direction (Section 12.9.2.5)

C_{p0} = diaphragm design acceleration coefficient at the structure base (Section 12.10.3.2.1)

C_{pi} = diaphragm design acceleration coefficient at 80% of the structural height above the base, h_n (Section 12.10.3.2.1)

C_{pn} = diaphragm design acceleration coefficient at the structural height, h_n (Section 12.10.3.2.1)

C_{px} = diaphragm design acceleration coefficient at level x (Section 12.10.3.2.1)

C_R = site-specific risk coefficient at any period (Section 21.2.1.1)

C_{R1} = mapped value of the risk coefficient at a period of 1 s as given by Fig. 22-19

C_{RS} = mapped value of the risk coefficient at short periods as given by Fig. 22-18

C_s = seismic response coefficient determined in Section 12.8.1.1 or 19.3.1 (dimensionless)

C_{s2} = higher mode seismic response coefficient (Section 12.10.3.2.1)

C_t = building period coefficient (Section 12.8.2.1)

C_{vx} = vertical distribution factor as determined (Section 12.8.3)

c = distance from the neutral axis of a flexural member to the fiber of maximum compressive strain [in. (mm)]

D = the effect of dead load

D_{clear} = relative horizontal (drift) displacement, measured over the height of the glass panel under consideration, which causes initial glass-to-frame contact. For rectangular glass panels within a rectangular wall frame, D_{clear} is set forth in Section 13.5.9.1

D_{pI} = seismic relative displacement; see Section 13.3.2

D_s = the total depth of stratum in Eq. (19.3-4) [ft (m)]

d_c = the total thickness of cohesive soil layers in the top 100 ft (30 m); see Section 20.4.3 [ft (m)]

d_i = the thickness of any soil or rock layer i [between 0 and 100 ft (between 0 and 30 m)]; see Section 20.4.1 [ft (m)]

d_S = the total thickness of cohesionless soil layers in the top 100 ft (30 m); see Section 20.4.2 [ft (m)]

E = effect of horizontal and vertical earthquake-induced forces (Section 12.4)

E_{cl} = The capacity-limited horizontal seismic load effect, equal to the maximum force that can develop in the element as determined by a rational, plastic mechanism analysis

F_a = short-period site coefficient (at 0.2-s period); see Section 11.4.4

F_i, F_n, F_x= portion of the seismic base shear, V, induced at level i, n, or x, respectively, as determined in Section 12.8.3

F_p = the seismic force acting on a component of a structure as determined in Sections 12.11.1 and 13.3.1

F_{px}= diaphragm seismic design force at Level x

F_{PGA} = site coefficient for peak ground acceleration (PGA); see Section 11.8.3

F_v = long-period site coefficient (at 1.0-s period); see Section 11.4.4

f_c' = specified compressive strength of concrete used in design

f_s' = ultimate tensile strength [psi (MPa)] of the bolt, stud, or insert leg wires. For ASTM A307 bolts or ASTM A108 studs, it is permitted to be assumed to be 60,000 psi (415 MPa)

f_y = specified yield strength of reinforcement [psi (MPa)]

f_{yh} = specified yield strength of the special lateral reinforcement [psi (kPa)]

$G = \gamma v_s^2/g$ = the average shear modulus for the soils beneath the foundation at large strain levels [psf (Pa)]

$G_0 = \gamma v_{s0}^2/g$ = the average shear modulus for the soils beneath the foundation at small strain levels [psf (Pa)]

g = acceleration due to gravity

H = thickness of soil

h = height of a shear wall measured as the maximum clear height from top of foundation to bottom of diaphragm framing above, or the maximum clear height from top of diaphragm to bottom of diaphragm framing above

h = average roof height of structure with respect to the base; see Chapter 13

h^* = effective height of the building as determined in Chapter 19 [ft (m)]

h_c = core dimension of a component measured to the outside of the special lateral reinforcement [in. (mm)]

h_i, h_x = the height above the base to level i or x, respectively

h_n = structural height as defined in Section 11.2

h_p = the height of the rectangular glass panel

h_{sx} = the story height below level $x = (h_x - h_{x-1})$

I_e = the Importance Factor as prescribed in Section 11.5.1

I_p = the component importance factor as prescribed in Section 13.3.1

i = the building level referred to by the subscript i; i =1 designates the first level above the base

K_p = the stiffness of the component or attachment (Section 13.3.3)

K_{xx}, K_{rr} = rotational foundation stiffness [Eqs. (19.3-9) and (19.3-19) [ft-lb/degree (N-m/rad)]

K_y, K_r = translational foundational stiffness [Eqs. (19.3-8) and (19.3-18)] [lb/in. (N/m)]

KL/r = the lateral slenderness ratio of a compression member measured in terms of its effective length, KL, and the least radius of gyration of the member cross section, r

k = distribution exponent given in Section 12.8.3

k_a = coefficient defined in Sections 12.11.2.1 and 12.14.7.5

L = overall length of the building (ft or m) at the base in the direction being analyzed

M_t = torsional moment resulting from eccentricity between the locations of center of mass and the center of rigidity (Section 12.8.4.1)

M_{ta} = accidental torsional moment as determined in Section 12.8.4.2

m = a subscript denoting the mode of vibration under consideration; that is, m =1 for the fundamental mode

N = standard penetration resistance, ASTM D1586

N = number of stories above the base (Section 12.8.2.1)

$\overline{N}$ = average field standard penetration resistance for the top 100 ft (30 m); see Sections 20.3.3 and 20.4.2

$\overline{N}_{ch}$ = average standard penetration resistance for cohesionless soil layers for the top 100 ft (30 m); see Sections 20.3.3 and 20.4.2

N_i = standard penetration resistance of any soil or rock layer i [between 0 and 100 ft (between 0 and 30 m)]; see Section 20.4.2

n = designation for the level that is uppermost in the main portion of the building

PGA = mapped MCEG peak ground acceleration shown in Figs. 22-9 through 22-13

PGA_M = MCEG peak ground acceleration adjusted for site class effects; see Section 11.8.3

PI = plasticity index, ASTM D4318

P_x= total unfactored vertical design load at and above level x, for use in Section 12.8.7

Q_E = effect of horizontal seismic (earthquake-induced) forces

R = response modification coefficient as given in Tables 12.2-1, 12.14-1, 15.4-1, and 15.4-2

R_p = component response modification factor as defined in Section 13.3.1

R_s = diaphragm design force reduction factor (Section 12.10.3.5)

R_X = response modification coefficient in the X direction (Section 12.9.2.5)

R_Y = response modification coefficient in the Y direction (Section 12.9.2.5)

S_1 = mapped MCE_R, 5% damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.2

S_{aM} = the site-specific MCE_R spectral response acceleration parameter at any period

S_{D1} = design, 5% damped, spectral response acceleration parameter at a period of 1 s as defined in Section 11.4.5

S_{DS} = design, 5% damped, spectral response acceleration parameter at short periods as defined in Section 11.4.5

S_{M1} = the MCE_R, 5% damped, spectral response acceleration parameter at a period of 1 s adjusted for site class effects as defined in Section 11.4.4

S_{MS} = the MCE_R, 5% damped, spectral response acceleration parameter at short periods adjusted for site class effects as defined in Section 11.4.4.

S_S = mapped MCE_R, 5% damped, spectral response acceleration parameter at short periods as defined in Sections 11.4.2, 11.4.4

s_h = spacing of special lateral reinforcement [in. (mm)]

s_u = undrained shear strength; see Section 20.4.3

$\overline{s}_u$ = average undrained shear strength in top 100 ft (30 m); see Sections 20.3.3 and 20.4.3, ASTM D2166,or ASTM D2850

s_{ui} = undrained shear strength of any cohesive soil layer i [between 0 and 100 ft (0 and 30 m)]; see Section 20.4.3

T = the fundamental period of the building

$T_0 = 0.2S_{D1}/S_{DS}$

$\tilde{T}$= the fundamental period as determined in Chapter 19 T_a = approximate fundamental period of the building as determined in Section 12.8.2

T_L = long-period transition period as defined in Section 11.4.6

T_{lower} = period of vibration at which 90% of the actual mass has been recovered in each of the two orthogonal directions of response (Section 12.9.2). The mathematical model used to compute T_{lower} shall not include accidental torsion and shall include P-delta effects.

T_p= fundamental period of the component and its attachment (Section 13.3.3)

$T_S = S_{D1}/S_{DS}$

T_{upper} = the larger of the two orthogonal fundamental periods of vibration (Section 12.9.2). The mathematical model used to compute T_{upper} shall not include accidental torsion and shall include P-delta effects

V = total design lateral force or shear at the base

V_{EX} = maximum absolute value of elastic base shear computed in the X direction among all three analyses performed in that direction (Section 12.9.2.5)

V_{EY} = maximum absolute value of elastic base shear computed in the Y direction among all three analyses performed in that direction (Section 12.9.2.5)

V_{IX} = inelastic base shear in the X direction (Section 12.9.2.5)

V_{IY} = inelastic base shear in the Y direction (Section 12.9.2.5)

V_t = design value of the seismic base shear as determined in Section 12.9.1.4.1

V_X = ELF base shear in the X direction (Section 12.9.2.5)

V_x = seismic design shear in story x as determined in Section 12.8.4

V_Y = ELF base shear in the Y direction (Section 12.9.2.5)

$\tilde{V}$= reduced base shear accounting for the effects of soil structure interaction as determined in Section 19.3.1

$\tilde{V}_1$ = portion of the reduced base shear, $\tilde{V}_1$ contributed by the fundamental mode, Section 19.3, in kip (kN)

ΔV = reduction in V as determined in Section 19.3.1, in kip (kN)

ΔV_1 = reduction in V_1 as determined in Section 19.3.1, in kip (kN)

v_s = shear wave velocity at small shear strains (greater than 10^{-3}% strain); see Section 19.2.1, in ft/s (m/s)

$\tilde{v}_s$ = average shear wave velocity at small shear strains in top 100 ft (30 m); see Sections 20.3.3 and 20.4.1

v_{si} = the shear wave velocity of any soil or rock layer i (between 0 and 100 ft (between 0 and 30 m)); see Section 20.4.1

v_{so} = average shear wave velocity for the soils beneath the foundation at small strain levels, Section 19.2.1.1 in ft/s (m/s)

W = effective seismic weight of the building as defined in Section 12.7.2. For calculation of seismic-isolated building period, W is the total effective seismic weight of the building as defined in Sections 19.2 and 19.3, in kip (kN)

W = effective seismic weight of the building as defined in Sections 19.2 and 19.3, in kip (kN)

W_c = gravity load of a component of the building

W_P = component operating weight, in lb (N)

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

w = moisture content (in percent), ASTM D2216

w_i,w_n,w_x = portion of W that is located at or assigned to level i, n, or x, respectively

x = level under consideration, 1 designates the first level above the base

z = height in structure of point of attachment of component with respect to the base; see Section 13.3.1

z_s = mode shape factor, Section 12.10.3.2.1

β = ratio of shear demand to shear capacity for the story between levels x and $x - 1$

$\overline{\beta}$ = fraction of critical damping for the coupled structure– foundation system, determined in Section 19.2.1

β_0 = foundation damping factor as specified in Section 19.2.1.2

Γ_{m1},Γ_{m2} = first and higher modal contribution factors, respectively, Section 12.10.3.2.1

γ = average unit weight of soil, in lb/ft^3 (N/m^3)

Δ = design story drift as determined in Section 12.8.6

Δ_{fallout} = the relative seismic displacement (drift) at which glass fallout from the curtain wall, storefront, or partition occurs

Δ_a = allowable story drift as specified in Section 12.12.1

Δ_{ADVE} = average drift of adjoining vertical elements of the seismic force-resisting system over the story below the diaphragm under consideration, under tributary lateral load equivalent to that used in the computation of δ_{MDD} Fig. 12.3-1, in in. (mm)

δ_{MDD} = computed maximum in-plane deflection of the diaphragm under lateral load, Fig. 12.3-1, in in. (mm)

δ_{max} = maximum displacement at level x, considering torsion, Section 12.8.4.3

δ_M = maximum inelastic response displacement, considering torsion, Section 12.12.3

δ_{MT} = total separation distance between adjacent structures on the same property, Section 12.12.3

δ_{avg} = the average of the displacements at the extreme points of the structure at level x, Section 12.8.4.3

δ_x = deflection of level x at the center of the mass at and above level x, Eq. (12.8-15)

δ_{xc} = deflection of level x at the center of the mass at and above level x determined by an elastic analysis, Section 12.8.6

δ_{xm} = modal deflection of level x at the center of the mass at and above level x as determined by Section 19.3.2

$\overline{\delta}_x$, $\overline{\delta}_{x1}$ = deflection of level x at the center of the mass at and above level x, Eqs. (19.2-13) and (19.3-3), in in. (mm) θ = stability coefficient for P-delta effects as determined in Section 12.8.7

ηx = Force scale factor in the X direction (12.9.2.5)

ηy = Force scale factor in the Y direction (12.9.2.5)

ρ = a redundancy factor based on the extent of structural redundancy present in a building as defined in Section 12.3.4

User Note: Electronic values of mapped acceleration parameters and other seismic design parameters are provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76.

ρ_s = spiral reinforcement ratio for precast, prestressed piles in Section 14.2.3.2.6

λ = time effect factor

Ω_0 = overstrength factor as defined in Tables 12.2-1, 15.4.-1, and 15.4-2

Ω_v = Diaphragm shear overstrength factor (Section 14.2.4.1.3)

11.4 SEISMIC GROUND MOTION VALUES

11.4.1 Near-Fault Sites. Sites satisfying either of the following conditions shall be classified as near fault:

1. 9.5 miles (15 km) of the surface projection of a known active fault capable of producing M_w7 or larger events, or
2. 6.25 miles (10 km) of the surface projection of a known active fault capable of producing M_w6 or larger events.

EXCEPTIONS:

1. Faults with estimated slip rate along the fault less than 0.04 in. (1 mm) per year shall not be considered.
2. The surface projection shall not include portions of the fault at depths of 6.25 mi (10 km) or greater.

11.4.2 Mapped Acceleration Parameters. The parameters Ss and S1 shall be determined from the 0.2- and 1-s spectral response accelerations shown in Figs. 22-1, 22-3, 22-5, 22-6, 22-7, and 22-8 for Ss and Figs. 22-2, 22-4, 22-5, 22-6, 22-7, and 22-8 for S1. ~~Where S_1 is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 11.7.~~

11.4.3 Site Class. Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with Chapter 20. For situations in which site investigations, performed in accordance with Chapter 20 reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, the site coefficients F_a, F_v and F_{PGA} shall be taken as unity (1.0)

11.4.4 Default Site Class Properties. Where the soil properties are not known in sufficient detail to determine the site class and the authority having jurisdiction or geotechnical data determine that Site Class E or F soils are not present, it shall be permitted to use the site coefficients F_a, F_v and F_{PGA} associated with Site Class D except that the value of F_a shall not be taken less than 1.2.~~, subject to the requirements of Section 11.4.4, shall be used unless the authority having jurisdiction or geotechnical data determine that Site Class E or F soils are present at the site.~~

~~For situations in which site investigations, performed in accordance with Chapter 20, reveal rock conditions consistent with Site Class B, but site specific velocity measurements are not made, the site coefficients F_a, F_v, and F_{PGA} shall be taken as unity (1.0).~~

11.4.4~~5~~ Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters. The MCE_R spectral response acceleration parameters for short periods (SMS) and at 1 s (SM 1), adjusted for site class effects, shall be determined by Eqs. (11.4-1) and (11.4-2), respectively.

$$S_{MS} = F_a S_S \quad (11.4\text{-}1)$$

$$S_{M1} = F_v S_1 \quad (11.4\text{-}2)$$

where

S_S = the mapped MCE_R spectral response acceleration parameter at short periods as determined in accordance with Section 11.4.2, and

S_1 = the mapped MCE_R spectral response acceleration parameter at a period of 1 s as determined in accordance with Section 11.4.2

where site coefficients F_a and F_v are defined in Tables 11.4-1 and 11.4-2, respectively. ~~Where Site Class D is selected as the default site class per Section 11.4.3, the value of F_a shall not be less than 1.2.~~ Where the simplified design procedure of Section 12.14 is used, the value of F_a shall be determined in accordance with Section 12.14.8.1, and the values for F_v, $_{SMS}$, and $_{SM1}$ need not be determined.

Table 11.4-1 Short-Period Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_s.

Table 11.4-2 Long-Period Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.
[a]Also, see requirements for site-specific ground motions in Section 11.4.8.

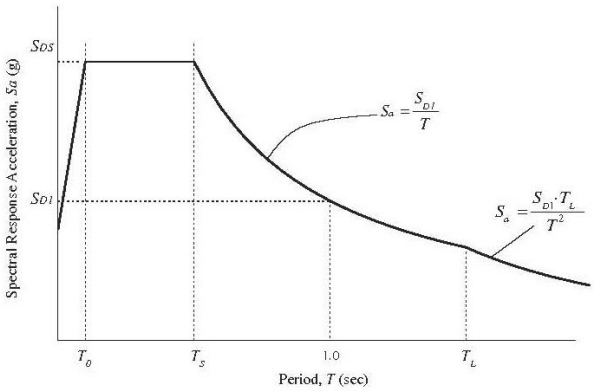

FIGURE 11.4-1 Design Response Spectrum

11.4.~~5~~ 6 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 Eqs. (11.4-3) and (11.4-4), respectively. Where the alternate simplified 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.

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

$$S_{D1} = \frac{2}{3} S_{M1} \quad (11.4\text{-}4)$$

11.4.~~6~~ 7 Design Response Spectrum. Where a design response spectrum is required by this standard and site-specific ground motion procedures are not used, the design response spectrum curve shall be developed as indicated in Fig. 11.4-1 and as follows:

1. For periods less than T_0, the design spectral response acceleration, S_a, shall be taken as given in Eq. (11.4-5):

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T0}\right) \quad (11.4\text{-}5)$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken as equal to S_{DS}.

3. For periods greater than T_S and less than or equal to T_L, the design spectral response acceleration, S_a, shall be taken as given in Eq. (11.4-6):

$$S_a = \frac{SD1}{T} \quad (11.4\text{-}6)$$

4. For periods greater than T_L, S_a shall be taken as given in Eq. (11.4-7):

$$S_a = \frac{SD1 * TL}{T\text{^}2} \quad (11.4\text{-}7)$$

where

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

S_{D1} = the design spectral response acceleration parameter at a 1-s period

T = the fundamental period of the structure, s

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

$T_S = S_{D1}/S_{DS}$, and

T_L = long-period transition period(s) shown in Figs. 22-14 through 22-17.

11.4.~~7~~ 8 Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum. Where an MCE_R response spectrum is required, it shall be determined by multiplying the design response spectrum by 1.5.

11.4.~~8~~ 9 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. A ground motion hazard analysis shall be performed in accordance with Section 21.2 for the following:

1. seismically isolated structures and structures with damping systems on sites with S_1 greater than or equal to 0.6,
2. structures on Site Class E sites with S_s greater than or equal to 1.0, and.
3. structures on Site Class D and E sites with S_1 greater than or equal to 0.2.

EXCEPTION: A ground motion hazard analysis is not required for structures other than seismically isolated structures and structures with damping systems where:

1. Structures on Site Class E sites with S_S greater than or equal to 1.0, provided the site coefficient F_a is taken as equal to that of Site Class C.
2. Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T < 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L > T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$.
3. Structures on Site Class E sites with S_1 greater than or equal to 0.2, provided that T is less than or equal to T_s and the equivalent static force procedure is used for design.

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_G peak ground acceleration parameter shall be determined in accordance with Section 21.5.

11.5 IMPORTANCE FACTOR AND RISK CATEGORY

11.5.1 Importance Factor. An Importance Factor, I_e, shall be assigned to each structure in accordance with Table 1.5-2.

11.5.2 Protected Access for Risk Category IV. Where operational access to a Risk Category IV structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for Risk Category IV structures. Where operational access is less than 10 ft (3.048 m) from an interior lot line or another structure on the same lot, protection from potential falling debris from adjacent structures shall be provided by the owner of the Risk Category IV structure.

11.6 SEISMIC DESIGN CATEGORY

Structures shall be assigned a Seismic Design Category ~~in accordance with this section~~as determined from Figures 11.6.1 or 11.6.2.

Add Figure 11.6.1: SDC Map for Risk Category I/II/III Structures

Add Figure 11.6.2: SDC Map for Risk Category IV Structures

~~Risk Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Risk Category IV structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a Seismic Design Category based on their Risk Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in accordance with Section 11.4.5. Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 11.6-1 or 11.6-2, irrespective~~

TABLE 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

of the fundamental period of vibration of the structure, T. The provisions in Chapter 19 shall not be used to modify the spectral response acceleration parameters for determining Seismic Design Category.

Where S_1 is less than 0.75, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone where all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, determined in accordance with Section 12.8.2.1 is less than $0.8T_s$ where T_s is determined in accordance with Section 11.4.6.
2. In each of two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Eq. (12.8-2) is used to determine the seismic response coefficient C_s.
4. The diaphragms are rigid in accordance with Section 12.3; or, for diaphragms that are not rigid, the horizontal distance between vertical elements of the seismic force-resisting system does not exceed 40 ft (12.192 m).

Where the alternate simplified design procedure of Section 12.14 is used, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone, using the value of S_{DS} determined in Section 12.14.8.1, except that where S_1 is greater than or equal to 0.75, the Seismic Design Category shall be E.

11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

Buildings and other structures assigned to Seismic Design Category A need only comply with the requirements of Section 1.4. Nonstructural components in SDC A are exempt from seismic design requirements. In addition, tanks assigned to Risk Category IV shall satisfy the freeboard requirement in Section 15.6.5.1.

11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

11.8.1 Site Limitation for Seismic Design Categories E and F. A structure assigned to Seismic Design Category E or F shall not be located where a known potential exists for an active fault to cause rupture of the ground surface at the structure.

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 faulting or 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.

EXCEPTION: Where approved by the authority having jurisdiction, a site-specific geotechnical report is not required where prior evaluations of nearby sites with similar soil conditions provide direction relative to the proposed construction.

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 all of the following, as applicable:

1. The determination of dynamic seismic lateral earth pressures on basement and retaining walls caused by design earthquake ground motions.
2. The potential for liquefaction and soil strength loss 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 11.4.8 or (2) the peak ground acceleration PGA_M, from Eq. (11.8-1).

$$PGA_M = F_{PGA} \cdot PGA \tag{11.8-1}$$

where

PGA_M = MCE_G peak ground acceleration adjusted for site class effects.

PGA = Mapped MCE_G peak ground acceleration shown in Figs. 22-9 through 22-13.

F_{PGA} = Site coefficient from Table 11.8-1.

where Site Class D is selected as the default site class per Section 11.4.3, the value of F_{PGA} shall not be less than 1.2.

3. Assessment of potential consequences of liquefaction 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 down-drag and reduction in axial and lateral soil reaction for pile foundations, increases in soil lateral pressures on retaining walls, and flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, 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.

TABLE 11.8-1 Site Coefficient F_{PGA}

	Mapped Maximum Considered Geometric Mean (MCE_G) Peak Ground Acceleration, PGA					
Site Class	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA = 0.5	PGA ≥ 0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.2	1.2	1.2	1.2	1.2
D	1.6	1.4	1.3	1.2	1.1	1.1
E	2.4	1.9	1.6	1.4	1.2	1.1
F	See Section 11.4.8					

Note: Use straight-line interpolation for intermediate values of PGA.

11.9 VERTICAL GROUND MOTIONS FOR SEISMIC DESIGN

11.9.1 General. If the option to incorporate the effects of vertical seismic ground motions is exercised in lieu of the requirements of Section 12.4.2.2, the requirements of this section are permitted to be used in the determination of the vertical design earthquake ground motions. The requirements of Section 11.9 shall only apply to structures in Seismic Design Categories C, D, E, and F.

11.9.2 MCE_R Vertical Response Spectrum. Where a vertical response spectrum is required by this standard and site-specific procedures are not used, the MCE_R vertical response spectral acceleration, S_{aMv}, shall be developed as follows:

1. For vertical periods less than or equal to 0.025 s, S_{aMv} shall be determined in accordance with Eq. (11.9-1) as follows:

$$S_{aMv} = 0.3C_vS_{MS} \quad (11.9\text{-}1)$$

2. For vertical periods greater than 0.025 s and less than or equal to 0.05 s, S_{aMv} shall be determined in accordance with Eq. (11.9-2) as follows:

$$S_{aMv} = 20C_vS_{MS}(T_v\text{-}0.025)+0.3C_vS_{MS} \quad (11.9\text{-}2)$$

3. For vertical periods greater than 0.05 s and less than or equal to 0.15 s, S_{aMv} shall be determined in accordance with Eq. (11.9-3) as follows:

$$S_{aMv} = 0.8C_vS_{MS} \quad (11.9\text{-}3)$$

4. For vertical periods greater than 0.15 s and less than or equal to 2.0 s, S_{aMv} shall be determined in accordance with Eq. (11.9-4) as follows:

$$S_{aMv} = 0.3C_vS_{MS}\left(\frac{0.15}{Tv}\right)^{0.75} \quad (11.9\text{-}4)$$

where

C_v = is defined in terms of S_S in Table 11.9-1,

S_{MS} = the MCE_R spectral response acceleration parameter at short periods, and

T_v = the vertical period of vibration.

TABLE 11.9-1 Values of Vertical Coefficient C_v

Mapped MCE_R Spectral Response Parameter at Short Periods[a]	Site Class A, B	Site Class C	Site Class D, E, F
$S_S \geq 2.0$	0.9	1.3	1.5
$S_S = 1.0$	0.9	1.1	1.3
$S_S = 0.6$	0.9	1.0	1.1
$S_S = 0.3$	0.8	0.8	0.9
$S_S \leq 0.2$	0.7	0.7	0.7

[a]Use straight-line interpolation for intermediate values of S_S.

S_{aMv} shall not be less than one-half of the corresponding S_{aM} for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.

For vertical periods greater than 2.0 s, S_{aMv} shall be developed from a site-specific procedure; however, the resulting ordinate of S_{aMv} shall not be less than one-half of the corresponding S_a for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.

In lieu of using the above procedure, a site-specific study is permitted to be performed to obtain S_{aMv} at vertical periods less than or equal to 2.0 s, but the value so determined shall not be less than 80% of the S_{aMv} value determined from Eqs. (11.9-1) through (11.9-4).

11.9.3 Design Vertical Response Spectrum. The design vertical response spectral acceleration, $S_{av,}$ shall be taken as two-thirds of the value of S_{aMv} determined in Section 11.9.2.

11.10 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

CHAPTER 12

SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

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

12.14.1 General

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 from Figures 11.6-1 and 11.6-2. ~~Table 11.6-1 using the value of S_{DS} from Section 12.14.8.1, except that where $S1$ is greater than or equal to 0.75, the Seismic Design Category shall be E.~~ The simplified design procedure is permitted to be used if the following limitations are met:

1. The structure shall qualify for Risk Category I or II in accordance with Table 1.5-1.
2. The site class, defined in Chapter 20, shall not be Site Class E or F.
3. The structure shall not exceed three stories above grade plane.
4. The seismic force-resisting system shall be either a bearing wall system or a building frame system, as indicated in Table 12.14-1.
5. The structure shall have at least two lines of lateral resistance in each of two major axis directions. At least one line of resistance shall be provided on each side of the center of weight in each direction.
6. The center of weight in each story shall be located not further from the geometric centroid of the diaphragm than 10% of the length of the diaphragm parallel to the eccentricity.
7. For structures with cast-in-place concrete diaphragms, overhangs beyond the outside line of shear walls or braced frames shall satisfy the following:

$$a \le d/3 \qquad (12.14\text{-}1)$$

where

a = the distance perpendicular to the forces being considered from the extreme edge of the diaphragm to the line

of vertical resistance closest to that edge, and

d = the depth of the diaphragm parallel to the forces being considered at the line of vertical resistance closest to the edge.

All other diaphragm overhangs beyond the outside line of shear walls or braced frames shall satisfy the following:

$$a \le d/5 \qquad (12.14\text{-}2)$$

8. For buildings with a diaphragm that is not flexible, the forces shall be apportioned to the vertical elements as if the diaphragm were flexible. The following additional requirements shall be satisfied:

a. For structures with two lines of resistance in a given direction, the distance between the two lines is at least 50% of the length of the diaphragm perpendicular to the lines;
b. For structures with more than two lines of resistance in a given direction, the distance between the two most extreme lines of resistance in that direction is at least 60% of the length of the diaphragm perpendicular to the lines;

Where two or more lines of resistance are closer together than one-half the horizontal length of the longer of the walls or braced frames, it shall be permitted to replace those lines by a single line at the centroid of the group for the initial distribution of forces, and the resultant force to the group shall then be distributed to the members of the group based on their relative stiffnesses.

9. Lines of resistance of the seismic force-resisting system shall be oriented at angles of no more than 15 deg from alignment with the major orthogonal horizontal axes of the building.

10. The simplified design procedure shall be used for each major orthogonal horizontal axis direction of the building.

11. System irregularities caused by in-plane or out-of-plane offsets of lateral force-resisting elements shall not be permitted.

EXCEPTION: Out-of-plane and in-plane offsets of shear walls are permitted in two-story buildings of light-frame construction provided that the framing supporting the upper wall is designed for seismic force effects from overturning of the wall amplified by a factor of 2.5.

12. The lateral load resistance of any story shall not be less than 80% of the story above.

CHAPTER C11

SEISMIC DESIGN CRITERIA

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 U.S. building. However, SDCs also depend on a building's occupancy and, therefore, its desired performance. ~~Furthermore, unlike the traditional implementation of seismic zones, the ground motions used to define the SDCs include the effects of individual site conditions on probable ground-shaking intensity.~~

In developing the ground-motion 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 current design philosophy was adopted from the 1997 NEHRP provisions and Commentary (FEMA 1997a and FEMA 1997b), 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 SDC E

An important change was made to the determination of SDC when the current design philosophy was adopted. 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 *SDS* and $_{SD1,}$ 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 one category higher for Risk Category IV structures than for lower risk structures. This rating has the effect of increasing the confidence that the design and construction requirements can deliver the intended performance in the extreme event.

Note that the tables in the standard 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. Note that this boundary is defined in terms of mapped bedrock outcrop motions affecting response at 1 s, not site-adjusted values, to better discriminate between sites near and far from faults. Short-period response is not normally as affected as the longer period response. 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.

Since their introduction into the Provisions, in the 1997 edition, SDC has been determined on a site-specific 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. 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 to the next. The 2021 edition of the Provisions specified determination of SDC based on a default site class condition. This was done to provide more uniform and consistent seismic design and construction practice throughout regions and also to minimize the fluctuation of SDC assignments resulting from minor changes in design values obtained from the NSHM maps.

~~For most buildings,~~ ~~t~~The SDC is determined without consideration of the building's period. Structures are assigned to an SDC based on the more severe condition determined from 1-s acceleration and short-period acceleration. This assigning is 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 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 misclassify-ing a building's SDC by inaccurately estimating the fundamental period, the standard generally requires that the more severe SDC determined on the basis of short- and long-period shaking be used.

Another reason for this requirement is a desire to simplify building regulation by requiring all buildings on a given soil profile in a particular region to be assigned to the same SDC, regardless of the structural type. This assignment has the advantage of permitting uniform regulation in the selection of seismic force-resisting systems, inspection and testing requirements, seismic design requirements for nonstructural components, and similar aspects of the design process regulated on the basis of SDC, within a community.

~~Notwithstanding the above, it is recognized that classification of a building as SDC C instead of B or D can have a significant impact on the cost of construction. Therefore, the standard includes an exception permitting the classification of buildings that can reliably be classified as having short structural periods on the basis of short-period shaking alone.~~

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, ~~particular site classes,~~ or particular Seismic Design Categories for all or part of the area of their jurisdiction. For example,

1. An area with a historical practice of high seismic zone detailing might mandate a minimum SDC of D regardless of ground motion or site class.

2. 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.~~
3. ~~An area with unusual soils might require use of a particular site class unless a geotechnical investigation proves a better site class.~~

Seismic Design Category (SDC) maps were developed in accordance with the following rules:

1. Where S_1 is less than or equal to 0.04 and *Ss* is less than or equal to 0.15, Seismic Design Category A is designated.
2. Where the mapped spectral response acceleration parameter at 1-s period, S1, is greater than or equal to 0.75 Seismic Design Category E is designated for Risk Category I, II and III structures, and Seismic Design Category F is designated for Risk Category IV structures.
3. In all other regions Seismic Design Category is assigned based on Risk Category and the design spectral response acceleration parameters, S_{DS} and SD1, determined assuming default site class conditions. In each location, SDC is assigned as the more severe condition determined in accordance with Table C11.6-1 or C11.6-2.

[Add Tables C11.6-1 and C11.6-2]

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

As previously described, the reason the 2021 Provisions specify determination of SDC based on default site class, rather than actual site class conditions is to promote uniformity of design and construction practices in regions, and also to minimize fluctuation in required SDC within a region as a result of relatively modest changes in mapped values of seismic hazard from one edition of the national seismic hazard maps to the another edition. One model for maintaining SDC stability when later editions of the national seismic maps become available is as follows:

1. New SDC maps should be developed based on the updated national seismic hazard maps and ASCE 7 criteria. Previously published and new SDC maps should be compared to identify areas of significant change.
2. Where SDCs change between previously published and new map versions:
    a. Variations ≥ 10% of the controlling (S_{ds} or S_{d1}) design earthquake spectral response acceleration should be investigated to determine cause of variation.
    b. Where these variations are determined to be a result of updated information on regional seismicity or ground motion attenuation models for which there is strong consensus the new SDC mapped categories can be updated accordingly.
    c. Where these variations are determined to be a result of updated information on regional seismicity or ground motion attenuation models that are still undergoing significant development and for which there is not general consensus, the new SDC mapped categories can remain unchanged.
    d. At all locations with NSHM variations of < 10%, SDCs should remain unchanged.

Proposals | Files | Admin

JiQiu Yuan

National Institute of Building Sciences

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P17 No.3 (reballot) - SDC_Proposal_Balloted REV1

Status

Finalized

Scope

Chapter 11: Update Seismic Design Category Definition and modify Section 11.6 to refer to Seismic Design Category Maps in lieu of Tables 11.6-1 and 11.6-2. Insert Seismic Design Category Maps in Section 11.6 for use by the designer. Remove analytical procedures used to determine Seismic Design Category.

Section 12.14.1.1 Simplified Design Procedure: update reference to Chapter 11 identifying how Seismic Design Category is to be determined to point to new Chapter 11 Seismic Design Category Maps.

11.6 Commentary: update commentary to include discussion on the need and development of Seismic Design Category maps in Chapter 11, Section 11.6. Commentary also includes information currently provided in Sections 11.4 through 11.6 outlining the procedures used to determine the Seismic Design Category.

Proposal IT3-1A-Rev.-2018-04-14 (Multi-period Spectral Proposal): The SDC Map proposal assumes incorporation of the MSP proposal with all relevant sections prior to 11.6 already modified by such. The MSP proposal language was not repeated here for clarity.

Proponent

Julie Furr

Voting Period

07-03-2018 4:06 PM – 07-20-2018 11:59 PM

Proponent Comment Period

07-22-2018 10:05 AM – 08-12-2018 11:59 PM

Supporting Files

Figure_11.6.1_SDC_Map_I_II_III.docx 2018-07-03 11:12:08
P17_No3_SDC_Proposal_Balloted_REV1.docx 2018-07-03 11:12:08

Proposal Updates

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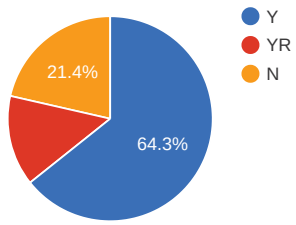
Vote Summary for P17 No.3 (reballot) - SDC_Proposal_Balloted REV1

Results Report

Total Voting	Voted	50% Rule	Yes	Yes with Reservations	No	Not Voting	Online Ballot	PUC Meeting Final
15	14	Meet	9	2	3	0	Pass 	The Proposal is passed and forwarded to PUC (see detailed discussion at the 2018-8-14 P17 meeting minutes)

Vote and Comment Summary

Download Votes
Download Votes and Responses

Vote Key

Yes — Yes with reservations

No — **NV** Not Voting

Response Key

Persuasive — Non-Persuasive

NR Non-Responsive — Editorial/Persuasive

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
Bonneville		General	General	I remain opposed to the idea of establishing seismic design categories independently of site class, so I would vote no for that reason. Fortunately this proposal, which is intended to address oscillations in seismic design parameters, especially SDC, is no longer needed, or at least not nearly as-needed. The PUC proposal on SDC consolidation being concurrently balloted (IT 01-1) addresses this issue more comprehensively, and does so without ignoring the effect of site class on seismic demand. That proposal, which offers other appropriate simplifications in the way we categorize sites for seismic design purposes (as well as in analysis), will also minimize oscillations between SDC's by broadening the bounds of them. Where there are currently three classes of SDC at the low and moderate seismic levels, there would now be two, and where there are three classes at the high seismic level, there will now be one. I would prefer that we go with that approach and not burden future PUC's with drawing maps, especially ones that ignore an important factor in ground motion.	If the SDC consolidation proposal fails or is withdrawn, so that that alternative is not available (which I would not like to see happen) I would change my vote to yes if site-specific determination of SDC is permitted.		

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
Crouse		Proposed SDC Maps		In many locations there are very small patches of an SDC within a much larger area of the next smaller SDC, which may be due to the presence of active faults or concentration of seismicity. However, there is one location in south central Idaho where the opposite is observed, i.e., a small patch of SDC B that is one SDC increment lower than the SDC C for the larger surrounding area. This small patch appears to include Twin Falls, but what is the reason for the lower ground motion resulting in a lower SDC in this small area?	Investigate and modify if necessary, depending on other proposals that may affect this proposal.		
Dolan							
Enfield							
Furr							
Hamburger							
Harris		General	General	My “NO” on this proposal is not because I am unsympathetic to the problems caused by the “yo-yo” effect of changes in the ground motions used for design. That is truly a problem. I have not seen a solution that I believe is viable, although the method used in this proposal is the best of any I have seen in that regard. My fundamental objection is that the effect of site conditions on the ground motion demand is purposely ignored. The argument proposed for this is that building designers, contractors, and regulators are incapable of dealing with multiple design categories in one city. In engineering practice one must strike a balance between chasing unattainable optimization and approximations that can become too crass; such approximation can	Withdraw the proposal		

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
				become either too costly or too unsafe. In my judgment, based primarily on my experience, engineers are certainly capable of dealing with multiple SDCategories. Jurisdictions are certainly capable of making decisions that are reasonable for their particular setting. And contractors, fabricators, and suppliers adapt to constraints more binding that our SDCategories. Also in my opinion, the effect of ignoring site effects is too coarse, except perhaps in the truly high seismic hazard areas. For low to moderate areas the ground motion varies dramatically with site class. This is not just a theoretical concern. The Loma Prieta earthquake is a teaching example – not for California, but for low to moderate hazard areas. The bedrock motion in San Francisco and Oakland during that event was in the range of the MCE for the bottom end of SDCategory B. There was real damage in soft soil areas, where those motions were amplified to moderately high levels. Those same amplifications will not occur under an MCE event at that location, but they are a real lesson for places with lower hazards. The lesson cuts two ways: if the site is hard and the rock motion is low, don't waste money and time on seismic protection, whereas if the site is soft, beware.			
Heintz							
Holmes							
Hooper				While the proposed	I would change my vote to		

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
				mapping of SDC will provide uniformity in design and construction in locations that are on an SDC border, the overall geographical area affected by this is actually quite small. Overall, I'm not in favor of preparing SDC maps. Additional reasons include: • The proposed changes will result in added conservatism to the building code, merely to solve a problem that only impacts small areas of the country which are generally on the SDC C/D boundary. • The code should allow for a site-specific determination of SDC. The authors intentionally chose not to incorporate this to stabilize construction practice within a region. Design constructability should remain within the realm of engineering judgement. It's understandable that there is risk involved with a contractor who infrequently needs to build with stricter detailing. However, infrequently working a project with relaxed detailing requirements does not pose the same risk. • In the MPRS ballot, the default site class is defined based on Site Class C, CD, and D "unless the authority having jurisdiction or geotechnical data	"yes" if site-specific determination of the SDC was retained.		

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	Proponent's Response
				determine that Site Class DE, E or F soils are present at the site." How will this be incorporated in the maps? If there is no provision for a site specific SDC there will be sites that are unconservative.			
Kircher				I have two reservations: (1) Mapped SDC regions shown in Figure 11.6.1 (and Figure 11.6.2) are based on the "default site class" (e.g., of the MPRS proposal) that may change before adoption affecting SDC regions shown in thes maps. (2) Mapped SDC regions shown in Fgiure 11.6.1 (and Figure 11.6.2) are overly precise and do not consider jurisdictional boundaries. Practical smoothing of SDC boundaries (e.g., considering jurisdictional boundaries) would improve the usefulness of the mapped regions of SDC. At the very least, small "islands" of a given SDC in the midst of larger region of neighboring SDC could be removed.	See comment (2)		
Pekelnicky							
Siu							
Stewart							

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nehrp

PROPOSAL P17-3-Rev. 1-2018-07-01

Seismic Design Category Maps

Submitted by: Julie C. Furr, PE
901-426-6010
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Original Proposal Submission date: 4/11/2018

SCOPE: Part 1 and Corresponding Part 2 Commentary

Chapter 11: Update Seismic Design Category Definition and modify Section 11.6 to refer to Seismic Design Category Maps in lieu of Tables 11.6-1 and 11.6-2. Insert Seismic Design Category Maps in Section 11.6 for use by the designer. Remove analytical procedures used to determine Seismic Design Category.

Section 12.14.1.1 Simplified Design Procedure: update reference to Chapter 11 identifying how Seismic Design Category is to be determined to point to new Chapter 11 Seismic Design Category Maps.

11.6 Commentary: update commentary to include discussion on the need and development of Seismic Design Category maps in Chapter 11, Section 11.6. Commentary also includes information currently provided in Sections 11.4 through 11.6 outlining the procedures used to determine the Seismic Design Category.

Proposal IT3-1A-Rev.-2018-04-14 (Multi-period Spectral Proposal): The SDC Map proposal assumes incorporation of the MSP proposal with all relevant sections prior to 11.6 already modified by such. The MSP proposal language was not repeated here for clarity.

REASON FOR PROPOSAL:

Seismic parameters have oscillated in response to updates in the National Seismic Hazard Maps (NSHM) and ASCE 7 provisions, creating what is commonly referred to as the yo-yo effect, wherein mapped hazard values, and required design forces oscillate from one edition of the standard to the next. Often these oscillations are within the margin of error for ground motion calculations and may simply be the result of tweaks in GMPEs or other modeling techniques. However, in some regions this change is enough to jump the numerical boundaries between different Seismic Design Categories (SDCs) as currently established in Tables 11.6-1 and 11.6-2. Because system detailing requirements and limitations are established through the designated SDC, these variations mean systems have been allowed/disallowed/allowed as new ASCE 7 versions are published. Aside from stakeholder frustration, this has real consequential impacts from loss of public confidence in ASCE 7 seismic provisions to lack of established design experience by engineers due to changing design requirements and also inability of construction practice to stabilize within a region.

SDC maps are proposed as a means to establish stability in Seismic Design Category and minimize the practical effects of oscillations on the design and construction communities, without artificially constraining ground motion values established by science. SDC maps, which are site class independent, will allow stabilization of design and construction practice within regions. As the NSHM's evolve over time, the PUC and other code groups will be able to review the effect of changes in mapped parameters, and identify geographical regions where SDCs would change. At these locations, a subjective review can be performed to determine if the underlying cause(s) of ground motion variation justifies changing the SDC in that region. If the change is a) within the reported margin of error, or b) due to tweaks in scientific methodology, future code committees can allow the SDC to remain as previously published, promoting stability. Alternately, if the change is due to significant new scientific information or understanding, and results in a substantive change in risk to the built infrastructure, the future code committees can change the SDC. The SDC map will provide a stable set of system detailing requirements and limitations, similar to the old zone maps, that stakeholders will grow to be familiar with. Where changes do occur, a justifiable rationale can be provided to stakeholders defending and explaining the change in that region.

Note that an earlier version of this proposal was balloted to the Project 17 committee and failed. Comments generally fell into the following categories:

1. The proposal was not coordinated with the multi-period spectrum proposal
2. The proposal did not include a copy of the maps that would be used in the Provisions
3. The proposal did not allow the use of Site Class to determine SDC, resulting in excessive seismic design requirements for structures on sites with better conditions than the default site class

This revised proposal is coordinated with the multi-period spectrum proposal in that it references the construction of the SDC maps using the default site class, as defined in the multi-period proposal. This proposal also includes the map (Figure 11.6-1) constructed using the 2014 National Seismic Hazard Model. Figure 11.6-2, which would be applicable for Risk Category IV structures would be similar to Figure 11.6-1 except that areas shown as SDC B or C will be shown as SDC C and D respectively.

It is recognized that the updated National Seismic Hazard model will result in a significant change in these maps. That should be considered by the PUC when considering adoption of the updated seismic hazard maps. It should also be noted that IT-01 is considering a proposal that will consolidate the present 6 seismic design categories (A, B, C, D, E and F) into three categories: (low, moderate, and high). If that proposal is successful, Figure 11.6-1 and 11.6-2 will change accordingly.

The working group elected to continue to make determination of SDC independent of Site Class because it believed the benefits of promoting uniformity of seismic design and construction practice within a region outweighed those associated with permitting lesser seismic design on some sites with firm soils or rock.

Modification to CHAPTER 11 SEISMIC DESIGN CRITERIA

Sections 11.6

11.6 SEISMIC DESIGN CATEGORY

Each structure shall be assigned to a Seismic Design Category. The Seismic Design Category for structures in Risk Category I, II or III shall be ~~in accordance with this section~~as determined from Figure 11.6.1. The Seismic Design Category for structures in Risk Category IV shall be determined from Figure 11.6.2.

Add Figure 11.6.1: SDC Map for Risk Category I/II/III Structures

Add Figure 11.6.2: SDC Map for Risk Category IV Structures

~~Risk Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Risk Category IV structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a Seismic Design Category based on their Risk Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in accordance with Section 11.4.5. Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 11.6-1 or 11.6-2, irrespective~~

~~TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter~~

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

~~TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter~~

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

~~of the fundamental period of vibration of the structure, T. The provisions in Chapter 19 shall not be used to modify the spectral response acceleration parameters for determining Seismic Design Category.~~

Where S_1 is less than 0.75, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone where all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, determined in accordance with Section 12.8.2.1 is less than $0.8T_s$, where T_s is determined in accordance with Section 11.4.6.
2. In each of two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Eq. (12.8-2) is used to determine the seismic response coefficient C_s.
4. The diaphragms are rigid in accordance with Section 12.3; or, for diaphragms that are not rigid, the horizontal distance between vertical elements of the seismic force-resisting system does not exceed 40 ft (12.192 m).

Where the alternate simplified design procedure of Section 12.14 is used, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone, using the value of S_{DS} determined in Section 12.14.8.1, except that where S_1 is greater than or equal to 0.75, the Seismic Design Category shall be E.

Modification to CHAPTER 12 SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

Section 12.14.1

12.14.1 General

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 from Figure 11.6-1. ~~Table 11.6-1 using the value of S_{DS} from Section 12.14.8.1, except that where S1 is greater than or equal to 0.75, the Seismic Design Category shall be E.~~ The simplified design procedure is permitted to be used if the following limitations are met:

[balance of section to remain unchanged]

Modify C11 SEISMIC DESIGN CRITERIA

Modify 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 U.S. building. However, SDCs also depend on a building's occupancy and, therefore, its desired performance. ~~Furthermore, unlike the traditional implementation of seismic zones, the ground motions used to define the SDCs include the effects of individual site conditions on probable ground shaking intensity.~~

In developing the ground-motion 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 current design philosophy was adopted from the 1997 NEHRP provisions and Commentary (FEMA 1997a and FEMA 1997b), 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 SDC E

An important change was made to the determination of SDC when the current design philosophy was adopted. 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 one category higher for Risk Category IV structures than for lower risk structures. This rating has the effect of increasing the confidence that the design and construction requirements can deliver the intended performance in the extreme event.

Note that the tables in the standard 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. Note that this boundary is defined in terms of mapped bedrock outcrop motions affecting response at 1 s, not site-adjusted values, to better discriminate between sites near and far from faults. Short-period response is not normally as affected as the longer period response. 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.

Since their introduction into the Provisions, in the 1997 edition, SDC has been determined on a site-specific 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 to the next. The 2021 edition of the Provisions specified determination of SDC based on a default site class condition and in accordance with Risk-Category specific maps.

A default site class was selected, as opposed to actual site class conditions, to promote uniformity of seismic design and construction practices within regions and also to minimize the fluctuation of SDC assignments resulting from relatively modest changes in mapped values of seismic hazard from one edition of the national seismic hazard maps to the another edition.

~~For most buildings,~~ ~~t~~The SDC is determined without consideration of the building's period. Structures are assigned to an SDC based on the more severe condition determined from 1-s acceleration and short-period acceleration. This assigning is 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 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 misclassify-ing a building's SDC by inaccurately estimating the fundamental period, the standard generally requires that the more severe SDC determined on the basis of short- and long-period shaking be used.

Another reason for this requirement is a desire to simplify building regulation by requiring all buildings on a given soil profile in a particular region to be assigned to the same SDC, regardless of the structural type. This assignment has the advantage of permitting uniform regulation in the selection of seismic force-resisting systems, inspection and testing requirements, seismic design requirements for nonstructural components, and similar aspects of the design process regulated on the basis of SDC, within a community.

~~Notwithstanding the above, it is recognized that classification of a building as SDC C instead of B or D can have a significant impact on the cost of construction. Therefore, the standard includes an exception permitting the classification of buildings that can reliably be classified as having short structural periods on the basis of short-period shaking alone.~~

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, ~~particular site classes,~~ or particular Seismic Design Categories for all or part of the area of their jurisdiction. For example,

1. An area with a historical practice of high seismic zone detailing might mandate a minimum SDC of D regardless of ground motion ~~or site class.~~

2. 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.

3. ~~An area with unusual soils might require use of a particular site class unless a geotechnical investigation proves a better site class.~~

Seismic Design Category (SDC) maps were developed in accordance with the following rules:

1. Where S_1 is less than or equal to 0.04 and *Ss* is less than or equal to 0.15, Seismic Design Category A is designated.
2. Where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 Seismic Design Category E is designated for Risk Category I, II and III structures, and Seismic Design Category F is designated for Risk Category IV structures.
3. In all other regions Seismic Design Category is assigned based on Risk Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined assuming default site class conditions. In each location, SDC is assigned as the more severe condition determined in accordance with Table C11.6-1 or C11.6-2.

[Add Tables C11.6-1 and C11.6-2]

TABLE C11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

TABLE C11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

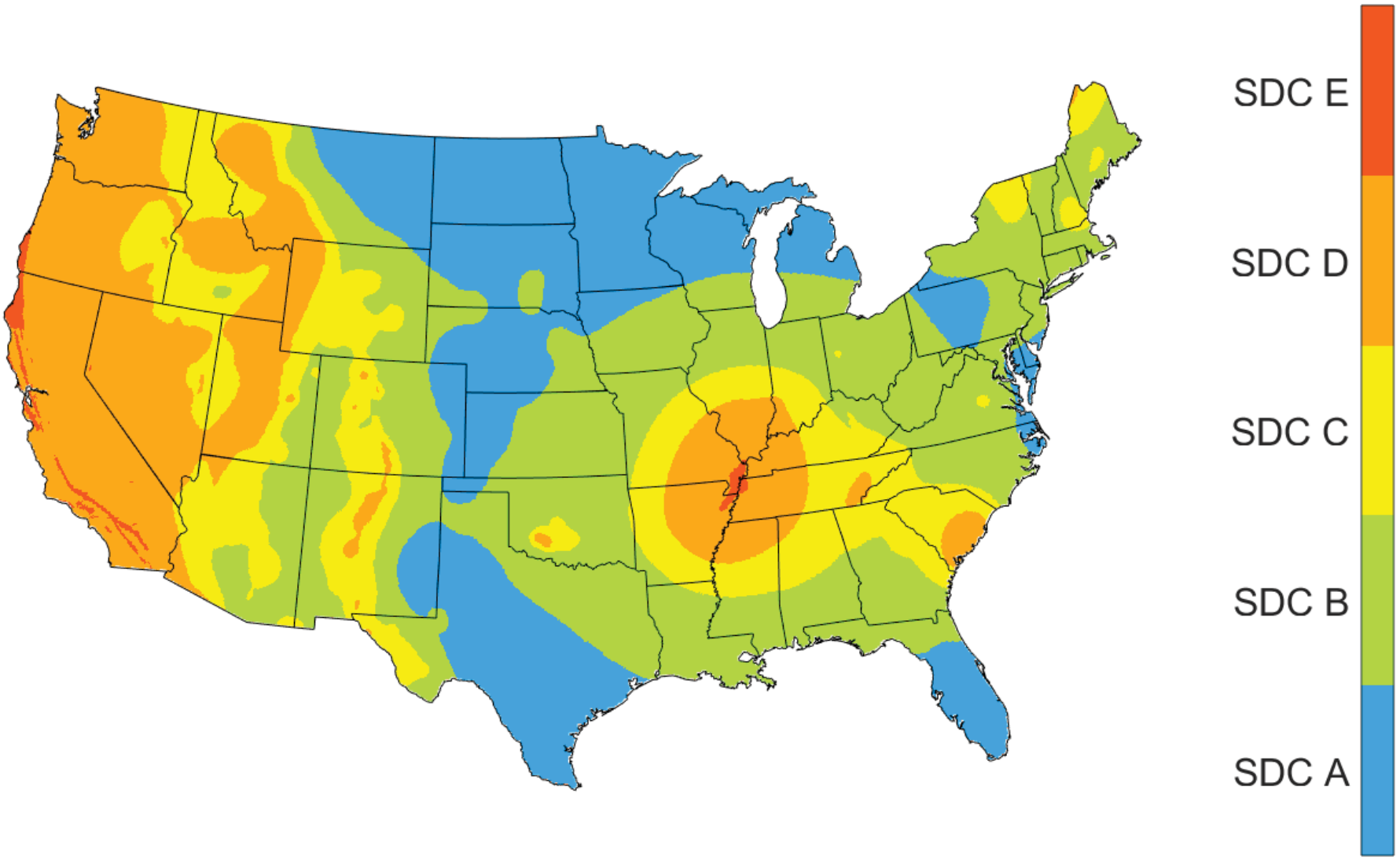

Figure 11.6.1: SDC Map for Structures in Risk Category I/II/III (Based on 2014 USGS Update)

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JiQiu Yuan

National Institute of Building Sciences

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P17 - Project 17 ballot on Deterministic Caps

Status

Finalized

Scope

Please vote yes or no to the following:

In areas near high-activity faults, rather than limiting risk-targeted ground motions by deterministic caps, MCER ground motion should be determined using a graded probabilistic risk, with the range of target risks and gradation to be determined

Proponent

Jonathan Stewart

Voting Period

01-30-2018 3:00 PM – 02-12-2018 11:59 PM

Proposal Updates

Created: 01-30-2018 10:10 AM
Updated: 05-08-2018 1:49 PM

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Vote Summary for P17 - Project 17 ballot on Deterministic Caps

Results Report

Total Voting	Voted	50% Rule	Yes	Yes with Reservations	No	Not Voting	Online Ballot	PUC Meeting Final
15	14	Meet	5	1	8	0	Fail 'null' is not an object ×	

Vote and Comment Summary

Download Votes

Vote Key

Y Yes
YR Yes with reservations
N No
NV Not Voting

Last Name	Vote	Page #	Line #	Comment	Suggested Change	File	PUC Meeting
Bonneville	N			Accepting the previous decision to retain the current risk model.	No change suggested.		
Crouse	Y						
Dolan	Y						
Enfield	N	1	1	Filler text as instructed. The risk that this proposal will cause ASCE 7-22 to not be adopted, or for ASCE 7 to be questioned in general is too great. I would prefer for the code to be ideologically consistent, but	Filler text as instructed.		

				the risk to ASCE adoption is taking priority.			
Furr	Y						
Hamburger	N	1	1	This is not a good idea	Stay with Deterministic cap		
Harris	YR	all	all	I cast this vote because I believe the proposal has a germ of the proper future direction. I do not expect it to suceed in this cycle, but if we do not pursue it, then the chance for future improvement is diminished. In my opinion the fundamental reason that we place a deterministic cap on ground motions is that we do not believe the treatment of uncertainty in ground motion prediction is correct where the primary uncertainty is the amplitude of the ground motion rather than the occurence of the event. However, we actually don't care about that as much as the effect of the very high ground motions on design and construction. This last fact is something that we should be able to quantify and build into our methodology for arriving at design ground motions. In other words, we establish an acceptable risk level that varies with the cost of building to withstand high ground motions. This particular study should proceed in parallel with what we need right now, which is the multi-period definition of ground motions. BSSC's job is to look to the future, thus we should not toss this out at this time.			
Heintz	N			I support ideas for removal of deterministic caps, and for being more transparent about actual collapse probabilities, but preferrred other ideas such as changing the return period on the definition of MCE rather than this proposal.	I preferrred other ideas previously discussed, such as changing the return period on the definition of MCE rather than this proposal.		
Holmes	N			Increased technical transparency is offset by increased difficultly in explaining the rationale and probable national reaction.			
				None	None		

Hooper	Y						
Kircher	N	none	none	I am in favor of retaining the (maximum magnitude) approach of ASCE 7-16 for determination of the determinsitc MCEr, possibly with an updated set of maximum magnitudes	None		
Pekelnicky	N			I am voting no on this ballot for several reasons. First, this approach was discussed by the acceptable risk working group over a year ago and did not have a lot of enthusiasm behind it. Second, as was brough up the AR group, we would effectively be using our collective judgement to set when, where, and how much the risk is to adjust. That is something that requires more time to come to consensus on than we have time in PRoject 17 (unless there is an extension). Third, in my opinion, this still promulgates the issue of providing non-uniform risk across the country.	Use a uniform return period of 1,500 years, which will provide between a 1 and 2% risk of collapse with an epsilon between 0 and 1 for most of the country.		
Siu	N	General	General	While I'm not a fan of the deterministic cap, and very much agree that the proposal would increase transparency, my concern is that there will be a backlash that could result in not adopting ASCE 7-22 in its entirety in the 2024 IBC, as almost happened with ASCE 7-16 and the 2018 IBC. And if ASCE 7-22 doesn't get adopted, then we're at status quo (or worse) anyway. 99% of the country, even though they aren't directly affected by this proposal, will ask the question, "If higher risk is OK in those parts of California, why isn't it OK everywhere else?" It looks like California engineers aren't willing to live up to the same standard as everyone else. I know that leveling the risk across the country was voted down, but this proposal will just highlight what some building officials and engineers already view as an	Since leveling risk is not on the table, maintain current deterministic cap methodology.		

				inequity, making acceptance and adoption more difficult.			
Stewart	Y						

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