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NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provisions

2026 Edition

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NEHRP (National Earthquake Hazards Reduction Program)

Recommended Seismic Provisions

for New Buildings and Other Structures

2026 Edition

Volume II: Part 3 Resource Papers

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

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

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2026 NEHRP RECOMMENDED SEISMIC PROVISIONS FOR NEW BUILDINGS AND OTHER STRUCTURES

PART 3, RESOURCE PAPERS

This part of the 2026 NEHRP Provisions consists of a series of Part 3, Resource Papers. They introduce new concepts and procedures for trial use by the design community, researchers, and standards-development and code-development organizations. They also represent Issue Team efforts on substantive proposals for topics that require further consideration by the seismic design community and additional research before being considered for Parts 1 and 2 provisions. In some cases, the resource papers provide additional information and background for the recommended changes and commentary in Parts 1 and 2. Like Parts 1 and 2 of the provisions, these resource papers have been approved for inclusion in this volume by both the 2026 Provisions Update Committee (PUC) and the Building Seismic Safety Council (BSSC) member organizations. These resource papers are self-contained and not written in standards language format. Feedback from these users is encouraged.

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RESOURCE PAPER 1 STRUCTURAL STAINLESS STEEL SEISMIC FORCE-RESISTING SYSTEMS

B.W. Schafer¹, F. Meza², N. Baddoo³, B.R. Baer⁴, B. Manley⁵

RP1-1 Introduction

Since the last edition of ASCE 7/NEHRP, stainless steel has seen significant formal standardization. The development and adoption of AISC 370-21 and ASCE 8-22 provide a robust and level playing field for the formal use of stainless steel in building and non-building civil structures. However, no equivalent seismic standards have been developed for structural stainless steel—leaving a gap that engineers must fill without formal guidance.

Currently, the market is too small to justify multi-year expenditures to develop a seismic structural stainless steel standard—but the market cannot grow without these standards. Given stainless steel utilization in critical national facilities such as nuclear plants, water treatment facilities, food processing, pharmaceuticals, and multiple other industrial settings, these limitations have negative societal impacts that need to be addressed.

A task group within the AISC 370 standards committee developed a state-of-the-art understanding of the seismic performance of structural stainless steel and summarized their findings in Meza et al. (2025) (<https://doi.org/10.31224/4354>). The objective of this resource paper is to pull forward key recommendations and suggest how structural stainless steel seismic force-resisting systems (SFRS) might be utilized in future editions of ASCE 7. In addition, this resource paper should be the first step

¹ Department of Civil and Systems Engineering, Johns Hopkins University, <schafer@jhu.edu>

² Steel Construction Institute, UK, <F.Meza@steel-sci.com>

³ Steel Construction Institute, UK, <n.baddoo@steel-sci.com>

⁴ Consulting Structural Engineer, US, <baeroffice@att.net>

⁵ American Institute of Steel Construction, US, <manley@aisc.org>

toward prioritizing a future BSSC issue team to further develop seismic structural stainless steel provisions in the next edition of the BSSC NEHRP Provisions.

RP1-2 Draft Provisions for Chapter 14

Given that there is little expectation that a standalone seismic structural stainless steel standard will be created in the near term, the bulk of any provisions would need to be provided in Chapter 14 of ASCE 7. Draft language is provided along with highlighted text that provides brief commentary to address the reasoning where appropriate.

14.1 STEEL

Structures, including foundations, constructed of steel to resist seismic loads shall be designed and detailed in accordance with this standard, including the reference documents and additional requirements provided in this section.

14.1.1 Reference Documents The design, construction, and quality of steel members that resist seismic forces shall conform to the applicable requirements, as amended herein, of ANSI/AISC 341, ANSI/AISC 360, ANSI/AISC 370, ANSI/AISI S100, ANSI/AISI S230, ANSI/AISI S240, ANSI/AISI S310, ANSI/AISI S400, ANSI/SDI-QA/QC, ANSI/SDI-SD, ANSI/SJI-100, ANSI/SJI-200, ASCE 8, and ASCE 19.

... (14.1.2 – 14.1-9 not shown) ...

[Entirely New Section – therefore not shown in legislative text.]

14.1.10 Structural Stainless Steel

14.1.10.1 General The design of structural stainless steel for buildings and structures shall be in accordance with ANSI/AISC 370. Where required, the seismic design of structural stainless steel structures shall be in accordance with the additional provisions of Section 14.1.10.2.

The provisions in ANSI/AISC 370 for increased strength of HSS due to cold working are permitted to be used in conjunction with the requirements herein.

14.1.10.2 Seismic Requirements for Structural Stainless Steel Structures The design of structural stainless steel structures to resist seismic forces shall be in accordance with the provisions of Section 14.1.10.2.1 or 14.1.10.2.2, as applicable.

14.1.10.2.1 Seismic Design Categories B and C. Structural stainless steel structures assigned to Seismic Design Category B or C shall be of any construction permitted by the applicable reference documents in Section 14.1.1. Where a carbon steel response modification coefficient, R , is used in accordance with Table 12.2-1 for the design of structural stainless steel structures assigned to

Seismic Design Category B or C, the applicable requirements of Table 12.2-1 shall be subject to the following modifications:

1. Stainless steel $\Omega_o = 1.5 \times$ carbon steel Ω_o , but not greater than R , and
2. Stainless steel $C_d = 1.5 \times$ carbon steel C_d , but not greater than R .

Structural stainless steel systems utilizing carbon steel response modification coefficients, R , shall be designed and detailed in accordance with the requirements of AISC 341, subject to the modifications of 14.1.10.3.

EXCEPTION: The response modification coefficient, R , designated for “steel systems not specifically detailed for seismic resistance, excluding cantilever column systems” in Table 12.2-1 shall be permitted for structural stainless steel systems designed and detailed in accordance with AISC 370.

Commentary 14.1.10.2.1 Exception: “For steel systems not specifically detailed for seismic resistance, excluding cantilever columns,” the overstrength factor, Ω_o , and deflection amplification factor, C_d , are already at their maximum values (equal to R), and overstrength and ductility in stainless steel systems are not lower than in carbon steel systems, as discussed further in Meza et al. (2025).

14.1.10.2.2 Seismic Design Categories D through F. Where a carbon steel response modification coefficient, R , is used in accordance with Table 12.2-1 for the design of structural stainless steel structures assigned to Seismic Design Category D through F, the applicable requirements of Table 12.2-1 shall be subject to the following modifications:

1. Stainless steel $\Omega_o = 1.5 \times$ carbon steel Ω_o , but not greater than R , and
2. Stainless steel $C_d = 1.5 \times$ carbon steel C_d , but not greater than R .

Structural stainless steel systems utilizing carbon steel response modification coefficients, R , shall be designed and detailed in accordance with the requirements of AISC 341, subject to the modifications of 14.1.10.3.

Commentary 14.1.10.2.2: Justification for using the same response modification coefficient, R , in structural stainless steel seismic force-resisting systems as carbon steel systems is provided in Meza et al. (2025). The reasoning is primarily based on material and member performance—not extensive system-level experiments or numerical studies. Mitigating this uncertainty in the use of the same response modification coefficient, R , is the application of increased overstrength factor, Ω_o , and deflection amplification factor C_d , for structural stainless steel. Additional overstrength in stainless steel is expected due to higher amounts of post-yield strain hardening and cyclic hardening—such effects are alloy and grade-dependent, with austenitic stainless steels exhibiting a higher strain hardening modulus (E_{st}), and ratios F_u/F_y and F_{cyc}/F_{mon} than duplex stainless steels. Increasing Ω_o by a factor of 1.5 is expected to be close to a mean response for large strain amplitudes in austenitic and an upper bound on duplex stainless steels. In the absence of more refined overstrength numbers, this is recommended. Deflection amplification is also

conservatively increased, primarily in recognition of the rounded nature of the stress-strain curve at smaller strains. Based on current research, it is expected that a refined analysis of a specific system could result in higher response modification coefficients, R , similar or lower overstrength factors, Ω_o , and lower deflection amplification factor, C_d , than currently recommended, but the provided method gives the best available estimates within current knowledge. Application of the proposed method results in the seismic response modification coefficients for common structural stainless steel seismic force-resisting systems, as indicated in Table 1.

Seismic Force-Resisting System ^a		R	Ω_o	C_d
Moment frames	OMF	3.5	3.5	3.5
	IMF	4.5	4.5	4.5
	SMF	8	4.5	8
Centrally braced frames	OCBF	3.25	3	3.25
	SCBF	6	3	6
Eccentrically braced frames	EBF	8	3	6
Buckling-restrained braced frames	BRBF	8	3.75	7.5
Note: OMF = Ordinary Moment Frame; IMF = Intermediate Moment Frame; SMF = Special Moment Frame; OCBF = Ordinary Centrally Braced Frame; SCBF = Special Centrally Braced Frame; EBF = Eccentrically Braced Frame; BRBF = Buckling-Restrained Braced Frame.				

Note a: the Table 1 coefficients are recommended for design; however, particularly for the highest ductility systems such as SMF, EBF, and BRBF, the registered design professional should carefully consider exceptions to AISC 341 beyond those addressed herein and, wherever possible, seek experimental evidence of performance.

Table 1. Structural Stainless Steel SFRS

14.1.10.3 Modifications to AISC 341 for Application to Stainless Steel Structures The carbon steel provisions of AISC 341 shall be applied to structural stainless steel seismic force-resisting systems subject to the modifications in this section.

Materials shall include stainless steel alloys as listed in AISC 370, Section A3.1b. The expected material strength factors R_y and R_t shall be modified in accordance with Table 14.1-1.

R_y		R_t	
Austenitic	Duplex	Austenitic	Duplex
3.1	2.6	1.4	1.4

Table 14.1-1. Expected Material Strength Factors for Structural Stainless Steel SFRS

Commentary 14.1.10.3: The provided values for the expected material strength factors R_y and R_t are simplified from the development provided in Meza et al. (2025). These factors include (1) the ratio of the expected yield (ultimate) stress with a 5% probability of exceedance to the nominal specified yield (ultimate) stress, (2) an estimate of the cyclic hardening effect, and (3) an estimate of the monotonic strain hardening within the expected strain range. The resulting expected yield stress is significantly larger for austenitic stainless steels than carbon steel.

References to AISC 360 for nominal member and connection strength calculations (P_n , V_n , M_n , etc.) shall be replaced by the equivalent provisions in AISC 370, Chapters D through K.

Commentary 14.1.10.3, cont.: AISC 370 allows the nominal strength to be calculated per AISC 370 Chapters D through K, which essentially employs the elastic-plastic approximation that AISC 360 employs—or per the Continuous Strength Method (CSM) Appendix 2, which uses the actual stress-strain response, or a close approximation, to determine the member capacity. The use of CSM in seismic design is beyond the scope of the current provisions, but initial guidance is provided in Meza et al. (2025).

Demand critical welds, as required by AISC 341, Section A3.4b, shall be subject to the additional requirements of AWS D1.8 clauses 6.1, 6.2, and 6.3. Stainless steel demand critical welds meet the requirements of AWS D1.6 and, through documented testing, the performance (toughness) requirements of AWS D1.8.

Commentary 14.1.10.3, cont: Stainless steel welding specifications do not have a direct equivalent to the carbon steel AWS D1.8 seismic provisions. With the right filler metal and weld procedure, stainless steel weld metal can meet CVN requirements such as 20 ft-lbf at -20°F; however, given the lack of a prescriptive standard, the performance must be documented through testing, e.g., from the manufacturer, the fabricator, tested by a third-party lab, or a weld specialist or engineer.

Stability bracing in accordance with AISC 341, Section D1.2a, Moderately Ductile Members, shall be modified for use with AISC 370 by replacing the existing AISC 341, Eq. D1-2 with the following:

$$L_b = \frac{2.5L_p}{\sqrt{R_y}} \quad (\text{D1-2})$$

Where L_p is defined in AISC 370 and R_y is given in Table 14.1-1. Stability bracing, L_b , for AISC 341 D1.2b for Highly Ductile Members shall be taken as $\frac{1}{2} L_b$ calculated in accordance with Eq. D1-2.

Commentary 14.1.10.3, cont.: AISC 341 uses the same $\frac{1}{2}$ reduction in AISC 341, Section D1.2b for stability bracing for highly ductile members. AISC 370 bracing requirements are alloy dependent, generally following AISC 360 requirements in terms of stiffness, but using 1.5 times AISC 360 requirements in terms of required brace strength. The proposed modifications make AISC 341 bracing provisions consistent with AISC 370, as derived in Meza et al. (2025).

Composite construction with stainless steel and concrete shall be excluded.

Commentary 14.1.10.3, cont.: AISC 370 does not currently include provisions for composite construction, so seismic composite construction remains outside the scope of these recommendations.

Any exceptions to AISC 341 beyond those addressed in this section shall be noted and justified by the registered design professional for approval by the AHJ.

Commentary 14.1.10.3, cont.: Use of AISC 341 for stainless steel, even with the modifications provided here, requires judgment by the RDP. This clause recognizes the possibility and requires that the RDP note these conditions in their design and seek approval by the AHJ.

RP1-3 Draft Provisions for Chapter 15

Chapter 15 covers non-building structures similar to buildings, non-building structures not similar to buildings, and tanks and vessels. These are all structures that may be commonly comprised of stainless steel. Although AISC 370 primarily covers building-like structures, an attempt to provide useful guidance for applications with stainless steel has been made in these proposed changes. All proposed changes to Chapter 15 are in the commentary only, so they are shown in grey highlighted text; other text exists in the standard.

15.4 STRUCTURAL DESIGN REQUIREMENTS

15.4.1 Design Basis Non-building structures that have specific seismic design criteria established in reference documents shall be designed using the standards as amended herein. Where reference documents are not cited herein, nonbuilding structures shall be designed in compliance with Sections 15.5 and 15.6 to resist minimum seismic lateral forces that are not less than the requirements of Section 12.8, with the following additions and exceptions.

Add Commentary 15.4.1: Structural stainless steel non-building structures may be designed in accordance with Section 14.1.10. Application of the method in Section 14.1.10, as detailed in Meza et al. (2025), results in the seismic response modification coefficients for common structural stainless steel nonbuilding seismic force-resisting systems, as presented in Table 2.

Seismic Coefficients for Non-building Structures		R	Ω_o	C_d
Steel storage racks		4	3	4
Steel cantilever storage racks	OMF	2.5	2.5	2.5
	OBF	3.25	3	3.25
Building frame systems	OCBF	3.25	3	3.25

	SCBF	6	3	6
Moment-resisting frame systems	OMF	3.5	3.5	3.5
	IMF	4.5	4.5	4.5
	SMF	8	4.5	8
Trussed towers (freestanding or guyed)		3	3	3
Signs and billboards		3	2.7	3
Steel lighting system support pole structures		1.5	1.5	1.5
Note: OMF = Ordinary Moment Frame; IMF = Intermediate Moment Frame; SMF = Special Moment Frame; OCBF = Ordinary Concentrically Braced Frame; SCBF = Special Concentrically Braced Frame				

Table 2. Structural Stainless Steel SFRS for Nonbuilding Structures Similar to and Not Similar to Buildings

15.7 TANKS AND VESSELS

15.7.1 General This section applies to all tanks, vessels, bins, silos, and similar containers storing liquids, gases, and granular solids supported at the base (hereafter referred to generically as “tanks and vessels”). Tanks and vessels covered herein include reinforced concrete, prestressed concrete, steel, aluminum, and fiber-reinforced plastic materials. Tanks supported on elevated levels in buildings shall be designed in accordance with Section 15.3.

Add Commentary 15.7.1: In the standard, the term “steel tanks” generally refers to welded, bolted, or corrugated carbon steel tanks. Stainless steel tanks are appropriate for use when the referenced material standard (per API, AWWA, etc.) includes stainless steel guidance.

RP1-4 Draft Provisions for Chapter 16

In general, the Nonlinear Response History Analysis (NLRHA) provisions of Chapter 16 are broad and applicable to structural stainless steel. As Meza et al. (2025) highlights, NLRHA is encouraged for stainless steel applications when the appropriate information is available (e.g., fully hysteretic moment-rotation performance of key connections) since the provided seismic response modification coefficients and procedures are conservative and approximate. ASCE 7, Chapter 16 points to ASCE 41 and highlights cyclic strength degradation. While structural stainless steel is not supported in ASCE 41, cyclic strength hardening/enhancement is as much a concern as degradation. Given the narrow use case for stainless steel, it is recommended to simply add commentary, shown in grey highlighted text, to Section 16.3.1 modeling.

Add Commentary 16.3.1: Modeling the seismic response of structural stainless steel may require extra effort by the registered design professional since ASCE 41 provides no guidance, and in some

cases, the material response is unique, providing cyclic strength enhancement rather than degradation. Meza et al. (2025) provides initial guidance from the available literature on the seismic response of structural stainless steel. Engineers using NLRHA will need tests or validated simulations for key nonlinear components, and this may be especially appropriate for high-value, high-consequence, stainless steel structures as employed in industrial and power (e.g., nuclear) settings.

RP1-5 Future needs for further development of seismic stainless steel.

The provided resource paper provides initial guidance for the seismic design of structural stainless steel, but significant work remains to fully enable these systems. Any of the following could provide important additions and could be the focus of a future issue team, working group, or research team:

1. Refine R_y and R_t factors for calculating the expected strength of austenitic and duplex stainless steels, including potential separation of variance (mean/nominal) and strain hardening (including cyclic effects). This is both doable and could provide essential improvements.
2. Provide detailed characterization of the cyclic hardening behavior of austenitic and duplex stainless steel, including the effects that different product forms and the level of cyclic strain may have on this phenomenon. Consideration of expected and lower-bound characterizations of cyclic hardening for use in seismic design and conversion into factors for use.
3. Standardize the extension of the Continuous Strength Method (CSM) for both equivalent lateral force design and nonlinear response history analysis. Recent Eurocode guidance (CEN 2023, Gardner and Walport 2024) for nonlinear static analysis of frames by CSM provides a starting point.
4. Develop stainless steel system archetypes and perform formal FEMA P695 (FEMA, 2009) development of seismic response modification coefficients for structural stainless steel seismic-force resisting systems. This is a significant task, but one likely most aligned with BSSC Provisions Update Committee (PUC) expectations and needs.
5. Utilize a formal standards process to develop a complete stainless steel equivalent to AISC 341.
6. Further refine expected strength and capacity-based design provisions for stainless steel members with strain hardening and cyclic hardening.
7. Develop and/or extend pre-qualified connections for structural stainless steel. This is a significant task but likely aligned with BSSC PUC expectations and needs.
8. Develop hybrid details such that stainless steel may be incorporated into a carbon steel system (e.g., as an EBF) as the energy dissipating element and still meet ‘demand critical’ weld criteria. Also, further work on the suitability of bolted connections between dissimilar metals in an SFRS is needed.

RP1-6 Conclusions

This resource paper provides initial guidance and recommendations for seismic design of structural stainless steel systems. Significant additional work is needed to enable the full efficiency of these important structural systems—but the provided recommendations can form the basis for safe, codified seismic structural design, utilizing best current knowledge.

RP1-7 References

Meza, F., Baddoo, N., Baer, B.R., Manley, B., Schafer, B.W. (2025). “Review: Structural stainless steel seismic force-resisting systems.” Preprint available at EngrXiv: DOI <https://engrxiv.org/preprint/view/4354/version/5947>. (Submitted to AISC Engineering Journal for possible publication in early 2025.)

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RESOURCE PAPER 2 DIAPHRAGM ACCELERATION COMPARISON BETWEEN 12.10.1, 12.10.3 AND 13.3: OBSERVATIONS AND FUTURE STUDIES

RP2-1 Introduction

One of the purposes of Issue Team 08 (IT-08) is to investigate ways to unify the diaphragm design provisions in ASCE 7 (ASCE, 2022) Chapter 12. In ASCE 7 Section 12.10, there exist two different ways of calculating diaphragm demands in buildings that don't fall under the rigid wall – flexible diaphragm provisions of Section 12.10.4: the traditional method in Section 12.10.1 and the alternative method in Section 12.10.3. These two methods are fundamentally different in the way diaphragm demands are determined, often producing different acceleration distributions throughout the height of the building.

As part of ASCE 7-22, a revised methodology for calculating demands on nonstructural components over the height of a building was introduced in Section 13.3. One goal of IT-08 for the 2026 NEHRP provisions was to understand the differences in diaphragm acceleration distributions between the methods in Sections 12.10.1, 12.10.3 and 13.3 in ASCE 7-22. This was investigated using characteristics and site parameters of 49 real buildings located within low to high seismic regions.

This resource paper has been developed by a sub-group of Issue Team 8 to document the study conducted by the sub-group. It provides observations on how each force calculation procedure compares to one another and provides recommendations for future steps needed to better understand the differences in distributions among these three procedures. Section RP2-2 of this resource paper summarizes the current diaphragm force provisions of Sections 12.10.1 and 12.10.3, as well as the nonstructural component design force provisions of Section 13.3 of ASCE 7. Section RP2-3 presents the building parameters used in this study, all assumptions made in the calculations used to compare the methods, and all limitations to the data. Section RP2-4 presents the findings and observation of this study. Section RP2-5 presents conclusions of this study. Section RP2-6 of this resource paper recommends next steps and makes recommendations for future studies.

RP2-2 Background and Current Provisions

Extensive discussion of the provisions of Sections 12.10.1, 12.10.3, and 13.3 are provided in the commentary of ASCE 7 (ASCE, 2022), therefore they will only be summarized here. The provisions of ASCE 7 permit the use of one of three diaphragm design procedures. The two that this paper will focus on are the traditional method of Section 12.10.1 and the alternative method of Section 12.10.3. The design equations of Section 12.10.1 have long been recognized as the standard diaphragm force equations that many in practice have used in building design. Not only is the expected performance of diaphragms not defined for these equations, but they also do not necessarily ensure that the diaphragm is designed in a manner that can mobilize the inelastic behavior of the vertical elements in the seismic force-resisting system (SFRS).

The design provisions of Section 12.10.3 were introduced in ASCE 7-16 (ASCE, 2017) and recognize the effect of diaphragm ductility, the diaphragm's interaction with the vertical elements of the SFRS, and the SFRS inelastic response effects on floor acceleration. Currently, Section 12.10.3 is only mandatory for precast diaphragms in buildings assigned to Seismic Design Category C, D, E, and F, since research has shown that some precast diaphragms designed using the traditional method in Section 12.10.1 produced designs that did not meet the performance safety objectives of ASCE. It has also been observed in past earthquakes that buildings with precast diaphragms designed with the provisions of Section 12.10.1 have a higher risk of collapse.

The diaphragm provisions of Section 12.10.3 equate the diaphragm design force to the tributary weight of the diaphragm, w_{px} , multiplied by a diaphragm design acceleration coefficient C_{px} , and divided by a diaphragm design force reduction factor, R_s . This force reduction factor is material and system dependent and accounts for the global strength and deformation capacity of the diaphragm. Values of R_s range from 0.7-3.0. Where values of R_s are greater than 1.0, it is expected that these diaphragms have a well-defined yielding mechanism that can be used to estimate the global strength and deformation capacity of the diaphragm.

Design accelerations for nonstructural components have historically been higher than those for diaphragms. Nonstructural components typically lack a ductile yielding mechanism and redundancy that allow building components to be designed with lower accelerations. The Applied Technology Council (ATC) project 120 (NIST, 2018) aimed to develop a more robust set of design equations for nonstructural components that account for parameters that will affect the response of nonstructural components. In ASCE 7-22, significant revisions to the equations of Section 13.3 were introduced based on the results of the ATC-120 project. The new equations incorporated into Section 13.3 consider ground shaking intensity, seismic parameters of the SFRS, building modal period, and building ductility. It is pointed out in the commentary of Section 13.3 that the acceleration profiles over

the height of a building between Section 12.10.3 and 13.3 will differ due to the differences in the equations used to compute the acceleration profiles, with Section 13.3 typically producing more conservative estimates of accelerations. The provisions of Section 12.10.3 utilize more building and site-specific parameters that consider higher mode effects and the period of the building, as defined in Section 12.8.2, typically leading to lower accelerations.

RP2-3 Collection of Building Data

A set of 49 buildings designed by Magnusson Klemencic Associates were used to compare the acceleration distribution over the height of the building using Section 12.10.1, 12.10.3 and 13.3. The data set consisted of low, mid and high-rise buildings located within low-to-high seismic regions. Diaphragms in this data set consisted of cast-in-place concrete (shear and flexure-controlled), concrete-filled steel deck, wood sheathed, and cross-laminated timber diaphragms. Since no R_s value is provided in ASCE 7-22 for cross-laminated timber diaphragms, a value of 1.0 was used for this study. While a variety of diaphragm and lateral systems were used in this data set, it is not so comprehensive as to include every system and diaphragm combination possible. Building and site information relevant to the acceleration calculations of Section 12.10.1, 12.10.3 and 13.3 methods are presented in Table A-1. All variables that are defined in ASCE 7 are not defined in this document.

For each building, acceleration distributions were determined for each methodology described above. Section 13.3 equations consider the amplification of nonstructural component acceleration relative to the total floor acceleration. The total response is based on the period of the nonstructural component and the spectral acceleration of the floor acceleration at that period. For the Section 13.3 procedure, the following variables were set to unity to eliminate any bias that could be produced due to the particular characteristics of the nonstructural component: R_{po} , C_{ar} , I_p .

After observations described in section RP2-4 were made, additional questions were raised about the relative distributions computed using the equations in Section 12.10.3 and Section 13.3. To better understand the differences, a separate study was conducted by setting the R_s and z_s factors of Section 12.10.3 to unity. While the results of this separate study are not included in the figures below, the findings are that this eliminated any bias that could be introduced into the equations from diaphragm ductility or higher mode effects on inelastic distribution of the lateral system.

RP2-4 Comparison Between Acceleration Distributions

The acceleration distributions computed using the provisions of Sections 12.10.1, 12.10.3 and 13.3 for select buildings are shown in Figure C-1. For comparison to ASCE 7-22, the equations of Section 13.3 presented in ASCE 7-16 are shown on Figure C-1, but will not be discussed in this paper. Observations discussed in the following paragraphs were made by the task group when comparing acceleration distributions computed using Section 12.10.3 for the buildings looked at in this study.

Calculation steps for each building to compute Section 12.10.3 accelerations are shown in Table A-2. It was observed that most buildings with a constant acceleration distribution over the height utilized a SFRS featuring Buckling Restrained Braced Frames (BRBF). It is common for buildings with BRBF systems to utilize concrete on metal deck diaphragms, meaning an R_s of 2.0 is likely to be utilized for these buildings. If an R_s of 2.0 is used, the C_{p0} will always be controlled by the $0.2S_{dsle}$ lower bound limit. The mode shape factor in Section 12.10.3, z_s , accounts for the distribution of inelasticity throughout the height of the building. Systems are given low values of z_s if inelastic deformations are well distributed throughout the height of the building, such as in a BRBF system. Systems are given high z_s values when inelastic deformation is more concentrated, such as in a concrete bearing wall system. BRBF systems are given both a low z_s value and high R value in ASCE 7, resulting in low mode shape factors in the 12.10.3 equations and low C_s factors, respectively. These two factors, coupled with a relatively low overstrength factor, Ω_o , result in a C_{pi} governed by the lower limit of C_{p0} . With small Γ_{m1} and Γ_{m2} factors, it is likely that Equation 12.10-7 will be governed by the lower limit of C_{pi} , leading to a constant acceleration distribution throughout the height of the building. Where buildings with BRBF systems utilize diaphragms with an R_s lower than 2.0, a ‘kick’ at the roof will likely be exhibited. However, the increase in acceleration at the roof will likely be minor, as shown for Building 48 in Figure C-1.

Except for Building 1 and 47, all buildings investigated by this Task Group that utilized SFRS featuring concrete shear walls exhibited an increase in acceleration at the roof when calculated using Section 12.10.3. Concrete shear wall systems are given a z_s value of 1.0 in Section 12.10.3, meaning that high gamma factors are likely to be computed, relative to lateral systems with lower z_s factors. Buildings 1 and 47 do not result in an increase in acceleration at the roof level because they contain a high number of stories (45 and 39, respectively) and a low S_{d1} value relative to the other buildings with concrete shear walls. This combination results in a low C_{pn} value that is governed by the lower bound C_{pi} .

The building data set investigated had only one building that utilized a SFRS featuring steel moment frames, Building 2. This building did not exhibit an increase in acceleration at the roof. This is due to the building being only three stories tall and the system having a z_s value of 0.7, thus low gamma factors. An additional story added to this building would result in an increase in acceleration at the roof.

Calculation steps for Section 13.3 are shown in Table A-3. A comparison between the accelerations at the roof level between Section 12.10.1, 12.10.3 and 13.3 is provided in Table A-4. At the roof level, the accelerations computed using Section 13.3 were never found to be governed by the lower and upper acceleration limits of Equation 13.3-2 and 13.3-3, respectively. These equations are intended to bound accelerations that could be computed when the nonstructural component characteristics described in RP2-3 are not set to unity. Where these minimum or maximum limits control, it wouldn't be appropriate to set the nonstructural components equal to unity and these should be considered in the comparison. While it is noted in the commentary of Section 13.3 that the acceleration distributions for nonstructural components would result in larger accelerations than those computed with Section 12.10.3, some buildings showed much larger discrepancies than others, such as Buildings 15 and 42. The provisions of Section 12.10.3 contain the z_s factor that accounts for inelastic deformation distribution throughout the height of the building, while the provisions of 13.3 do not explicitly account for that. To fairly compare acceleration distributions between Chapter 13 and Section 12.10.3, it is recommended by this task group that future studies set the R_s and z_s values to unity. Doing this will eliminate any additional reduction in accelerations from inelastic demand of the SFRS that is accounted for in the 12.10.3 equations but is not captured in the Chapter 13 equations. This comparison between Section 12.10.3 and Chapter 13 with R_s and z_s set to unity is presented in Table A-4 for each building alongside the comparison of these two methodologies using the appropriate R_s and z_s factor for each building. Plots in Figure C1 do not reflect the comparison between Section 12.10.3 and Chapter 13 with R_s and z_s set to unity.

Discrepancies between accelerations computed using Section 12.10.3 and 13.3 can be observed in Table A-4 where $C_s/(C_{pn}/R_s)$ values are less than or greater than 1.0. This applies for comparisons with and without R_s and z_s set to unity. Focusing on the comparison with R_s and z_s set to unity for the reasons mentioned above, buildings with BRBF systems and single-story buildings with SRCW systems had the largest discrepancies with relatively small and large C_s/C_{pn} ratios, respectively. Buildings with a C_s/C_{pn} ratio less than 1.0 generally had relatively longer periods and were in moderate to high seismic regions. Buildings with a C_s/C_{pn} ratio greater than 1.0 generally had shorter periods and were in moderate to high seismic regions. Buildings with a C_s/C_{pn} ratio close to 1.0 generally had an ordinary concentric braced frame system and were in low seismic regions. There are exceptions to these observations in this data set, but these observations can help inform future studies with a broader data set of buildings or archetypes.

RP2-5 Conclusions

The motive behind this paper was to provide a comparison between the acceleration distributions of Sections 12.10 and 13.3 over the height of real-world buildings to help understand and describe why there are differences between the methods. In addition to illustrating the level of correlation, an

objective of the study was to determine whether there are differences between the vertical distributions that are glaring or not reasonably explained, suggesting the need for correction to one or more distributions.

Prior to this study, it was expected by IT-08 that accelerations computed using Section 12.10.1 will generally be lower than those computed with Section 12.10.3. It was also expected that accelerations for nonstructural components using Section 13.3 will generally be larger than those given by Section 12.10.3. Section 12.10 equations generate an acceleration profile that is specific to a given building and rely on the characteristics of the SFRS and other attributes of the building previously described in RP2-2, allowing less conservatism to be inherent in the equations.

The equations in Chapter 13 are based on empirical data and nonlinear analyses of building archetypes and aim to approximate floor accelerations using a limited number of building characteristics compared to the equations of Section 12.10. Due to the limited specific building and SFRS characteristics used within the Section 13.3 equations, accelerations are expected to be more conservative than those given by Section 12.10 in most cases. IT-08 understands this limitation of the Section 13.3 equations and accepts this discrepancy. The plots shown in Figure C-1 illustrate these differences in acceleration profiles over the building height for the buildings investigated in this study.

For the data set used in this study, it has been concluded that the differences in vertical distribution are reasonable and appropriate given the intent and methodologies used. There is no need identified for correction to the vertical distributions. Because the data set used was somewhat limited and because diaphragm systems addressed by ASCE 7 Section 12.10.3 are expanding, additional study with a larger data set is recommended, as noted below.

RP2-6 Recommended Further Steps

It is recommended that a larger building data set that is more comprehensive and includes other SFRS and diaphragm system combinations be studied and the acceleration distributions between Section 12.10.3 and Chapter 13 be further compared. These comparisons as presented in this paper are not intended to assess the accuracy of each method, but to understand the relative accelerations between these methods and the impact code changes have on the acceleration distributions. With the inclusion of R_s values for different diaphragm systems making their way into the provisions of ASCE 7, more buildings can utilize the design equations of Section 12.10.3. The following is a short list of building characteristics that is recommended to be included in the larger data set:

- Other diaphragm systems not considered or given limited consideration in this study
 - Concrete-filled steel deck diaphragms
 - Cross-laminated timber (CLT) diaphragms

- Wood panel diaphragms
- Bare steel deck diaphragms
- Rod braced diaphragms
- Precast diaphragms
- Other SFRS not considered or given limited consideration in this study
 - Moment-resisting frame systems featuring steel special moment frames
 - Bearing wall systems featuring light-frame shear walls
 - Bearing wall systems featuring masonry wall systems
 - Building frame systems featuring other braced frame types
 - Bearing wall systems featuring cross-laminated timber shear wall systems
- Buildings with vertical and horizontal combinations of SFRS
- Buildings with vertical and horizontal combinations of diaphragm systems
- Buildings with site characteristics representative of a broader range of low to high seismicity
- Buildings with a broader range of heights ranging from low-rise, mid-rise, to high-rise

If the future data set of buildings that can be obtained is limited, it is recommended to create a series of archetype buildings that contain the characteristics above to better understand the acceleration distributions obtained with the 12.10.3 provisions for a broad range of building types.

RP2-6 Acknowledgements

The following were contributors to the IT8 subgroup discussions and this resource paper: Alex Wilson, Brian Kehoe, Ron LaPlante, Scott Breneman, Dave McCormick, S.K. Ghosh.

RP2-7 References

ASCE, 2017. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, 2016 Edition (ASCE7-16), American Society of Civil Engineers, Reston, VA.

ASCE, 2022. Minimum Design Loads and Associated Criteria for Buildings and Other Structures, 2022 Edition (ASCE7-22), American Society of Civil Engineers, Reston, VA.

NIST (National Institute of Standards and Technology). 2018. Recommendations for improved seismic performance of non-structural components. GCR 18-917-43. Gaithersburg, MD:NIST.

Table A-1 Seismic Characteristics for Investigated Buildings

Building Number	Stories	Lateral System	R	SDC	S_{Ds}	I_e	R_s	C_s	Ω_0	S_{D1}	T_a
1	39	SRCW	5	D	0.95	1	1.5	0.05	2.5	0.42	4.75
2	3	MF	8	D	0.76	1	2	0.06	3	0.43	1.55
3	15	SRCW	6	E	0.80	1	1.5	0.08	2.5	0.63	2.27
4	5	BRB	8	D	1.36	1.25	2	0.18	3	0.69	0.84
5	3	SRCW	5	D	1.24	1	1.5	0.25	2.5	0.63	0.31
6	12	BRB	8	D	0.93	1	2	0.06	3	0.42	0.88
7	3	SRCW	5	D	1.24	1	1.5	0.25	2.5	0.63	0.31
8	24	SRCW	5	C	0.47	1	1.5	0.02	2.5	0.19	3.64
9	6	BF	3.25	B	0.14	1.25	2	0.05	2	0.07	0.47
10	7	BF	3	B	0.18	1	1	0.02	2	0.06	1.14
11	1	SRCW	5	D	1.00	1	1.5	0.20	2.5	0.52	0.14
12	1	SRCW	5	D	1.00	1	1.5	0.20	2.5	0.52	0.14
13	1	SRCW	5	D	1.00	1	1.5	0.20	2.5	0.52	0.14
14	2	SRCW	5	D	1.00	1.25	1.5	0.25	2.5	0.52	0.22
15	3	LFSW	6.5	D	1.00	1.25	3	0.25	3	0.52	0.22
16	6	SRCW	5	D	0.59	1.25	1.5	0.15	2.5	0.37	0.52
17	2	SRCW	5	D	0.59	1	1.5	0.12	2.5	0.37	0.36
18	2	SRCW	5	D	0.59	1	1.5	0.12	2.5	0.37	0.27
19	1	SRCW	5	D	0.59	1	1.5	0.12	2.5	0.37	0.14
20	1	SRCW	5	D	0.59	1	1.5	0.12	2.5	0.37	0.14
21	2	SRCW	5	D	0.59	1	1.5	0.12	2.5	0.37	0.28
22	8	BF	3	B	0.12	1	1	0.03	2	0.09	1.52
23	22	SRCW	5	E	1.32	1.25	1.5	0.33	2.5	0.73	0.10
24	1	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.07
25	7	BF	3	B	0.12	1	1	0.03	2	0.09	1.34
26	1	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.04
27	1	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.03
28	2	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.03
29	1	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.03
30	1	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.05
31	1	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.06
32	1	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.08
33	2	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.06
34	3	SRCW	5	E	1.32	1	1.5	0.26	2.5	0.73	0.13
35	9	BRB	8	E	1.55	1	2	0.06	3	0.74	1.30
36	33	SRCW	5	D	1.09	1.25	2	0.06	2.5	0.65	1.97
37	16	BRB	8	D	1.07	1	2	0.05	3	0.47	1.15
38	29	SRCW	6	D	0.86	1	1.5	0.04	2.5	0.43	1.37
39	26	SRCW	6	D	0.86	1	1.5	0.04	2.5	0.43	2.23
40	18	SRCW	6	D	0.86	1	1.5	0.05	2.5	0.43	1.68
41	15	SRCW	6	D	0.86	1	1.5	0.06	2.5	0.43	1.68
42	6	SRCW	6	D	1.03	1	2	0.17	2.5	0.45	0.36
43	4	BRB	8	D	1.03	1	2	0.07	2.5	0.45	1.53
44	11	BRB	8	E	1.51	1.25	2	0.08	3	0.72	2.05
45	10	BRB	8	E	1.51	1.25	2	0.08	3	0.72	1.75
46	7	BRB	8	E	1.51	1.25	2	0.08	3	0.72	1.50
47	45	SRCW	6	D	0.88	1	2	0.04	2.5	0.44	5.20
48	4	BRB	8	D	0.95	1.25	1.5	0.15	3	0.60	0.73
49	31	ORCW	4	B	0.14	1.25	1.5	0.02	2.5	0.07	1.37
Notes:											
SRCW = Bearing Wall System With Special Reinforced Concrete Shear Walls											
ORCW = Bearing Wall System With Ordinary Reinforced Concrete Shear Walls											
BRB= Building Frame System With Steel Buckling Restrained Braced Frames											
BF = Building Frame System With Steel Ordinary Concentrically Braced Frames											
LFSW = Bearing Wall System With Light-Frame Wood Shear Walls											
MF = Moment-Resisting Frame System With Steel Special Moment Frames											

Table A-2 Section 12.10.3 Diaphragm Acceleration Calculations

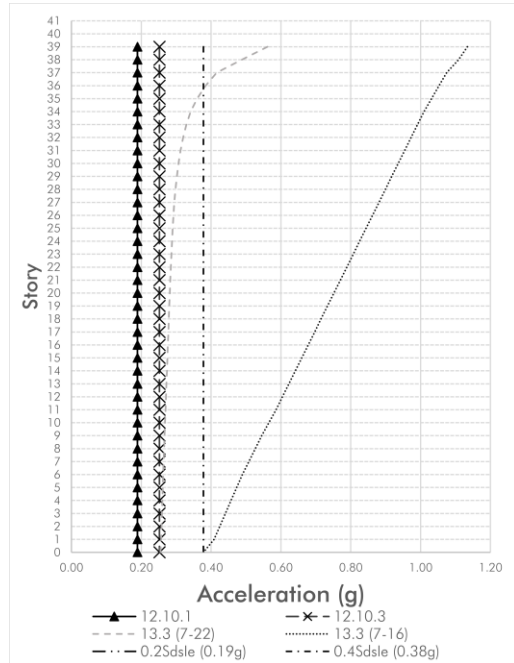
C _{pd} Calculations			C _{pi} Calculations				C _{pn} Calculations						Coefficient Summary					
Building Number	Eqn. 12.10-5	Eqn. 12.10-6	z _s	Eqn. 12.10-13	Eqn. 12.10-9	C _{pi}	Eqn. 12.10-14	Eqn. 12.10-10	Eqn. 12.10-11	Eqn. 12.10-12a, 12b	Cs2	C _{pn}	C _{pd} /R _s	C _{pi} /R _s	C _{pn} /R _s	C _{pn} /R _s > C _{pi} /R _s	Governing C _{pn} Eqn.	
1	0.19	0.38	1.00	1.49	0.17	0.38	0.85	5.76	0.95	0.37	0.37	0.38	0.25	0.25	0.25	No	12.10-12	
2	0.15	0.30	0.70	1.23	0.21	0.30	0.28	0.53	0.76	7.10	0.53	0.30	0.15	0.15	0.15	No	12.10-10	
3	0.16	0.32	1.00	1.47	0.27	0.32	0.78	2.00	0.80	1.49	0.80	0.70	0.21	0.21	0.46	Yes	12.10-11	
4	0.34	0.68	0.30	1.12	0.54	0.68	0.17	1.69	1.69	7.14	1.69	0.68	0.34	0.34	0.34	No	12.10-10	
5	0.25	0.50	1.00	1.33	0.75	0.75	0.40	0.87	1.24	10.53	0.87	0.90	0.33	0.50	0.60	Yes	12.10-10	
6	0.19	0.37	0.30	1.14	0.18	0.37	0.23	1.91	0.93	1.26	0.93	0.37	0.19	0.19	0.19	No	12.10-11	
7	0.25	0.50	1.00	1.33	0.75	0.75	0.40	0.87	1.24	10.53	0.87	0.90	0.33	0.50	0.60	Yes	12.10-10	
8	0.09	0.19	1.00	1.48	0.07	0.19	0.83	1.82	0.47	0.28	0.28	0.24	0.13	0.13	0.16	Yes	12.10-12	
9	0.04	0.07	1.00	1.42	0.14	0.14	0.63	0.20	0.18	0.58	0.18	0.19	0.04	0.07	0.09	Yes	12.10-11	
10	0.04	0.07	1.00	1.43	0.05	0.07	0.66	0.24	0.18	0.32	0.18	0.13	0.07	0.07	0.13	Yes	12.10-11	
11	0.20	0.40	1.00	1.00	0.45	0.45	0.00	0.40	1.00	0.00	0.00	0.50	0.27	0.30	0.33	Yes	12.10-12	
12	0.20	0.40	1.00	1.00	0.45	0.45	0.00	0.40	1.00	0.00	0.00	0.50	0.27	0.30	0.33	Yes	12.10-12	
13	0.20	0.40	1.00	1.00	0.45	0.45	0.00	0.40	1.00	0.00	0.00	0.50	0.27	0.30	0.33	Yes	12.10-12	
14	0.25	0.50	1.00	1.25	0.70	0.70	0.23	0.69	1.25	21.67	0.69	0.80	0.33	0.47	0.53	Yes	12.10-10	
15	0.25	0.50	1.00	1.33	0.90	0.90	0.40	0.88	1.25	10.83	0.88	1.06	0.25	0.30	0.35	Yes	12.10-10	
16	0.15	0.30	1.00	1.42	0.47	0.47	0.63	0.85	0.74	3.06	0.74	0.70	0.20	0.31	0.47	Yes	12.10-11	
17	0.12	0.24	1.00	1.25	0.33	0.33	0.23	0.33	0.59	12.23	0.33	0.38	0.16	0.22	0.25	Yes	12.10-10	
18	0.12	0.24	1.00	1.25	0.33	0.33	0.23	0.33	0.59	12.23	0.33	0.38	0.16	0.22	0.25	Yes	12.10-10	
19	0.12	0.24	1.00	1.00	0.27	0.27	0.00	0.24	0.59	0.00	0.00	0.30	0.16	0.18	0.20	Yes	12.10-12	
20	0.12	0.24	1.00	1.00	0.27	0.27	0.00	0.24	0.59	0.00	0.00	0.30	0.16	0.18	0.20	Yes	12.10-12	
21	0.12	0.24	1.00	1.25	0.33	0.33	0.23	0.33	0.59	12.23	0.33	0.38	0.16	0.22	0.25	Yes	12.10-10	
22	0.02	0.05	1.00	1.44	0.07	0.07	0.69	0.18	0.12	0.41	0.12	0.11	0.05	0.07	0.11	Yes	12.10-11	
23	0.33	0.66	1.00	1.48	1.10	1.10	0.82	5.86	1.65	1.44	1.44	1.70	0.44	0.73	1.13	Yes	12.10-12	
24	0.26	0.53	1.00	1.00	0.59	0.59	0.00	0.53	1.32	0.00	0.00	0.66	0.35	0.40	0.44	Yes	12.10-12	
25	0.02	0.05	1.00	1.43	0.07	0.07	0.66	0.16	0.12	0.48	0.12	0.11	0.05	0.07	0.11	Yes	12.10-11	
26	0.26	0.53	1.00	1.00	0.59	0.59	0.00	0.53	1.32	0.00	0.00	0.66	0.35	0.40	0.44	Yes	12.10-12	
27	0.26	0.53	1.00	1.00	0.59	0.59	0.00	0.53	1.32	0.00	0.00	0.66	0.35	0.40	0.44	Yes	12.10-12	
28	0.26	0.53	1.00	1.25	0.74	0.74	0.23	0.73	1.32	24.27	0.73	0.84	0.35	0.50	0.56	Yes	12.10-10	
29	0.26	0.53	1.00	1.00	0.59	0.59	0.00	0.53	1.32	0.00	0.00	0.66	0.35	0.40	0.44	Yes	12.10-12	
30	0.26	0.53	1.00	1.00	0.59	0.59	0.00	0.53	1.32	0.00	0.00	0.66	0.35	0.40	0.44	Yes	12.10-12	
31	0.26	0.53	1.00	1.00	0.59	0.59	0.00	0.53	1.32	0.00	0.00	0.66	0.35	0.40	0.44	Yes	12.10-12	
32	0.26	0.53	1.00	1.00	0.59	0.59	0.00	0.53	1.32	0.00	0.00	0.66	0.35	0.40	0.44	Yes	12.10-12	
33	0.26	0.53	1.00	1.25	0.74	0.74	0.23	0.73	1.32	24.27	0.73	0.84	0.35	0.50	0.56	Yes	12.10-10	
34	0.26	0.53	1.00	1.33	0.79	0.79	0.40	0.92	1.32	12.13	0.92	0.95	0.35	0.53	0.64	Yes	12.10-10	
35	0.31	0.62	0.30	1.13	0.18	0.62	0.21	2.48	1.55	3.10	1.55	0.62	0.31	0.31	0.31	No	12.10-11	
36	0.27	0.55	1.00	1.48	0.20	0.55	0.85	7.09	1.36	0.85	0.85	0.75	0.27	0.27	0.38	Yes	12.10-12	
37	0.21	0.43	0.30	1.14	0.14	0.43	0.24	2.84	1.07	1.03	1.03	0.43	0.21	0.21	0.21	No	12.10-12	
38	0.17	0.34	1.00	1.48	0.13	0.34	0.84	3.94	0.86	0.51	0.51	0.45	0.23	0.23	0.30	Yes	12.10-12	
39	0.17	0.34	1.00	1.48	0.13	0.34	0.83	3.56	0.86	0.58	0.58	0.50	0.23	0.23	0.33	Yes	12.10-12	
40	0.17	0.34	1.00	1.47	0.17	0.34	0.80	2.53	0.86	0.85	0.85	0.71	0.23	0.23	0.47	Yes	12.10-12	
41	0.17	0.34	1.00	1.47	0.18	0.34	0.78	2.14	0.86	1.03	0.86	0.70	0.23	0.23	0.47	Yes	12.10-11	
42	0.21	0.41	1.00	1.42	0.55	0.55	0.63	1.18	1.03	3.01	1.03	0.88	0.21	0.27	0.44	Yes	12.10-11	
43	0.21	0.41	0.30	1.11	0.17	0.41	0.15	0.87	1.03	5.02	0.87	0.41	0.21	0.21	0.21	No	12.10-10	
44	0.38	0.76	0.30	1.14	0.25	0.76	0.22	3.59	1.89	3.01	1.89	0.76	0.38	0.38	0.38	No	12.10-11	
45	0.38	0.76	0.30	1.14	0.25	0.76	0.22	3.31	1.89	3.35	1.89	0.76	0.38	0.38	0.38	No	12.10-11	
46	0.38	0.76	0.30	1.13	0.25	0.76	0.20	2.46	1.89	5.02	1.89	0.76	0.38	0.38	0.38	No	12.10-11	
47	0.18	0.35	1.00	1.49	0.13	0.35	0.86	6.14	0.88	0.33	0.33	0.35	0.18	0.18	0.18	No	12.10-12	
48	0.24	0.47	0.30	1.11	0.44	0.47	0.15	1.01	1.18	8.33	1.01	0.52	0.32	0.32	0.34	Yes	12.10-10	
49	0.04	0.07	1.00	1.48	0.05	0.07	0.84	0.87	0.18	0.10	0.10	0.10	0.05	0.05	0.07	Yes	12.10-12	

Table A-3 Section 13.3 Diaphragm Acceleration Calculations at Roof Level

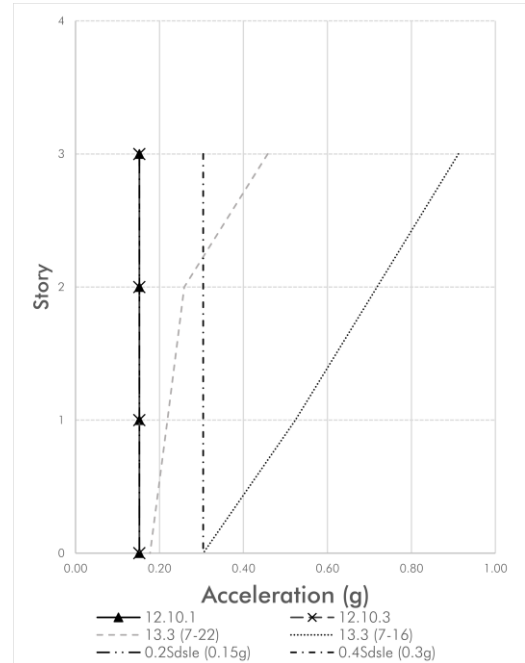
Building and Component Characteristics									C _s Calculations At Roof							
Building Number	S _{ds}	I _p	T _a	R	I _e	Ω	C _{AR}	R _{po}	F _{p,max}	F _{p,min}	R _μ	a ₁	a ₂	H _f	C _s	
1	0.95	1	4.75	5	1	2.5	1	1	1.51	0.28	1.48	0.21	0.99	2.20	0.56	
2	0.76	1	1.55	8	1	3	1	1	1.22	0.23	1.71	0.65	0.93	2.58	0.46	
3	0.80	1	2.27	6	1	2.5	1	1	1.28	0.24	1.62	0.44	0.97	2.41	0.47	
4	1.36	1	0.84	8	1.25	3	1	1	2.71	0.51	1.53	1.19	0.77	2.96	1.05	
5	1.24	1	0.31	5	1	2.5	1	1	1.99	0.37	1.48	2.50	0.00	3.50	1.17	
6	0.93	1	0.88	8	1	3	1	1	1.49	0.28	1.71	1.14	0.79	2.93	0.64	
7	1.24	1	0.31	5	1	2.5	1	1	1.99	0.37	1.48	2.50	0.00	3.50	1.17	
8	0.47	1	3.64	5	1	2.5	1	1	0.76	0.14	1.48	0.27	0.99	2.26	0.29	
9	0.14	1	0.47	3.25	1.25	2	1	1	0.28	0.05	1.30	2.13	0.28	3.40	0.15	
10	0.18	1	1.14	3	1	2	1	1	0.29	0.05	1.30	0.88	0.88	2.75	0.15	
11	1.00	1	0.14	5	1	2.5	1	1	1.60	0.30	1.48	2.50	0.00	3.50	0.94	
12	1.00	1	0.14	5	1	2.5	1	1	1.60	0.30	1.48	2.50	0.00	3.50	0.94	
13	1.00	1	0.14	5	1	2.5	1	1	1.60	0.30	1.48	2.50	0.00	3.50	0.94	
14	1.00	1	0.22	5	1.25	2.5	1	1	2.00	0.38	1.33	2.50	0.00	3.50	1.06	
15	1.00	1	0.22	6.5	1.25	3	1	1	2.00	0.38	1.38	2.50	0.00	3.50	1.01	
16	0.59	1	0.52	5	1.25	2.5	1	1	1.18	0.22	1.33	1.92	0.41	3.33	0.59	
17	0.59	1	0.36	5	1	2.5	1	1	0.95	0.18	1.48	2.50	0.00	3.50	0.56	
18	0.59	1	0.27	5	1	2.5	1	1	0.95	0.18	1.48	2.50	0.00	3.50	0.56	
19	0.59	1	0.14	5	1	2.5	1	1	0.95	0.18	1.48	2.50	0.00	3.50	0.56	
20	0.59	1	0.14	5	1	2.5	1	1	0.95	0.18	1.48	2.50	0.00	3.50	0.56	
21	0.59	1	0.28	5	1	2.5	1	1	0.95	0.18	1.48	2.50	0.00	3.50	0.56	
22	0.12	1	1.52	3	1	2	1	1	0.20	0.04	1.30	0.66	0.93	2.59	0.10	
23	1.32	1	0.1	5	1.25	2.5	1	1	2.64	0.50	1.33	2.50	0.00	3.50	1.39	
24	1.32	1	0.07	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
25	0.12	1	1.34	3	1	2	1	1	0.20	0.04	1.30	0.75	0.91	2.66	0.10	
26	1.32	1	0.04	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
27	1.32	1	0.03	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
28	1.32	1	0.03	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
29	1.32	1	0.03	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
30	1.32	1	0.05	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
31	1.32	1	0.06	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
32	1.32	1	0.08	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
33	1.32	1	0.06	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
34	1.32	1	0.13	5	1	2.5	1	1	2.11	0.40	1.48	2.50	0.00	3.50	1.25	
35	1.55	1	1.3	8	1	3	1	1	2.48	0.46	1.71	0.77	0.91	2.67	0.97	
36	1.09	1	1.97	5	1.25	2.5	1	1	2.18	0.41	1.33	0.51	0.96	2.47	0.81	
37	1.07	1	1.15	8	1	3	1	1	1.71	0.32	1.71	0.87	0.88	2.75	0.69	
38	0.86	1	1.37	6	1	2.5	1	1	1.37	0.26	1.62	0.73	0.91	2.64	0.56	
39	0.86	1	2.23	6	1	2.5	1	1	1.37	0.26	1.62	0.45	0.97	2.42	0.51	
40	0.86	1	1.68	6	1	2.5	1	1	1.37	0.26	1.62	0.60	0.94	2.54	0.54	
41	0.86	1	1.68	6	1	2.5	1	1	1.37	0.26	1.62	0.60	0.94	2.54	0.54	
42	1.03	1	0.36	6	1	2.5	1	1	1.65	0.31	1.62	2.50	0.00	3.50	0.89	
43	1.03	1	1.53	8	1	2.5	1	1	1.65	0.31	1.88	0.65	0.93	2.59	0.57	
44	1.51	1	2.05	8	1.25	3	1	1	3.02	0.57	1.53	0.49	0.96	2.45	0.97	
45	1.51	1	1.75	8	1.25	3	1	1	3.02	0.57	1.53	0.57	0.95	2.52	0.99	
46	1.51	1	1.5	8	1.25	3	1	1	3.02	0.57	1.53	0.67	0.93	2.60	1.02	
47	0.88	1	5.2	6	1	2.5	1	1	1.40	0.26	1.62	0.19	0.99	2.19	0.47	
48	0.95	1	0.73	8	1.25	3	1	1	1.89	0.36	1.53	1.37	0.70	3.07	0.76	
49	0.14	1	1.37	4	1.25	2.5	1	1	0.28	0.05	1.30	0.73	0.91	2.64	0.12	
Notes:																
C _s = F _p /W _p , peak acceleration of nonstructural component at roof																

Table A-4 Comparison Between Sections 12.10.1, 12.10.3 and 13.3 Accelerations at Roof Level

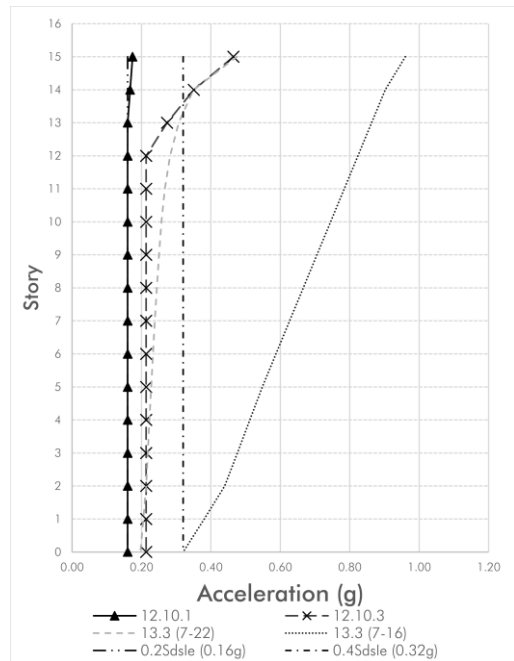
Building and Seismic Parameters								Comparison							
Building Number	R _s	Stories (N)	SFRS	S _{ds}	T _a	R	z _s	F _p /W _i	C _{pn} /R _s	C _{pn} (R _s and z _s =1.0)	C _s	(C _{pn} /R _s)/(F _p /W _i)	C _s /(C _{pn} /R _s)	C _{pn} /(F _p /W _i) w/ R _s and z _s =1.0	C _s /C _{pn} w/ R _s and z _s =1.0
22	1	8	BF	0.12	1.52	3	1	0.05	0.11	0.11	0.10	2.31	0.86	2.31	0.86
25	1	7	BF	0.12	1.34	3	1	0.05	0.11	0.11	0.10	2.25	0.91	2.25	0.91
9	2	6	BF	0.14	0.47	3.25	1	0.07	0.09	0.19	0.15	1.34	1.57	2.67	0.78
10	1	7	BF	0.18	1.14	3	1	0.06	0.13	0.13	0.15	2.43	1.15	2.43	1.15
6	2	12	BRB	0.93	0.88	8	0.3	0.19	0.19	0.75	0.64	1.00	3.42	4.03	0.85
48	1.5	4	BRB	0.95	0.73	8	0.3	0.24	0.34	0.80	0.76	1.46	2.20	3.36	0.95
43	2	4	BRB	1.03	1.53	8	0.3	0.21	0.21	0.50	0.57	1.00	2.76	2.43	1.14
37	2	16	BRB	1.07	1.15	8	0.3	0.21	0.21	0.84	0.69	1.00	3.21	3.94	0.81
4	2	5	BRB	1.36	0.84	8	0.3	0.34	0.34	1.23	1.05	1.00	3.10	3.62	0.85
44	2	11	BRB	1.51	2.05	8	0.3	0.38	0.38	1.45	0.97	1.00	2.56	3.84	0.67
45	2	10	BRB	1.51	1.75	8	0.3	0.38	0.38	1.42	0.99	1.00	2.63	3.77	0.70
46	2	7	BRB	1.51	1.5	8	0.3	0.38	0.38	1.30	1.02	1.00	2.71	3.44	0.79
35	2	9	BRB	1.55	1.3	8	0.3	0.31	0.31	1.13	0.97	1.00	3.12	3.65	0.85
15	3	3	LFSW	1.00	0.22	6.5	1	0.25	0.35	1.06	1.01	1.40	2.87	4.20	0.96
2	2	3	MF	0.76	1.55	8	0.7	0.15	0.15	0.33	0.46	1.00	3.01	2.15	1.40
49	1.5	31	ORCW	0.14	1.37	4	1	0.07	0.07	0.10	0.12	0.94	1.73	1.41	1.15
8	1.5	24	SRCW	0.47	3.64	5	1	0.09	0.16	0.24	0.29	1.72	1.78	2.58	1.18
16	1.5	6	SRCW	0.59	0.52	5	1	0.23	0.47	0.70	0.59	2.07	1.27	3.10	0.85
17	1.5	2	SRCW	0.59	0.36	5	1	0.18	0.25	0.38	0.56	1.40	2.23	2.10	1.48
18	1.5	2	SRCW	0.59	0.27	5	1	0.24	0.25	0.38	0.56	1.06	2.23	1.59	1.48
19	1.5	1	SRCW	0.59	0.14	5	1	0.15	0.20	0.30	0.56	1.28	2.84	1.92	1.89
20	1.5	1	SRCW	0.59	0.14	5	1	0.15	0.20	0.30	0.56	1.32	2.84	1.98	1.89
21	1.5	2	SRCW	0.59	0.28	5	1	0.18	0.25	0.38	0.56	1.43	2.23	2.14	1.48
3	1.5	15	SRCW	0.80	2.27	6	1	0.17	0.46	0.70	0.47	2.66	1.02	4.00	0.68
38	1.5	29	SRCW	0.86	1.37	6	1	0.17	0.30	0.45	0.56	1.77	1.84	2.65	1.23
39	1.5	26	SRCW	0.86	2.23	6	1	0.17	0.33	0.50	0.51	1.95	1.53	2.92	1.02
40	1.5	18	SRCW	0.86	1.68	6	1	0.17	0.47	0.71	0.54	2.75	1.14	4.12	0.76
41	1.5	15	SRCW	0.86	1.68	6	1	0.17	0.47	0.70	0.54	2.73	1.15	4.09	0.76
47	2	45	SRCW	0.88	5.2	6	1	0.18	0.18	0.35	0.47	1.00	2.69	2.00	1.35
1	1.5	39	SRCW	0.95	4.75	5	1	0.19	0.25	0.38	0.56	1.33	2.23	2.00	1.49
11	1.5	1	SRCW	1.00	0.14	5	1	0.20	0.33	0.50	0.94	1.67	2.83	2.50	1.89
12	1.5	1	SRCW	1.00	0.14	5	1	0.20	0.33	0.50	0.94	1.67	2.83	2.50	1.89
13	1.5	1	SRCW	1.00	0.14	5	1	0.20	0.33	0.50	0.94	1.67	2.83	2.50	1.89
14	1.5	2	SRCW	1.00	0.22	5	1	0.34	0.53	0.80	1.06	1.56	1.99	2.34	1.33
42	2	6	SRCW	1.03	0.36	6	1	0.30	0.44	0.88	0.89	1.48	2.01	2.96	1.00
36	2	33	SRCW	1.09	1.97	5	1	0.27	0.38	0.75	0.81	1.38	2.15	2.76	1.08
5	1.5	3	SRCW	1.24	0.31	5	1	0.29	0.60	0.90	1.17	2.08	1.96	3.13	1.30
7	1.5	3	SRCW	1.24	0.31	5	1	0.28	0.60	0.90	1.17	2.11	1.96	3.16	1.30
23	1.5	22	SRCW	1.32	0.1	5	1	0.65	1.13	1.70	1.39	1.74	1.23	2.60	0.82
24	1.5	1	SRCW	1.32	0.07	5	1	0.26	0.44	0.66	1.25	1.67	2.83	2.50	1.89
26	1.5	1	SRCW	1.32	0.04	5	1	0.26	0.44	0.66	1.25	1.67	2.83	2.50	1.89
27	1.5	1	SRCW	1.32	0.03	5	1	0.26	0.44	0.66	1.25	1.67	2.83	2.50	1.89
28	1.5	2	SRCW	1.32	0.03	5	1	0.33	0.56	0.84	1.25	1.69	2.22	2.54	1.48
29	1.5	1	SRCW	1.32	0.03	5	1	0.26	0.44	0.66	1.25	1.67	2.83	2.50	1.89
30	1.5	1	SRCW	1.32	0.05	5	1	0.26	0.44	0.66	1.25	1.67	2.83	2.50	1.89
31	1.5	1	SRCW	1.32	0.06	5	1	0.26	0.44	0.66	1.25	1.67	2.83	2.50	1.89
32	1.5	1	SRCW	1.32	0.08	5	1	0.29	0.44	0.66	1.25	1.51	2.83	2.26	1.89
33	1.5	2	SRCW	1.32	0.06	5	1	0.38	0.56	0.84	1.25	1.48	2.22	2.22	1.48
34	1.5	3	SRCW	1.32	0.13	5	1	0.40	0.64	0.95	1.25	1.61	1.96	2.41	1.31
Notes:															
SRCW = Bearing Wall System With Special Reinforced Concrete Shear Walls															
ORCW = Bearing Wall System With Ordinary Reinforced Concrete Shear Walls															
BRB= Building Frame System With Steel Buckling Restrained Braced Frames															
BF = Building Frame System With Steel Ordinary Concentrically Braced Frames															
LFSW = Bearing Wall System With Light-Frame Wood Shear Walls															
MF = Moment-Resisting Frame System With Steel Special Moment Frames															
Cs = Fp/Wp, peak acceleration of nonstructural component at roof															



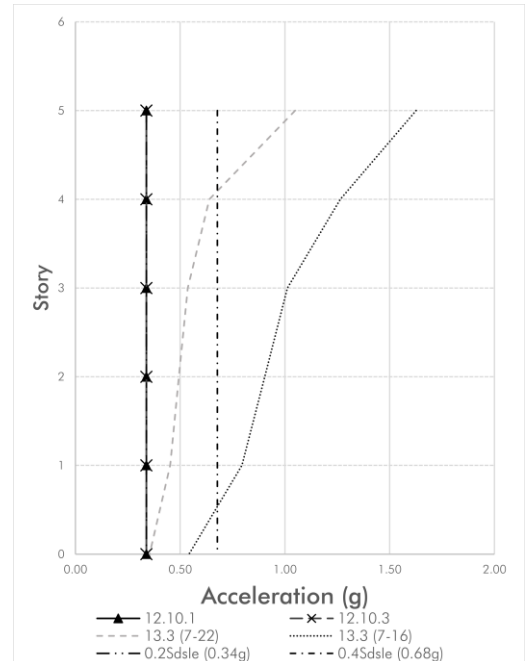
Building 1



Building 2



Building 3



Building 4

Figure C-1: Comparison of Acceleration Distribution Between Section 12.10.1, 12.10.3 and 13.3.

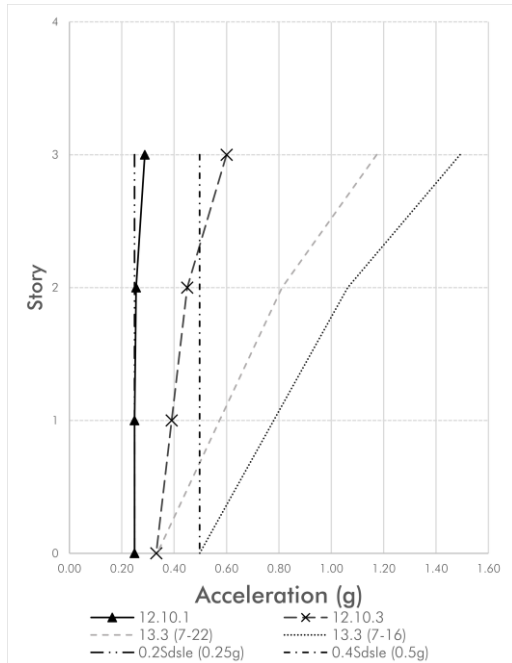
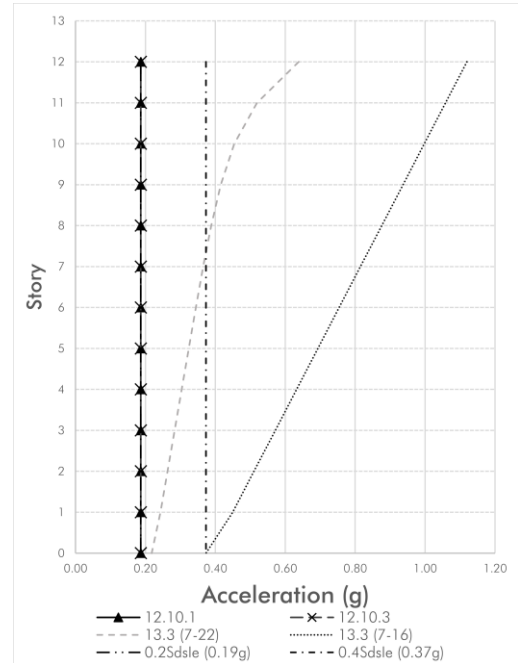
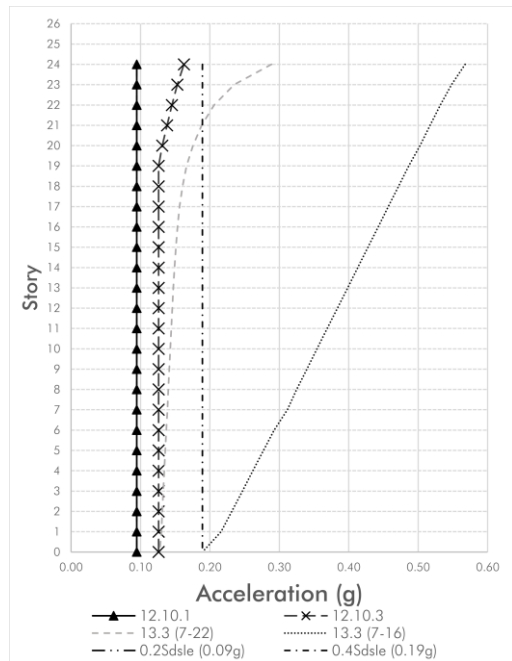
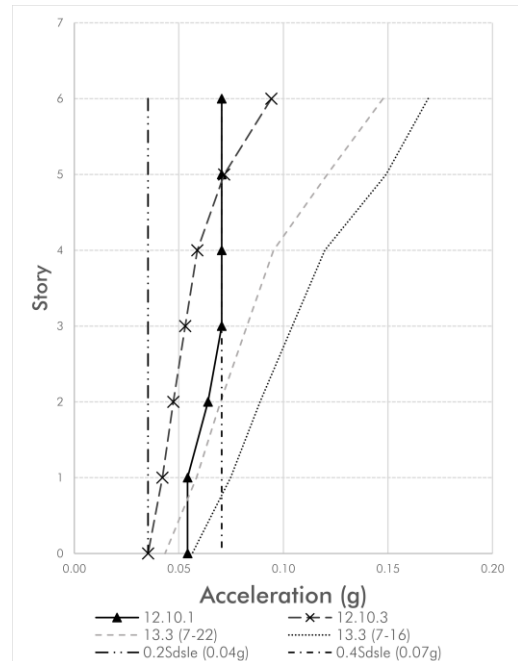
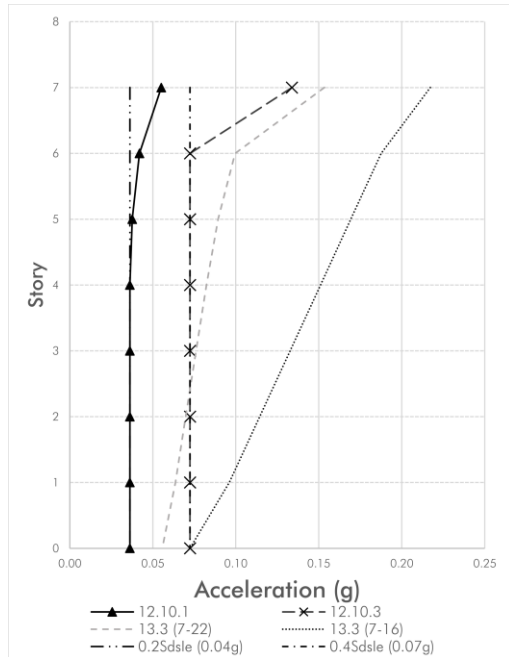
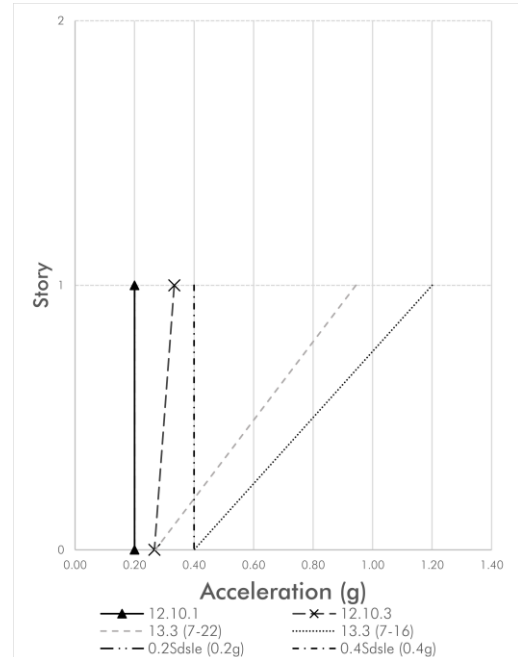


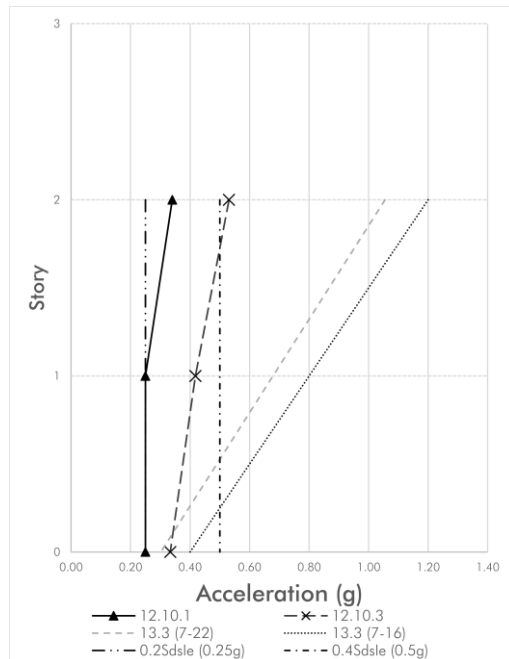
Figure C-1 cont.: Comparison of Acceleration Distribution Between Section 12.10.1, 12.10.3 and 13.3.



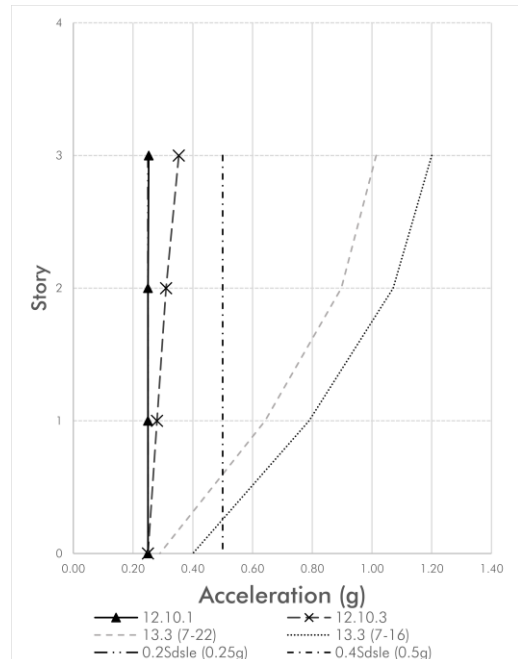
Building 10



Building 11



Building 14



Building 15

Figure C-1 cont.: Comparison of Acceleration Distribution Between Section 12.10.1, 12.10.3 and 13.3.

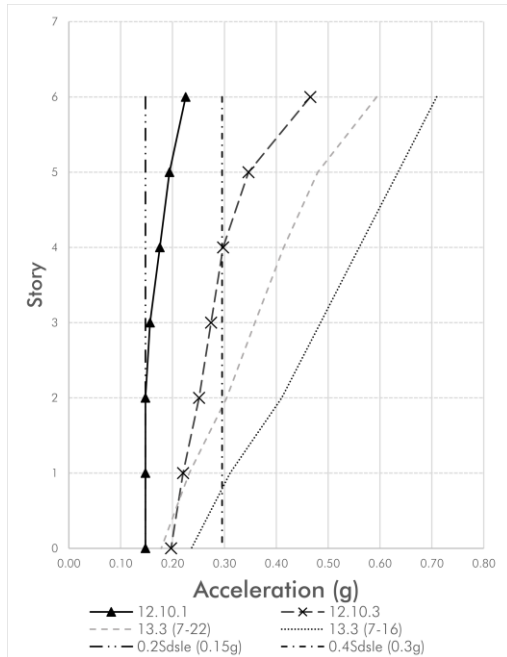
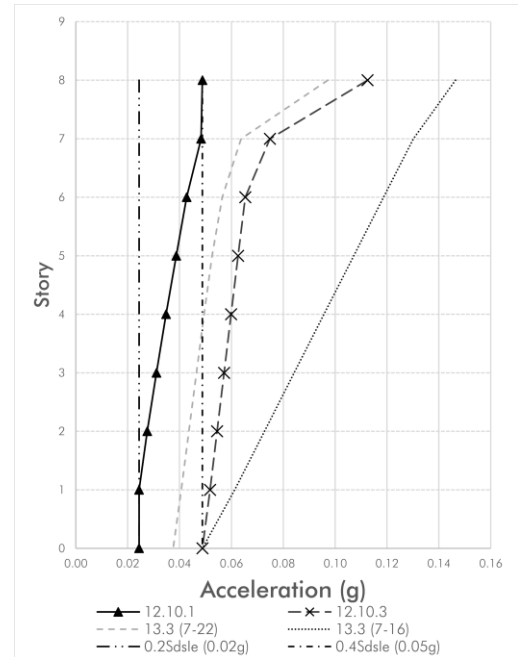
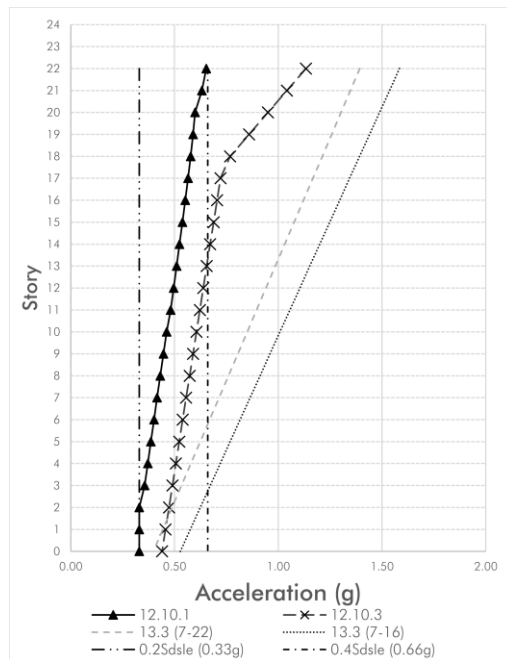
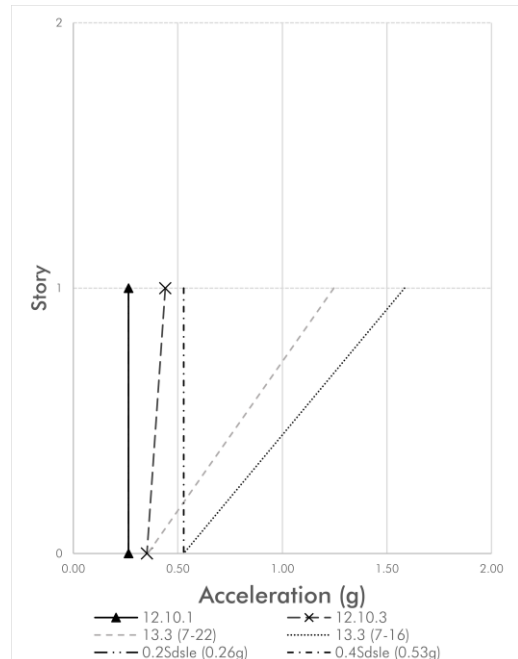


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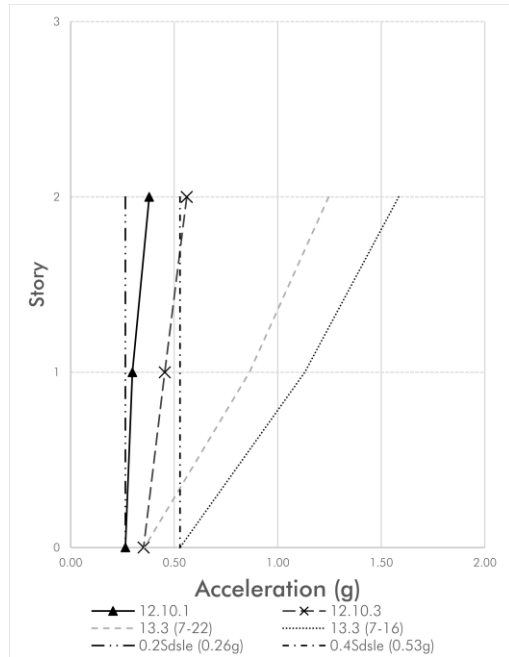
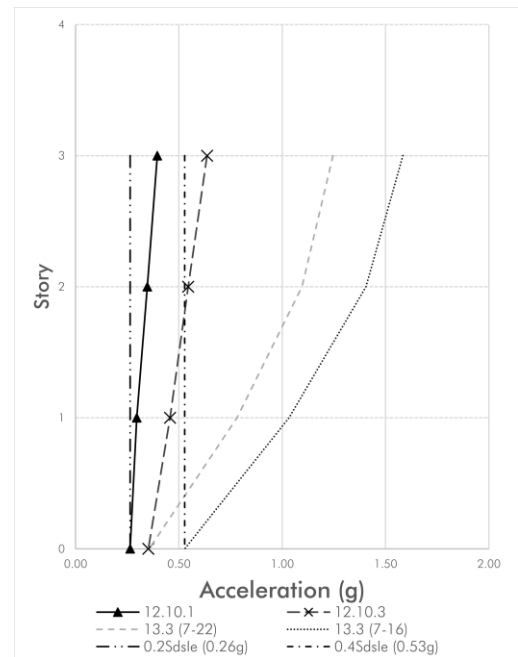
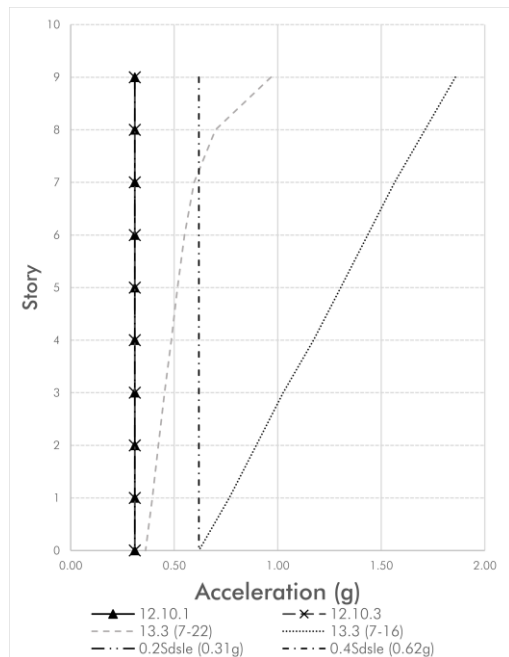
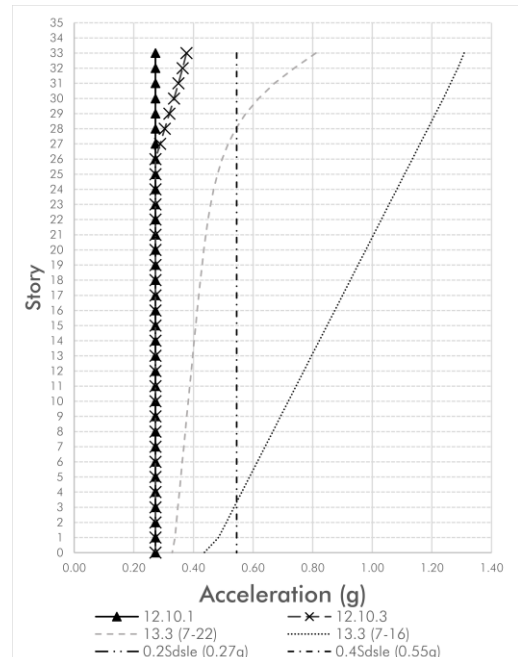


Figure C-1 cont.: Comparison of Acceleration Distribution Between Section 12.10.1, 12.10.3 and 13.3.

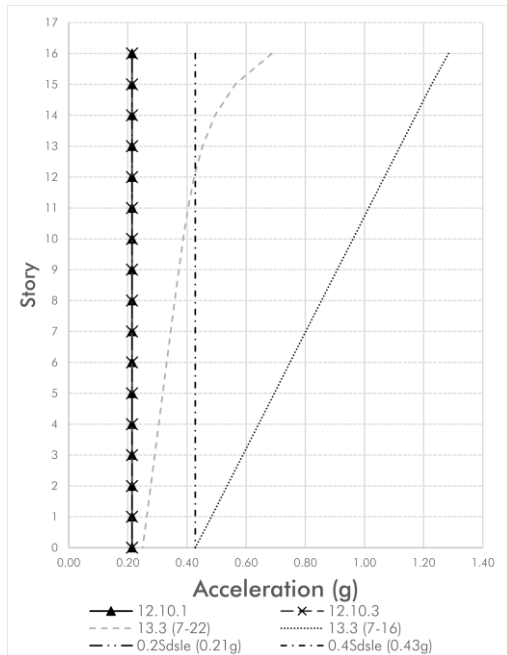
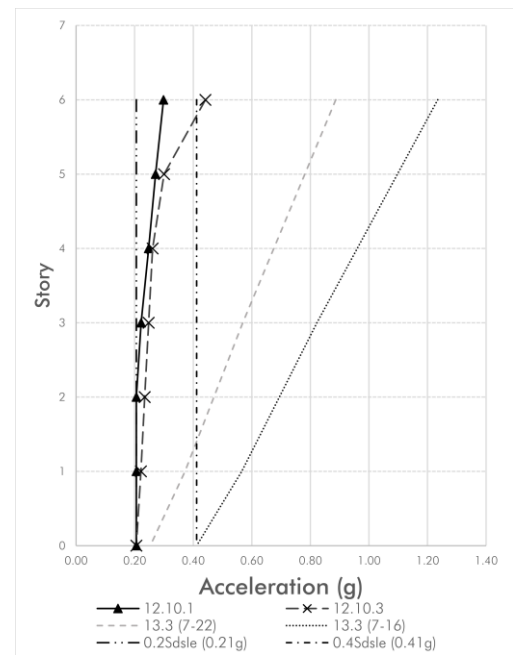
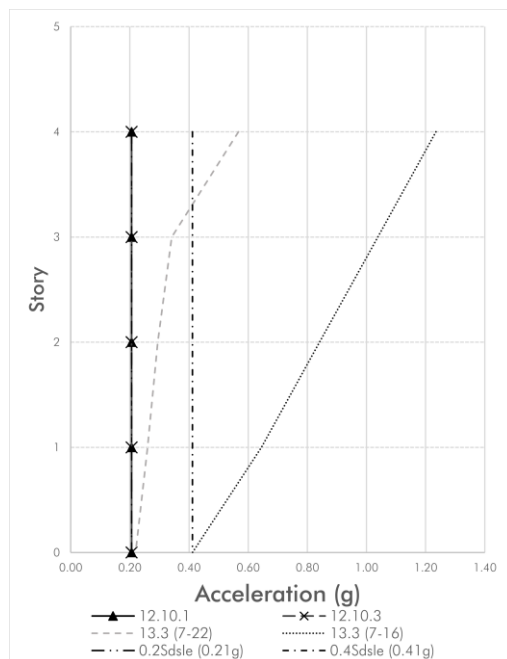
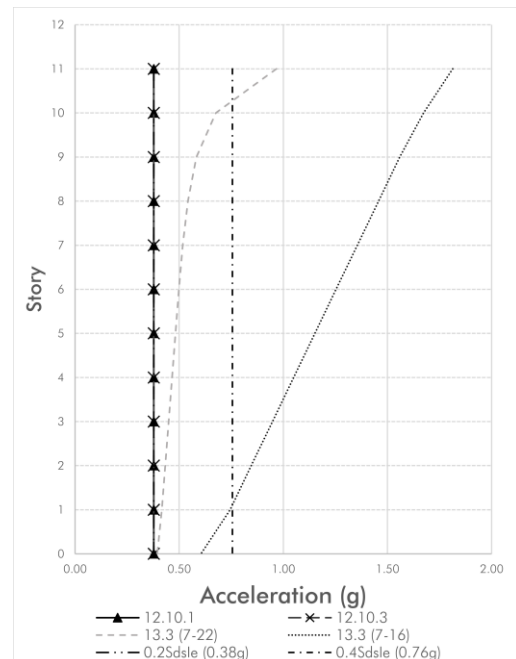


Figure C-1 cont.: Comparison of Acceleration Distribution Between Section 12.10.1, 12.10.3 and 13.3.

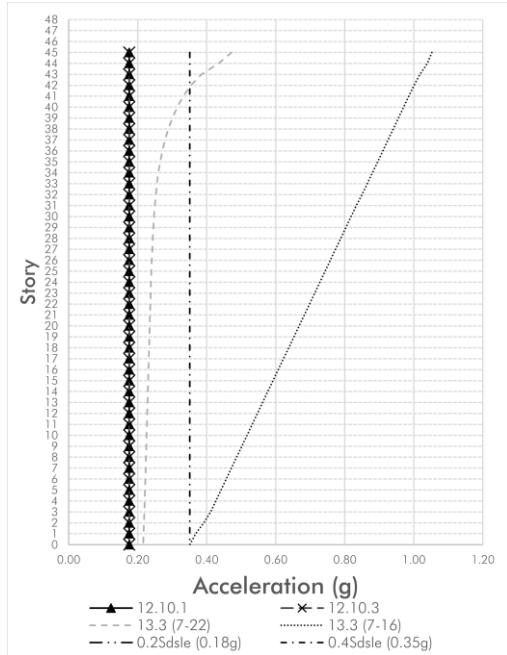
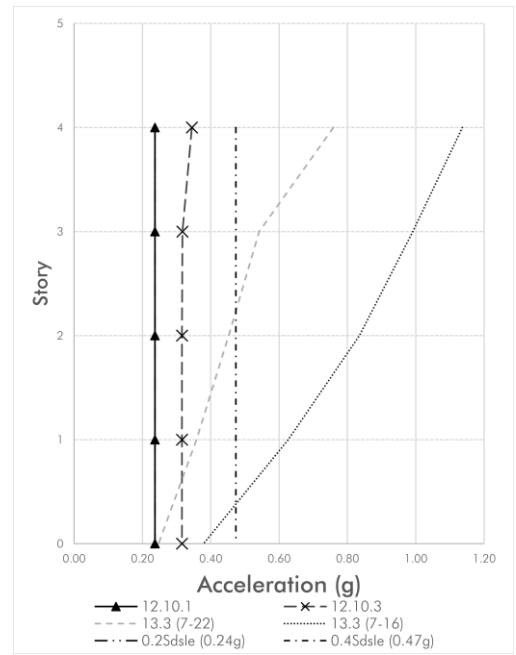
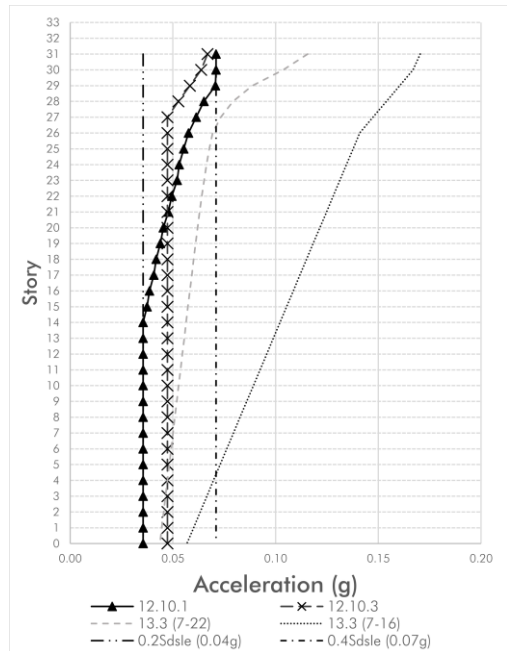


Figure C-1 cont.: Comparison of Acceleration Distribution Between Section 12.10.1, 12.10.3 and 13.3.

RESOURCE PAPER 3 DESIGN GROUND MOTIONS OF CHAPTER 22

RP3-1.0 Introduction

This resource paper has been prepared at the request of the Building Seismic Safety Council (BSSC) Provisions Update Committee (PUC) as part of the development of the 2026 National Earthquake Hazard Reduction Program Recommended Seismic Provisions for New Buildings and Other Structures (NEHRP Provisions). The objectives of this resource paper are to:

- 1) provide a recent history of seismic hazard characterization in the U.S. and the development of seismic design parameters for use in the NEHRP Provisions, and
- 2) provide an expanded illustration of the effects on seismic design parameters, seismic response spectra, and Seismic Design Category assignments, as a result of recent changes in seismic hazard characterization and NEHRP representation of seismic design parameters.

The NEHRP Provisions have been developed and updated by the BSSC PUC for the past several decades as an important resource for the seismic safety of the Nation's built environment (BSSC 1985, 1988, 1991, 1994, 1997, 2000, 2003, 2009, 2015, 2020). The National Seismic Hazard Models (NSHMs) are developed and updated by the U.S. Geological Survey (USGS) for the United States and its territories (Algermissen Perkins, 1976; Algermissen et al, 1990; Frankel et al. 1996, 2002; Petersen et al. 2008, 2014, 2020, 2024) and have been the scientific foundation for the seismic design ground motions of the NEHRP Provisions. Although there has always been a collaboration between the BSSC PUC engineers and USGS scientists, the derivation of the seismic design ground motions from the NSHMs has been defined by the BSSC PUC engineers since the inception of the NEHRP Provisions in 1985, whereas the NSHMs themselves have been updated by the USGS based on the best available science and data. About every ten years, through Projects 97 (documented in Appendix B of commentary to 1997 NEHRP, BSSC 1997), 07 (Kircher et al. 2010), and 17 (BSSC 2019), joint committees of BSSC-organized engineers and USGS researchers were formed to discuss major improvements to the procedures for development of the next generation of seismic design ground motions. These projects were concluded prior to development of the 1997, 2009, and 2020 NEHRP Provisions (BSSC 1997, 2009, 2020). The interactions between BSSC and USGS have also been important in the development of the NSHMs, which have not only been influenced by the best available scientific modeling and seismic data, but also by the needs and reviews of engineering users (e.g., the need for more accurate representation of spectral shapes and site amplifications in the 2020 NEHRP Provisions).

Between 1985 and 1991 (the first three editions of NEHRP Provisions, BSSC 1985, 1988, and 1991), the seismic design ground motions were Design Earthquake (DE) ground motions specified in terms of peak ground acceleration, PGA (or zero-period spectral acceleration), and 1.0-sec spectral acceleration (SA) for a reference rock site. The DE acceleration values were approximately based on earthquake ground motions with a 475-year return period based on probabilistic seismic hazard analysis (PSHA) at a reference site condition. Site amplification factors were approximated deterministically, outside of the PSHA, and applied for soil conditions other than the reference rock site, independent of acceleration levels. The DE rock acceleration values were capped at 0.4 g (down from the 0.6 g maximum contour in the Algermissen and Perkins, 1976, maps); the rationale for doing so can be traced back to the commentary of the ATC 3-06 Tentative Provisions (ATC 1978), which cites “doubt among many professionals” that large earthquakes could actually cause widespread accelerations as large as 0.6 g, that such accelerations could actually be transmitted through dense soils, and that designing for accelerations greater than 0.4 g would produce diminishing returns in earthquake-resistant construction. In the 1994 NEHRP Provisions (BSSC 1994), new site classes and site amplification factors, F_a for short 0.2-sec SA and F_v for long 1.0-sec SA, were introduced based on the average shear-wave velocity at the site in the top 30 meters of the soil, V_{s30} . These new site amplification factors varied with acceleration levels. During this time period, seismic design regulations in the U.S. did not directly reference the NEHRP Provisions. However, the Uniform Building Code (UBC), which was the primary model code in the western U.S. to promote public safety through standardized minimum design requirements, in its final edition dated 1997 (ICBO 1997), adopted seismic design criteria developed by the Seismology Committee of the Structural Engineers Association of California for high seismic areas. These criteria defined design ground motion parameters in terms of a Design Basis Earthquake (DBE), which was similar to the NEHRP Provisions, with maps defining the DBE PGA on a reference rock site based on a 475-year return period and capped at 0.4 g (Seismic Zone 4). In the 1997 UBC, site amplification factors similar to F_a and F_v were also introduced, along with adjustment factors (N_a and N_v) to ground motions for locations near active faults at Seismic Zone 4 locations in California and Nevada, based on the active fault near-source zones defined in Petersen et al. (2000). After the last edition of the UBC up to this day, most seismic design regulations in the U.S. follow the International Building Code (IBC) with its first edition published in 2000 (ICC 2000), with its seismic regulations based on the 1997 NEHRP Provisions (BSSC 1997).

In the 1997 NEHRP Provisions (BSSC 1997), the procedures to calculate the seismic design ground motions changed dramatically based on the recommendations from the BSSC PUC Project 97 (documented in Appendix B of commentary to 1997 NEHRP, BSSC 1997), which introduced the Maximum Considered Earthquake (MCE) ground motions corresponding to a 2,475-year return period and defined seismic design ground motions probabilistically based on the USGS 1996 NSHM (Frankel et al. 1996). These procedures also added deterministic caps in regions near active faults. The DE ground motions were maintained as the primary design ground motion for most structures and were defined as two thirds of the MCE. After the development of NEHRP Provisions, the seismic design maps were adopted into Chapter 9 of the 1998 and 2002 American Society of Civil Engineers (ASCE) Standard for Minimum Design Loads for Buildings and Other Structures, ASCE 7-98 (ASCE 2000) and ASCE 7-02 (ASCE 2003). The 2003 NEHRP (BSSC 2003) retained these procedures but updated the USGS hazard model to the 2002 NSHM (Frankel et al. 2002). In 2005, these NEHRP Provisions were adopted into the ASCE 7-05 Standard (ASCE 2005), which at the same time expanded the chapters of ASCE 7 and specified the procedures that determined the MCE and DE in its Chapters 11 and 21.

In recent NEHRP Provisions (i.e., BSSC 2009, 2015, and 2020) and the following ASCE 7 Standards (i.e., ASCE 7-10, 7-16, and 7-22; ASCE 2013, 2017, and 2021), Chapter 21 presents the site-specific ground motion procedures that were developed and updated by BSSC PUC engineers since 1997. Chapter 22 presents the seismic design ground motions in the form of maps or geodatabases that are developed and updated by USGS researchers, using the site-specific procedures of Chapter 21, along with the latest NSHMs approved by the BSSC PUC. However, there are some exceptions, such as using site coefficients based on Chapter 11 in the 2015 NEHRP, as discussed later.) Although engineers are allowed to perform their own site-specific analyses, ground motions in Chapter 22 are permitted to be used in design and provide a basis for achieving the minimum seismic safety. The 2003 IBC (ICC 2003) and later editions have adopted the seismic criteria from ASCE 7 to this day.

Values of seismic design ground motions have changed and evolved every cycle either because 1) the USGS estimate of seismic hazard has improved in certain regions due to more accurate scientific models or new data, or 2) the NEHRP definitions of seismic design parameters have changed due to new directions provided by the PUC, or in most cases, 3) both components have changed. Justifications and explanations of changes at select sites were presented in the commentary of Chapter 22 since the 2009 NEHRP Provisions. During the development of the latest 2026 NEHRP Provisions, unexpected seismic design parameter results from the 2020 NEHRP Provisions were identified and adjustments to the site-specific procedures of Chapter 21 were developed as a result. At the same time, major changes were introduced once again to the USGS NSHMs used for Chapter 22. As a result, the PUC requested a document that summarizes the changes over the years for a larger, more comprehensive set of locations and site classes and provides more information on justifications than is practical in the commentary of Chapter 22, to facilitate transparency and the ability for more rigorous review processes in future cycles. It was suggested to provide data not only for more sites, but for more periods and site classes due to design ground motions now being computed for 22 periods (including PGA) and eight site classes, compared to the previous three periods and one reference site class until the 2015 NEHRP Provisions. This resource paper provides that extra requested information.

In the following Section 2, we first provide a summary of the USGS NSHMs used to develop the seismic design ground motions of Chapter 22 in past cycles of the NEHRP Provisions. (The current 2023 NSHM used in the 2026 NEHRP Provisions is summarized in its commentary of Chapter 22). This is followed by a history of the evolving definitions of seismic design ground motions in the site-specific procedures of Chapter 21. Section 3 introduces a representative set of example sites, referred to hereafter as test sites, that are of interest either because they have a large population, interesting seismological or geological features, or locations corresponding to important infrastructure or facilities. The seismic design parameters defined in Chapter 21 (short and long period S_A , S_S and S_1 for the reference site class and S_{MS} and S_{M1} for default site conditions) are provided for these locations for the 2026 NEHRP Provisions and compared to their values from the four past cycles (ASCE 7-05, 7-10, 7-16, and 7-22). The locations are grouped into regions to better understand reasons for changes in each region. Whereas only S_S and S_1 data for the reference site class and S_{MS} and S_{M1} data for the default site conditions are shown in this paper, additional data are provided in an electronic supplement for every site class. Section 4 explores the spectral shapes and their variations by site class by presenting response spectral figures for every test site; a few locations are presented in this paper with all locations available in the electronic supplement.

These figures include the multi-period response spectra (MPRS) plots for the 2026 NEHRP Provisions and the ASCE 7-22 Standard. They provide complementary information to Section 3 by presenting spectral shapes that are not easy to decipher from only two design parameters, S_{MS} and S_{M1} . Finally, Section 5 presents Seismic Design Category (SDC) change maps for all site classes assuming the same definitions of SDC as in ASCE 7-22, as another means of demonstrating the changes due to the updates to the hazard model and design parameter definitions in the 2026 NEHRP Provisions.

RP3-2.0 Seismic Design Ground Motions in NEHRP Provisions

The seismic design ground motions of Chapter 22 as defined in Chapter 21 in recent NEHRP Provisions have had a probabilistic component and a deterministic component. The probabilistic component is developed through probabilistic seismic hazard analysis (PSHA). The use of PSHA to develop seismic design loads in the U.S. can be traced back to the 1970s. The USGS-produced seismic hazard maps of Algermissen and Perkins (1976), which featured maps of effective peak acceleration (A_a) and effective peak velocity-related acceleration (A_v) with a 10% probability of exceedance in 50 years on reference rock sites, were used initially as the basis for the seismic zone maps and seismic factors in the ATC 3-06 Tentative Provisions (ATC 1978), the 1985 NEHRP Provisions (BSSC 1985), and the 1988 UBC (ICBO 1988). These maps were largely retained through the 1994 NEHRP Provisions despite numerous updates and improvements to the USGS national seismic hazard maps (NSHM). The acronym NSHM that previously referred to the USGS national seismic hazard “maps” was later retained in the 2020 NEHRP Provisions to refer to the USGS national seismic hazard “models,” due in part to introduction of multi-period response spectra (MPRS) that increased the number of maps substantially from three (at PGA, 0.2-sec SA, and 1.0-sec SA for a reference site class) to 168 for a combination of 21 periods (22 if including PGA or zero-period SA) and eight different site classes of interest to the engineering community, at which time the NEHRP Provisions moved from “maps” to a more practical “geodatabase,” <https://doi.org/10.5066/F7NK3C76>, which was commonly accessed by third-party web interfaces, e.g., ASCE, that rely on USGS web services.

As alluded to in the Introduction section, the USGS NSHMs are used in combination with a procedure developed and updated by the BSSC PUC to be converted to seismic design ground motions in the NEHRP Provisions. This procedure has over the years included: 1) application of maximum-direction factors to define directionality of ground motions, 2) integration with a fragility function to compute risk-targeted ground motions, 3) capping by deterministic ground motions with specified lower limits, 4) application of site coefficient factors, and 5) approximating the spectral shape for a range of spectral periods. The above five elements have had their own models and assumptions as defined by the PUC. The last two were replaced with PSHA-derived values when the MPRS concept was adopted in 2020 NEHRP. The third was replaced with PSHA-derived alternatives in these 2026 NEHRP Provisions, thereby eliminating deterministic caps. The first two remain, as there has been no evidence of serious problems that would justify their replacement.

The remainder of this section first summarizes the significant improvements to each version of the USGS NSHMs that have been accepted in the NEHRP Provisions, starting with the 1996 NSHM and up to the 2023 NSHM used in the current provisions. It then summarizes the evolution of the NEHRP site-specific

procedures developed by the BSSC PUC that defined seismic design ground motions from 1997 to these 2026 NEHRP Provisions.

RP3-2.1 USGS National Seismic Hazard Models (NSHMs) in Chapter 22 of NEHRP

The USGS NSHMs perform PSHA on a grid spanning the entire U.S., e.g., 0.05 degrees spacing in the latest conterminous U.S. model, which is approximately 5 km. The primary outputs of PSHA are hazard curves, which can be presented as probabilities of exceeding specified levels of ground shaking measures (e.g., 0.2-sec SA) at a specific location over a given period (e.g., 50 years). Hazard curves are then converted to seismic design ground motions in different ways for various building codes (e.g., uniform-hazard ground motions at different hazard levels, risk-targeted ground motions at different risk levels). For example, reading the intensity that corresponds to a 10% probability of exceedance in 50 years, which would be equivalent to ground shaking with a return period of 475 years, results in a uniform hazard level across the U.S., whereas selecting the intensity that gives a 1%-in-50-years risk of collapse, after the hazard curve is integrated with an appropriate generalized fragility function (i.e., probability of collapse given a ground shaking intensity), conceptually results in a uniform risk of collapse across the U.S.

There are two fundamental input components to a PSHA: 1) an earthquake rupture forecast (ERF) or seismic source model that includes earthquake faults and background or smoothed seismicity in the vicinity of a location, with occurrence rates of potential seismic events of certain magnitudes, and 2) a ground-motion characterization (GMC) that combines ground motion models (GMMs) that estimate the ground shaking intensities for a given seismic event of known magnitude and other characteristics. The geology, tectonics, and earthquake potential represented in the NSHM ERF in the U.S. is diverse, ranging from active tectonic crustal earthquakes in the western U.S. (WUS) to subduction zone earthquakes in the Pacific northwest (PNW) to the stable continental earthquakes in the central and eastern U.S. (CEUS). The USGS NSHMs require GMMs that capture the propagation effects of each tectonic setting. GMMs are evaluated using a published set of selection criteria (Rezaeian et al. 2015) from the collection of all published GMMs. In Alaska, GMMs are similar to those in the WUS and PNW, whereas in Hawaii and Puerto Rico, GMMs include region-specific models that were calibrated to recent earthquake recordings. Both PSHA inputs (ERF and GMC) consist of many other component models, each having gone through their own rigorous review processes, including independent publications, before inclusion in the USGS NSHMs. Logic tree weights are carefully assigned to selected GMMs and ERF components to represent various modeling choices and reflect the overall confidence of the scientific community in each model, constituting epistemic uncertainties.

As emphasized by Box (1976), all models are wrong (i.e., no model is perfect), but some are useful. In every update of the USGS NSHM, accuracy improves in estimating seismic hazard based on the latest and best available data and scientific models. Historically, about every six years, an updated NSHM has been published. Typically, an NSHM includes significant updates to one or more regions rather than the entire country, due to factors such as a recent earthquake or completion of a regional study (Bozorgnia et al., 2022). For example, the WUS GMMs were completely updated to the Next Generation Attenuation (NGA) models in the 2008 and 2014 NSHMs, while this upgrade happened for CEUS GMMs in the 2018 NSHM, and for the PNW subduction zone GMMs in the 2023 NSHM, based on the availability of new science and

resources. The WUS GMMs did not see a significant change in the last two NSHMs (2018 and 2023), other than minor refinements to regional estimates of hazard.

For the conterminous U.S. (i.e., the lower 48 states), or CONUS for short, the 1996 USGS NSHM (Frankel et al. 1996) was the first model directly adopted by the NEHRP Provisions in 1997 and carried through the 2000 NEHRP Provisions. This model was updated by Frankel et al. (2002) for adoption by the 2003 NEHRP Provisions. The 2008 (Petersen et al. 2008) and 2014 (Petersen et al. 2014, 2015) updates of the USGS NSHMs for CONUS were adopted by the 2009 and 2015 NEHRP Provisions, respectively. In the 2018 NSHM update (Petersen et al. 2020), the USGS not only incorporated the latest available science and data but also collaborated closely with the BSSC PUC through Project ‘17 to accommodate the needs of the engineering community for more accurate estimates of site amplifications. For example, Rezaeian et al. (2021) provides details on CEUS-specific needs and resulting changes to the previous NSHMs that did not provide MPRS ground motion estimates in the CEUS; Shumway et al. (2018) presents a detailed description of the hazard geodatabase for a combination of 168 periods and site classes; and Kircher et al. (2020) develops a new method for approximating MPRS ground motions outside of CONUS. This resulted in the adoption of the 2018 NSHM (Petersen et al. 2020) and the MPRS concept in the 2020 NEHRP Provisions (BSSC 2020). A consequence of adopting MPRS was the elimination of site coefficients F_a and F_v , which will be discussed in more detail in the next section. The latest USGS NSHM was developed in 2023 for the 50 United States (Petersen et al. 2024), which included updated hazard models for Alaska, Hawaii, and CONUS. The prior hazard models for Alaska and Hawaii were about 20 years old, developed in 2007 and 1998, respectively. The 2023 NSHM has been adopted in these 2026 NEHRP Provisions. Table 1 lists the USGS NSHMs and the corresponding NEHRP Provisions along with only a few and the most iconic updates to the ERFs or GMCs in each revision. The reader is referred to the associated NSHM publications for a complete list of other important improvements in each NSHM. Also, the commentary to Chapter 22 of each NEHRP Provisions provides a summary of important updates in its adopted NSHM.

Table 1. The USGS NSHMs for the conterminous United States used in the NEHRP Provisions.

USGS NSHM	NEHRP Provisions (FEMA Report #) *ASCE 7 Standard	Major Updates in USGS NSHMs
Algermissen and Perkins (1976) Algermissen et al. (1990)	1985 & 1988 NEHRP <i>1st and 2nd Editions</i> (FEMA 95) 1991 NEHRP <i>3rd Edition</i> (FEMA 222; key basis for national codes)	<ul style="list-style-type: none">First version of NSHMs used in NEHRP Provisions as documented in Leyendecker et al. (1995)

	1994 NEHRP <i>4th Edition</i> (FEMA 222A)	
1996 NSHM Frankel et al. (1996)	1997 & 2000 NEHRP <i>5th & 6th Editions</i> (FEMA 302 & 368; key basis for IBC) * ASCE 7-98 & 7-02 (ASCE 2000, 2003)	<ul style="list-style-type: none"> ▪ New PSHA methodology based on an open consensus-building process involving the community and rigorous reviews. ▪ Implementation documented in Leyendecker et al. (2000)
2002 NSHM Frankel et al. (2002)	2003 NEHRP <i>7th Edition</i> (FEMA 450) *ASCE 7-05 (ASCE 2005)	<ul style="list-style-type: none"> ▪ More explicit treatment of uncertainties ▪ Updated earthquake catalog (recordings through Dec 2001) and new seismicity grids ▪ Improved removal of dependent and induced events ▪ Updated New Madrid source model ▪ Updated Charleston, SC source model ▪ Added new CEUS GMMs ▪ Updated fault recurrence parameters and Cascadia rupture zone geometry ▪ Added new WUS GMMs
2008 NSHM Petersen et al. (2008)	2009 NEHRP <i>8th Edition</i> (FEMA P-750) *ASCE 7-10 (ASCE 2013)	<ul style="list-style-type: none"> ▪ New WUS GMMs (NGA-West1) ▪ Better specification of the reference site class ▪ Addition of magnitude-uncertainty in earthquake catalogs ▪ Updated New Madrid seismic zone ▪ New CEUS GMMs
2014 NSHM Petersen et al. (2014, 2015)	2015 NEHRP <i>9th Edition</i> (FEMA P-1050) *ASCE 7-16 (ASCE 2017)	<ul style="list-style-type: none"> ▪ Broader ranges of uncertainties ▪ New CA source model (Uniform California Earthquake Rupture Forecast, UCERF3) ▪ WUS GMM Updates (used NGA-West2 based on twice as many recorded data relative to 2008) ▪ CEUS SSCn (seismic source characterization for nuclear facilities)

		<ul style="list-style-type: none"> ▪ Cascadia subduction zone source model updates
2018 NSHM Petersen et al. (2020)	2020 NEHRP <i>10th Edition</i> (FEMA P-2082) *ASCE 7-22 (ASCE 2021)	<ul style="list-style-type: none"> ▪ Inclusion of eight site classes everywhere in the U.S. for adoption in NEHRP to use as replacements for the eliminated F_a and F_v (see Table 2, 2020 NEHRP) ▪ Inclusion of 22 periods everywhere in the U.S. for use in calculating the redefined seismic design parameters S_{MS} and S_{M1} in NEHRP (see Table 2, 2020 NEHRP) ▪ CEUS GMM Updates (including NGA-East) ▪ New CEUS aleatory variability model ▪ New CEUS site-effect model ▪ Including long-period deep sedimentary basin amplifications in 4 WUS basins (Seattle, Los Angeles, Bay Area, and Salt Lake City) ▪ Updated seismic catalog (including earthquakes from 2013 to 2017)
2023 NSHM Petersen et al. (2024)	2026 NEHRP	<ul style="list-style-type: none"> ▪ New WUS earthquake rupture forecast ▪ PNW GMM Updates (including NGA-Subduction) ▪ Improving long-period deep sedimentary basin amplifications in 6 WUS basins (including simulations in Seattle and Los Angeles) ▪ WUS-CEUS boundary refinement ▪ New and updated seismic zones in CEUS ▪ Concurrent with new hazard models for Alaska and Hawaii

RP3-2.2 Definitions of Seismic Design Ground Motions in Chapter 21 of NEHRP

The BSSC PUC specifies the site-specific procedures for computing design ground motions from a PSHA in Chapter 21 of the NEHRP Provisions. The USGS uses these procedures to calculate the design ground motions of Chapter 22 from the PUC-adopted NSHMs each cycle (Table 1). The site-specific procedures of Chapter 21 have evolved over the years and, as previously mentioned, include: 1) application of maximum-direction factors, 2) computing uniform-hazard or risk-targeted ground motions, 3) capping by deterministic ground motions, 4) application of site coefficient factors, and 5) approximating the spectral shape for a range of spectral periods. The above list is ordered procedurally when converting hazard curves to design ground motions. The following summarizes the evolution of these components chronologically. Table 2 provides a summary of the design ground motion definition in each edition of the NEHRP Provisions.

As previously mentioned, in 1994 and prior versions of the NEHRP Provisions, uniform-hazard ground motions at a 10% probability of exceedance in 50 years (475-year return period) were used as the basis for the seismic zone maps and were called DE ground motions. Following Project 97's recommendations, the 1997 NEHRP introduced the MCE ground motions based on uniform-hazard ground motions at a 2% probability of exceedance in 50 years (2,475-year return period), subject to deterministic capping, and the DE ground motions were taken as two thirds of the MCE to be used as the input ground motions in structural design procedures. The shift from 10% to 2% in 50-year probabilistic ground motions was driven largely by concerns that the previous design basis may not have adequately protected structures in the CEUS against large and infrequent earthquakes similar to those observed in the 19th century in the New Madrid Seismic Zone and Charleston, SC. At the time, the consensus was that buildings in coastal California were adequately designed against the potential for collapse during large earthquakes. Away from the most active faults there, two thirds of 2%-in-50-year ground motions were roughly equal to the prior 10%-in-50-year ground motions and therefore this definition maintained continuity in design loads. In areas near the most active faults (e.g., the San Andreas), however, the probabilistic ground motions were much larger and therefore deterministic caps were introduced as 1.5 times the median ground motion from a governing active fault characteristic earthquake. The basis of the 1.5 factor provided in the commentary of the 1997 NEHRP Provisions suggest that this factor was selected by Project 97 to "maintain an appropriate level of conservatism in the design process" based on the collective judgment of Project 97 members that "if the structure experiences a level of ground motion 1.5 times the design level, the structure should have a low likelihood to collapse." Later, as will be discussed below for the 2009 NEHRP Provisions updates, Project 07 revised this 1.5 factor to be the 84th-percentile ground motion levels (roughly approximating one standard deviation) from characteristic earthquakes. This revision was used by Project 07 to increase this factor to 1.8 (in 2009 NEHRP) and provide a level of desired conservatism based on the judgment of PUC members for that cycle. In the 1997 NEHRP Provisions, the deterministic capping resulted in MCE ground motions in California with much higher probabilities of exceedance than the intended uniform hazard of 2% in 50 years.

The 1997 NEHRP Provisions also introduced the two-period MCE and DE response spectral shapes for design, which were anchored to SAs at 0.2 sec and 1.0 sec on a reference soft-rock site condition. The spectra for other site conditions were obtained by modifying the 0.2-sec SA and 1.0-sec SA with site amplification coefficients, F_a and F_v , respectively. These site coefficients were developed deterministically based on observations of site effects from ground motion records largely from shallow crustal earthquakes in active tectonic regions in California. The 1996 USGS NSHM was adopted by the 1997 NEHRP as the basis for determining probabilistic ground motions at 0.2 sec and 1.0 sec on a reference soft-rock site condition, which was carried through to the 2000 NEHRP. The 2003 NEHRP adopted the updated 2002 USGS NSHM (refer to Table 1).

The PGA for additional geotechnical investigations was defined as a ratio of the short period ground motion, $S_s/2.5$, at the reference site class and amplified with a site coefficient, F_{PGA} , for other site classes in 1997 to 2003 NEHRP Provisions.

About 10 years after the major revisions of the 1997 NEHRP Provisions on the definition of MCE and the introduction of site coefficients and the two-period spectral shape, following Project 07's

recommendations, the 2009 NEHRP Provisions featured the next set of significant changes to the definition of seismic design ground motions. To conceptually advance from uniform probability of ground-motion exceedance (hazard) to uniform probability of structural collapse (risk), the 2009 NEHRP Provisions introduced Risk-targeted Maximum Considered Earthquake (MCE_R) ground motions as the basis of design (Luco et al. 2007). The procedures for developing MCE_R ground motions involved convolution of the full ground-motion hazard curve with a generic collapse fragility curve, based on FEMA P-695 (ATC 2009), with a 10% probability of collapse given the occurrence of the MCE_R , producing ground motions that would result in a target 1% mean probability of collapse in 50 years in structures designed to the MCE_R . The parameter that determined the slope of the fragility function was assigned $\beta = 0.8$ by Project 07; this was later updated to 0.6 by the ASCE 7-10 Seismic Subcommittee, which deemed that to be a better estimate, based on FEMA P-695 (ATC, 2009) and its four components of this slope/uncertainty: record-to-record, design, test data, and modeling. Overall, the MCE_R ground motions are not very sensitive to the slope of the fragility curve when the “reliability target,” defined as the conditional probability of collapse given the MCE_R ground motion, is $p = 10\%$. This is illustrated in Figure 1 from Luco, (2020), which demonstrates that the risk-targeted ground motions would, on average, change by less than 5% with changes to $\beta = 0.4$ or 0.8.

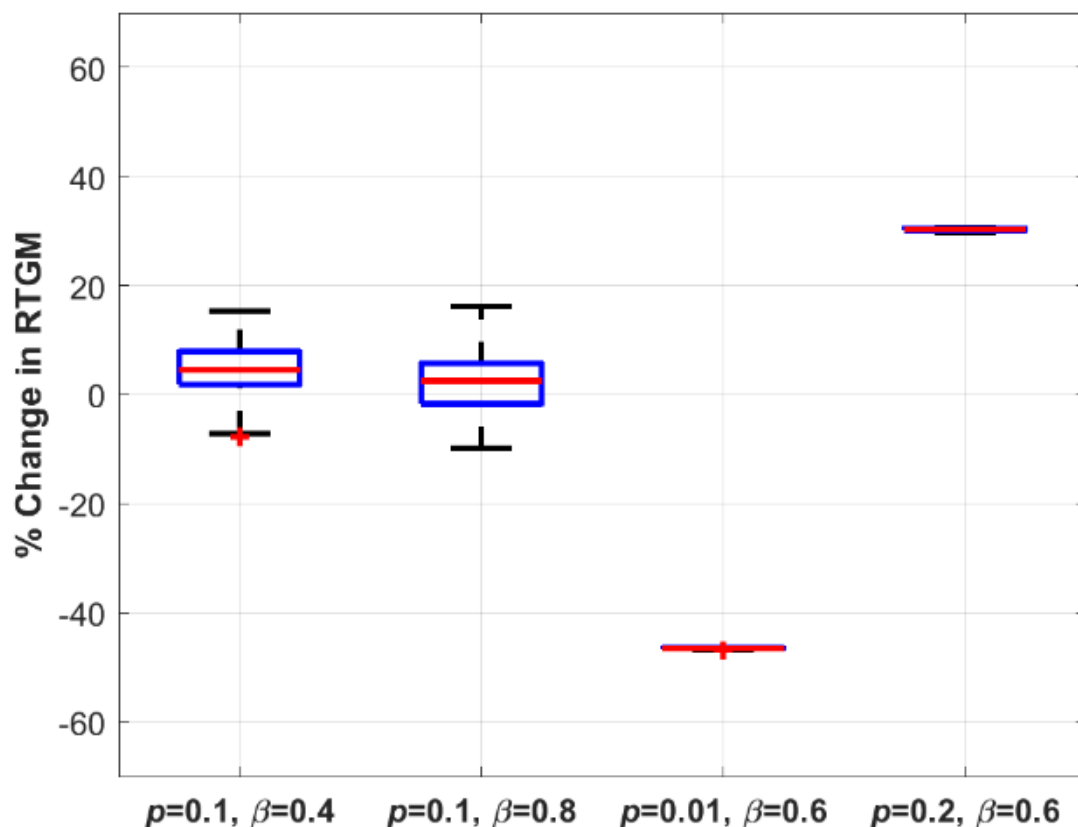


Figure 1. Boxplots of percent changes in risk-targeted ground motions relative to those calculated using the fragility curve parameters of the NEHRP Provisions ($p=0.1, \beta=0.6$). Source: Luco, 2020.

Deterministic capping was retained and applied to MCE_R procedures, resulting now in ground motions corresponding to higher 50-year collapse probabilities in areas governed by deterministic considerations. Regardless, the conditional collapse probability for a given MCE_R value at a site remains at 10% by definition, irrespective of whether the MCE_R value is probabilistic or deterministic. The deterministic ground motions were revised to 1.8 times the median ground motion from a governing characteristic earthquake. As previously mentioned, this 1.8 factor was interpreted by some as a more accurate approximation of one standard deviation, introducing a 20% increase in deterministic ground motions that was considered by the PUC as a desired conservatism near active faults in California. The 2009 NEHRP Provisions also included procedures for adjusting design ground motions from the horizontal geometric mean (or another approximate equivalent to the average horizontal ground motion, dependent on each GMM) to maximum-direction ground motion. The adoption of maximum-direction factors was controversial because despite the added benefits it was thought to introduce bias in the intended risk target among other issues that are outlined in Stewart et al. (2011). The site amplification coefficients and the two-period spectral shape continued through the 2009 and 2015 NEHRP Provisions, with updates in the 2015 NEHRP to F_a and F_v values, and the fragility function parameter $\beta = 0.6$ to follow the ASCE 7-10 Standard (Luco et al. 2015). The updates to F_a and F_v in the 2015 NEHRP were motivated by Seyhan and Stewart (2014) to update the reference site condition used for the coefficients to match the condition used in the NSHMs, i.e., 760 m/s, and to incorporate the substantial knowledge gains on site response over the past two decades. The MCE_G defined the PGA as the uniform-hazard ground motion at a 2% probability of exceedance in 50 years with a deterministic cap.

Table 2. The evolution of seismic design ground motions in NEHRP Provisions and ASCE 7 Standards.

NEHRP Provision (FEMA Report) *ASCE 7 Standard	Seismic Design Ground Motion (NA: Not Applicable)
1985, 1988, 1991, and 1994 NEHRP (FEMA 95, 95, 222, 222A)	<ul style="list-style-type: none"> ▪ DE: 10% probability of exceedance in 50yrs (uniform hazard) ▪ Directionality: Geometric Mean ▪ Site Effects: NA ▪ Spectral Shape: NA
1997 & 2000 NEHRP (FEMA 302, 368) * ASCE 7-98 & 7-02	<ul style="list-style-type: none"> ▪ DE: $\frac{2}{3}$ MCE ▪ MCE: 2% probability of exceedance in 50yrs (uniform hazard), with deterministic capping ▪ Deterministic Cap: 1.5×Median from Characteristic Earthquake ▪ Directionality: Geometric Mean

	<ul style="list-style-type: none"> ▪ Site Effects: Site Coefficients, F_a and F_v ▪ Spectral Shape: Two-Period Spectrum
2003 NEHRP (FEMA 450) *ASCE 7-05	<ul style="list-style-type: none"> ▪ Same as above
2009 NEHRP (FEMA P-750) *ASCE 7-10	<ul style="list-style-type: none"> ▪ MCE_R: 1% probability of collapse in 50yrs (uniform risk), with deterministic capping ▪ Fragility: 10% probability of collapse given MCE_R with $\beta = 0.8$ (*0.6 in ASCE 7-10) ▪ Deterministic Cap: 1.8×Median from Characteristic Earthquake ▪ Directionality: Maximum Direction ▪ Site Effects: Site Coefficients, F_a and F_v ▪ Spectral Shape: Two-Period Spectrum ▪ MCE_G: 2% probability of exceedance in 50yrs (uniform hazard), with deterministic capping
2015 NEHRP (FEMA P-1050) *ASCE 7-16	<ul style="list-style-type: none"> ▪ MCE_R: 1% probability of collapse in 50yrs (uniform risk) ▪ Directionality: Maximum Direction ▪ Fragility: 10% probability of collapse given MCE_R with $\beta = 0.6$ ▪ Deterministic Cap: 1.8×Median from Characteristic Earthquake ▪ Site Effects: Updated Site Coefficients, F_a and F_v ▪ Spectral Shape: Two-Period Spectrum ▪ MCE_G: 2% probability of exceedance in 50yrs (uniform hazard), with deterministic capping
2020 NEHRP (FEMA P-2082) *ASCE 7-22	<ul style="list-style-type: none"> ▪ MCE_R: 1% probability of collapse in 50yrs (uniform risk), with deterministic capping ▪ Fragility: 10% probability of collapse given MCE_R with $\beta = 0.6$ ▪ Deterministic Cap: Based on Scenario Earthquakes Selected from Disaggregation of Hazard ▪ Directionality: Updated Maximum Direction Factors ▪ Site Effects: USGS PSHA-based Site Amplifications. The site coefficients at 0.2 and 1.0 sec were replaced with site amplification adjustments at each spectral period within the PSHA, providing more accurate estimates of site effects, at times significantly different from the previous site coefficients with large impacts to design ground motions ▪ Spectral Shape: Multi-Period Response Spectra (MPRS) and Two-Period Spectrum Derived from MPRS

	<ul style="list-style-type: none"> ▪ MCE_G: 2% probability of exceedance in 50yrs (uniform hazard), with deterministic capping
2026 NEHRP	<ul style="list-style-type: none"> ▪ MCE_R: 1.5% probability of collapse in 50yrs (uniform risk) ▪ Fragility: 10% probability of collapse given MCE_R with $\beta = 0.6$ ▪ Deterministic Cap: Eliminated ▪ Directionality: Maximum Direction ▪ Site Effects: USGS PSHA-based Site Amplifications ▪ Spectral Shape: Multi-Period Response Spectra (MPRS) and Two-Period Spectrum Derived from MPRS ▪ MCE_G: 3% probability of exceedance in 50yrs (uniform hazard)

Once again, about 10 years after the major reforms of the 2009 NEHRP Provisions on the definition of MCE_R and introduction of maximum direction factors, Project 17 (BSSC 2019) was formed as a joint committee of practicing engineers and USGS scientists to advise the BSSC PUC in development of the 2020 NEHRP Provisions. At this time, issues had been noticed with deterministic caps and with site coefficients and spectral shapes at longer spectral periods on soft site classes in higher-hazard regions. In the interim, the 2015 NEHRP Provisions required site-specific analysis for estimating long-period ground motions on soft site classes in high hazard regions or provided the option of multiplying such ground motions by a factor of 1.5 to avoid underestimating ground motions in such situations. While the 2020 NEHRP Provisions continued to use the 1% in 50-year MCE_R and its deterministic capping as the basis of design, the procedures for developing MCE_R ground motions were updated significantly to leverage advances in seismic hazard modeling and data availability. Specifically, following Project 17's recommendations, the two-period spectral shape was supplemented by the introduction of more detailed and more accurate multi-period response spectra (MPRS). Development of MPRS involved direct calculation of probabilistic and deterministic ground motions at 21 spectral periods from 0.01 sec to 10 sec (22 periods if including PGA). Similarly, site coefficients were superseded by directly calculating MPRS for each site class, redefined in the 2020 NEHRP Provisions Chapter 20 as the eight Site Classes A, B, BC (reference soft rock), C, CD, D, DE, and E based on V_{S30} . Default site condition was defined as the maximum of Site Classes C, CD, and D. Overall, these updates resulted in more accurate design response spectra that better reflect the variation in spectral shapes across different regions, seismotectonic environments, and site conditions because the site effects in each GMM were now applied directly in the PSHA, as intended, rather than deterministically to the outputs of the PSHA as was done previously in applications of F_a and F_v . The resulting seismic design parameters in the 2020 NEHRP Provisions in the CEUS, in particular, were affected significantly by this change because in this region site coefficients were previously based on WUS data and models. The differences at long periods were significant and unexpected to many practicing engineers who were not familiar with the different CEUS soil structure. In the WUS, the application of MPRS addressed the potentially significant underestimation of design forces for long-period structures on soft site conditions in regions that could observe large earthquakes, which was the original motivating factor for Project 17's recommendation of MPRS. In addition, in the 2020 NEHRP Provisions

the definition of active faults, the determination of deterministic earthquakes, and the deterministic lower limits were also revised, as explained in the commentary there.

Adoption of MPRS also had several major implications for the USGS in developing NSHMs. Previously, USGS-developed NSHMs were primarily used for computation of mean-hazard ground motions at the reference site condition for PGA, 0.2-sec SA and 1.0-sec SA at a 5% damping ratio. As a result, the USGS GMM selection criteria were limited to the performance of models at these three periods and at one reference site condition. Other periods and site classes were also available through USGS reports and online resources over the past several cycles of updates (e.g., Shumway et al., 2018). However, these were not used in the NEHRP Provisions and therefore not scrutinized at the same level as PGA, 0.2-sec SA, and 1.0-sec SA. Furthermore, GMMs were not available at all regions in the U.S. for all periods and site classes of interest, particularly in the CEUS and for long periods on deep sedimentary basins of the WUS. Through Project 17's coordination, USGS updated their NSHM GMM selection criteria. Through coordination with GMM developers, existing models were updated to expand their applicability, and new models were developed to facilitate application of MPRS in all U.S. regions (refer to Rezaeian et al. 2021 for CEUS GMMs, to Powers et al. 2021 for WUS GMMs for deep sedimentary basins, and to Kircher et al. 2020 / FEMA P-2078 for locations outside CONUS). Another implication of adopting MPRS was a major move from the three maps in every region (for PGA, 0.2-sec SA, and 1.0-sec SA for reference site conditions) to a geo-database for the 22 periods and eight site classes accessed through USGS-developed web services that could be easily included in third-party, user-friendly web interfaces such as the [ASCE Hazard Tool](https://ascehazardtool.org/) (<https://ascehazardtool.org/>). Finally, the site coefficients F_a and F_v (and F_{PGA} for PGA) were eliminated from the NEHRP Provisions since they were directly captured in the USGS-provided design ground motion values.

Although it had not been quite 10 years since the development of the 2020 NEHRP Provisions, another major revision happened in the development of design ground motions in these 2026 NEHRP Provisions. This revision focused on the issues with deterministic caps that were identified in the previous cycle but were not fully resolved (Stewart et al., 2020). The deterministic lower limits based on a magnitude 8 event were being applied to areas where maximum earthquake magnitudes were less than 7. The deterministic caps introduced non-transparent higher hazard and risk levels near active faults than the uniform-hazard and risk-targeted ground motions corresponded to in the rest of the U.S. Defining deterministic events was simpler 30 years ago when faults were oversimplified and characteristic earthquakes were used in modeling the seismic hazard. Characteristic events are no longer specified in the USGS NSHMs, due to more information on seismic events and better models that allow for more realistic fault segmentations and multi-fault ruptures, among other improvements. As a result, determination of deterministic events is problematic. The definition of an active fault is another issue that has been revised in several cycles with subjective thresholds to include or exclude certain faults based on judgement of PUC members. Further, the deterministic lower limits, which were based on a magnitude 8 event at a distance of 12.5 km, were starting to cause problems in certain regions where practicing engineers argued such an event was not appropriate. The problems with deterministic caps became more pronounced with the introduction of MPRS in the 2020 NEHRP Provisions and the realization that at one given location different deterministic events control the ground motions for different periods and site classes. As explained in the commentary to Chapter 21 of these 2026 NEHRP Provisions, the deterministic caps are replaced with a fully probabilistic approach, also eliminating the deterministic lower limit. For MCE_R spectral response accelerations, the target risk of building collapse increases from 1% to 1.5% in 50 years to ensure a

uniform, transparent level of safety nationwide and for all spectral periods and site conditions. The 1.5% in 50 years target risk is based on the average collapse risk across the western United States that resulted from the deterministically capped MCE_R ground motions of ASCE 7-22. The “reliability” (conditional probability of collapse) remains at 10% collapse given the MCE_R ground motion. Similarly, for MCE_G PGA the uniform-hazard level increases from 2% to 3% in 50 years and deterministic caps are eliminated.

RP3-3.0 Design Ground Motions for Test Sites

In the NEHRP Provisions, a set of 34 example sites was used in the commentary of Chapter 22 to monitor and explain changes from one cycle to another. These sites were in CONUS and mainly located in western states with very few representations from CEUS. They were also all corresponding to largely populated cities where most past PUC representatives had practiced engineering design. A few additional sites were added to this set in the present, 2026, NEHRP Provisions, including two locations in the CEUS, two locations in Alaska, and two locations in Hawaii. However, it is acknowledged that a larger set would not be practical to include in the provisions. In this section, a more comprehensive set of example sites, referred to hereinafter as test sites, are introduced. The site names and coordinates for 178 sites in CONUS, 33 sites in Alaska, and 36 sites in Hawaii are presented in Table A1 of Appendix A, Table B1 of Appendix B, and Table C1 of Appendix C, respectively. These sites were selected because they either have a large population, or interesting seismological or geological features, or are locations of important infrastructures or facilities. The seismic design parameters as defined in Chapter 21, S_s and S_1 for the reference site class (at 0.2 sec and 1.0 sec), and S_{MS} and S_{M1} for the default site conditions (90% of the maximum acceleration and velocity at a range of periods) are provided for the test sites in Tables A2 and A3 for CONUS in Appendix A, in Tables B2 and B3 for Alaska in Appendix B, and in Tables C2 and C3 for Hawaii in Appendix C. The parameters S_s , S_1 , S_{MS} and S_{M1} are provided for this, 2026, NEHRP Provisions as well as the previous four cycles, referred to as ASCE 7-22, ASCE 7-16, ASCE 7-10, and ASCE 7-05 (refer to Table 1 and 2 for definitions of design parameters and the underlying USGS NSHMs for each cycle). In an electronic supplement, similar S_{MS} and S_{M1} data are provided every site class if applicable (e.g., ASCE 7-05, 7-10, and 7-16 did not have Site Classes DE, CD, or BC and therefore parameters are not listed for those revisions). Note that there may be additional guidelines and regulations in the present, 2026, NEHRP Provision that would limit the use of what is presented here for long period motions for Site Classes DE and E in the CEUS (e.g., to not be lower than the values of D). Since such limitations are currently under consideration at the time of the writing of this resource paper, they have not been discussed nor included here.

The 178 CONUS test sites include the previous 34 sites plus the two new CEUS sites in the current commentary to Chapter 22. To better understand the regional changes in ground motion parameters from one cycle to another, each CONUS test site were assigned to a region that is approximately defined as northern California (NorCA) for locations with latitudes above 37 degrees in the state of CA, central California (CntCA) for locations with latitudes between 36 and 37 degrees in the state of CA, southern California (SoCA) for locations with latitudes below 36 degrees in the state of CA, Pacific northwest (PNW) for locations in the states of WA and OR, Nevada (NV) for locations in the state of NV, intermountain west (IMW) for locations with longitudes between -115 and -100 degrees, and central and eastern U.S. (CEUS) for locations east of longitude -100 degrees.

The data for S_s and S_1 for the reference site class, $V_{s30}=760$ m/s, are mainly included to show how the hazard models of the USGS changed at set short and long periods of 0.2 and 1.0 sec without the application of site amplification coefficients that also evolved over the cycles. The data for S_{MS} and S_{M1} for the default site conditions show the influence of all changes on design parameters from one cycle to another. A list of the tables in the appendices and the electronic supplement ([Electronic Supplement for Resource NEHRP Part 3 Paper for Chapter 22 Ground Motions – ScienceBase-Catalog](#)) are presented below.

RP3-3.1 Tables for Conterminous U.S. (CONUS):

- **Table A1** (in Appendix A). The 178 CONUS test sites, site names, coordinates, region.
- **Table A2** (in Appendix A). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_s and S_1 (reference site class, B or BC depending on the edition, at 0.2 and 1.0 Sec) values for the 178 CONUS test sites. All values are rounded to two decimal places.
- **Table A3** (in Appendix A). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} (default site conditions as defined in each edition: in the 2026 NEHRP and ASCE 7-22, “Default” is the maximum of C, CD, and D; in ASCE 7-16, “D-default” is per Section 11.4.4 equivalent to the greater of C and D; in ASCE 7-10 and ASCE -05, default is D) values for the 178 CONUS test sites. Where values are listed as NaN, in ASCE 7-16, site-specific procedures were required due to known problems with the F_v parameter. All values are rounded to two decimal places.
- **Table A4** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class A for the 178 CONUS test sites. All values are rounded to two decimal places.
- **Table A5** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class B for the 178 CONUS test sites. All values are rounded to two decimal places.
- **Table A6** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class BC for the 178 CONUS test sites. All values are rounded to two decimal places.
- **Table A7** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class C for the 178 CONUS test sites. All values are rounded to two decimal places.
- **Table A8** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class CD for the 178 CONUS test sites. All values are rounded to two decimal places.
- **Table A9** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class D for the 178 CONUS test sites. All values are rounded to two decimal places.

- **Table A10** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class DE for the 178 CONUS test sites. All values are rounded to two decimal places.
- **Table A11** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class E for the 178 CONUS test sites. All values are rounded to two decimal places.

RP3-3.2 Tables for Alaska:

- **Table B1** (in Appendix B). The 33 Alaska (AK) test sites, site names, coordinates, region.
- **Table B2** (in Appendix B). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_s and S_1 (reference site class, B or BC depending on the edition, at 0.2 and 1.0 Sec) values for the 33 AK test sites. All values are rounded to two decimal places.
- **Table B3** (in Appendix B). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} (default site conditions as defined in each edition: in the 2026 NEHRP and ASCE 7-22, “Default” is the maximum of C, CD, and D; in ASCE 7-16, “D-default” is per Section 11.4.4 equivalent to the greater of C and D; in ASCE 7-10 and ASCE -05, default is D) values for the 33 AK test sites. Where values are listed as NaN, in ASCE 7-16, site-specific procedures were required due to known problems with the F_v parameter. All values are rounded to two decimal places.
- **Table B4** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class A for the 33 AK test sites. All values are rounded to two decimal places.
- **Table B5** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class B for the 33 AK test sites. All values are rounded to two decimal places.
- **Table B6** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class BC for the 33 AK test sites. All values are rounded to two decimal places.
- **Table B7** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class C for the 33 AK test sites. All values are rounded to two decimal places.
- **Table B8** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class CD for the 33 AK test sites. All values are rounded to two decimal places.

- **Table B9** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class D for the 33 AK test sites. All values are rounded to two decimal places.
- **Table B10** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class DE for the 33 AK test sites. All values are rounded to two decimal places.
- **Table B11** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class E for the 33 AK test sites. All values are rounded to two decimal places.

RP3-3.3 Tables for Hawaii:

- **Table C1** (in Appendix C). The 36 Hawaii (HI) test sites, site names, coordinates, region.
- **Table C2** (in Appendix C). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_s and S_1 (reference site class, B or BC depending on the edition, at 0.2 and 1.0 Sec) values for the 36 HI test sites. All values are rounded to two decimal places.
- **Table C3** (in Appendix C). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} (default site conditions as defined in each edition: in the 2026 NEHRP and ASCE 7-22, “Default” is the maximum of C, CD, and D; in ASCE 7-16, “D-default” is per Section 11.4.4 equivalent to the greater of C and D; in ASCE 7-10 and ASCE -05, default is D) values for the 36 HI test sites. Where values are listed as NaN, in ASCE 7-16, site-specific procedures were required due to known problems with the F_v parameter. All values are rounded to two decimal places.
- **Table C4** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class A for the 36 HI test sites. All values are rounded to two decimal places.
- **Table C5** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class B for the 36 HI test sites. All values are rounded to two decimal places.
- **Table C6** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class BC for the 36 HI test sites. All values are rounded to two decimal places.
- **Table C7** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class C for the 36 HI test sites. All values are rounded to two decimal places.
- **Table C8** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class CD for the 36 HI test sites. All values are rounded to two decimal places.

- **Table C9** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class D for the 36 HI test sites. All values are rounded to two decimal places.
- **Table C10** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class DE for the 36 HI test sites. All values are rounded to two decimal places.
- **Table C11** (in electronic supplement). Comparisons between the 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} of Site Class E for the 36 HI test sites. All values are rounded to two decimal places.

Scatter plots of seismic design parameters are shown in Figures 2 and 3 for CONUS for short and long periods, respectively. The USGS and PUC often use similar plots to understand if ground motion parameters consistently increase or decrease in certain regions, and if so, whether the changes are accurate and justifiable. More detailed explanations of the origin of changes larger than 20% (the limit to using site-specific ground motions at any location compared to Chapter 22 values in the 2020 NEHRP) are presented in the commentary of Chapter 22 in every revision.

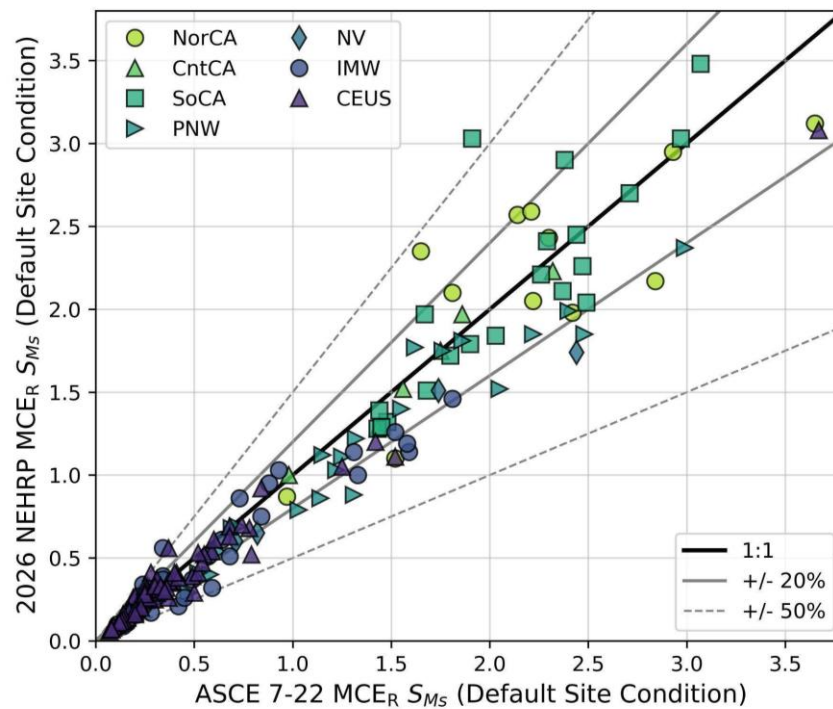


Figure 2. Scatter plot of S_{MS} for CONUS test sites for default site conditions changing from ASCE 7-22 on the horizontal axes to the 2026 NEHRP on the vertical axes, color-coded by regions.

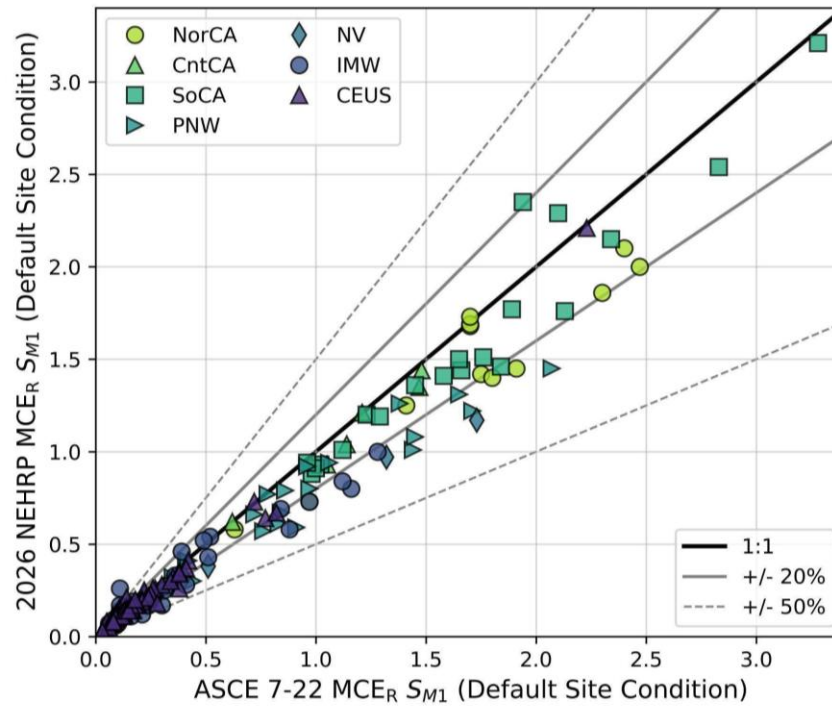


Figure 3. Scatter plot of S_{M1} for CONUS test sites for default site conditions changing from ASCE 7-22 on the horizontal axes to the 2026 NEHRP on the vertical axes, color-coded by regions.

RP3-4.0 Multi-Period Response Spectra (MPRS) Plots for Test Sites

In the previous section, the short-period and long-period ground motion parameters were presented for test sites for all site classes. In this section, MPRS plots demonstrate all periods of interest for every test site in a figure. These plots demonstrate the shape of the response spectrum for a period range between 0 and 5 sec to understand the changes in spectral shapes over different site classes and between the ASCE 7-22 and the 2026 NEHRP Provisions, the only two versions after the introduction of MPRS and removal of site coefficients. A few figures for representative sites from every region are shown below, but figures for every test site in CONUS, Alaska, and Hawaii are available through the electronic supplement ([Electronic Supplement for Resource NEHRP Part 3 Paper for Chapter 22 Ground Motions - ScienceBase-Catalog](#)). The changes in design ground motion parameters from the ASCE 7-22 to the 2026 NEHRP Provisions are discussed in the commentaries of Chapters 21 and 22 of the 2026 NEHRP Provisions. In these commentaries, changes are attributed to a combination of changes to the definitions of design parameters (e.g., removal of deterministic caps and change in the risk target) and improvements to seismic hazard estimates that are described in detail in USGS NSHM publications referenced in earlier sections.

These plots show the risk-targeted maximum considered earthquake, MCE_R (refer to Table 2), multi-period response spectra (MPRS) for the 2026 NEHRP Provisions and the ASCE 7-22 Standard for Site Classes A, B, C, D, and E for each test site. On each figure, corresponding values of S_{MS} and S_{M1} are also provided from Section 3 as complementary information.

Note that there may be additional guidelines and regulations in the ASCE 7 Standard that follow these 2026 NEHRP Provisions that would limit the use of what is presented here for 1) long period motions for Site Classes DE and E (e.g., to not be lower than the values for D in CEUS), and 2) short period truncation of MPRS (e.g., design per Chapters 12, 13, or 15). Since such limitations are under consideration at the time of writing this resource paper, they have not been included here.

The MPRS plots presented in this section illustrate how spectral shapes vary from one site class to another, and how this variation differs in different regions. In Figures 4, 5, and 6, for locations in San Jose, Santa Cruz, and Northridge in CA, the peak of the spectrum as measured by S_{MS} increases gradually and shifts towards slightly longer periods as the site class becomes softer; whereas at long periods, as measured by S_{M1} , values increase significantly from Site Class D to E, and can be very large. Whereas the former is typical behavior expected from previous site coefficients, the latter was unexpected to many practicing engineers who were not aware of the deficiencies of the previous site coefficients at long periods and soft soil conditions in highly seismic regions. The large values of S_{M1} are also due to their revised definitions in the 2020 NEHRP Provisions that considers the maximum response for a range of periods rather than only considering the 1.0-sec spectral period as was done in the past. These effects were explained in the commentary to Chapter 22 of the 2020 NEHRP Provisions.

Figure 7 illustrates the same for a location in Seattle, WA, where high seismicity due to subduction earthquakes causes completely different spectral shapes compared to examples in CA. However, the

spectral peaks still gradually increase and shift to longer periods as the site class softens. The shift to longer periods is much less pronounced in the lower seismicity location shown in Figure 8 for Las Vegas, NV, with more traditionally expected gradual increases in the design parameters. A somewhat similar behavior is observed in Figure 9 for Salt Lake City, UT, but there is an obvious decrease in the short period peaks from Site Class C down to D and E, a good example to demonstrate the reason for default site conditions being the maximum of Site Classes C, CD, and D.

Figures 10 and 11 are examples of two CEUS locations where significant changes were observed from 2015 to 2020 NEHRP Provisions due to the introduction of MPRS and elimination of site coefficients F_a and F_v . The soil structure in the CEUS is very different from that of the western States, but the site coefficients that were developed for WUS were previously being used in the CEUS. As shown in these two figures, ground motions in the CEUS have a high frequency content (i.e., peaks at shorter periods) with nonlinear soil effects at long periods lowering the spectral values. As a result, ground motions do not increase in the same way as in the WUS when site classes become softer.

Finally, Figures 12-16 illustrate the spectral shapes for example locations in Alaska and Hawaii.

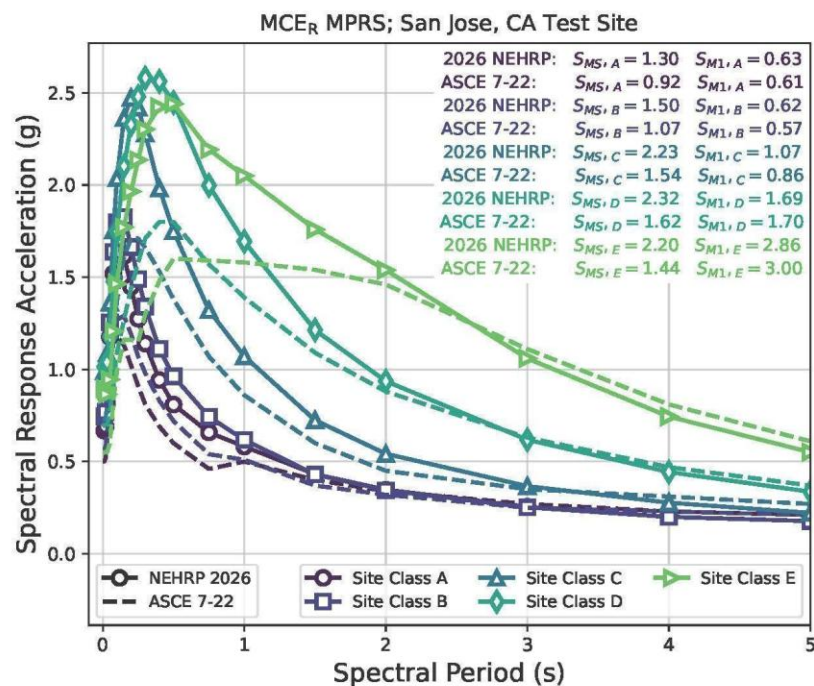


Figure 4. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for San Jose, CA, NorCA (refer to Table A1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

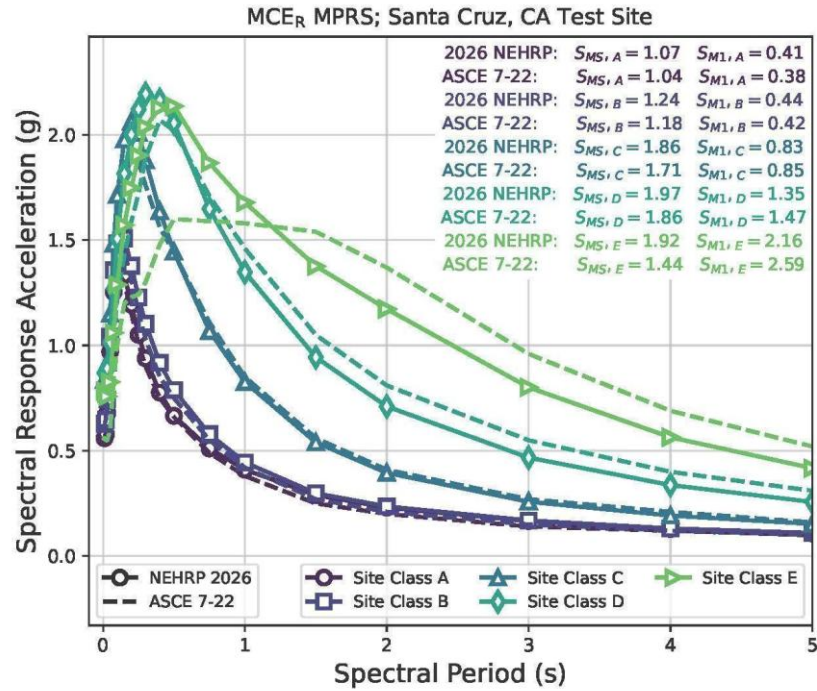


Figure 5. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Santa Cruz, CA, CntCA (refer to Table A1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

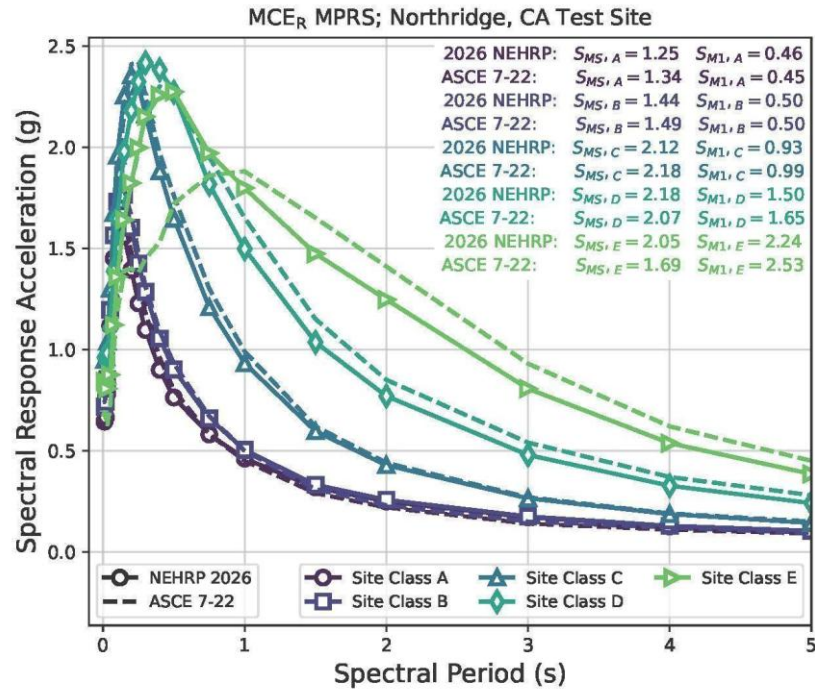


Figure 6. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Northridge, CA, SoCA (refer to Table A1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

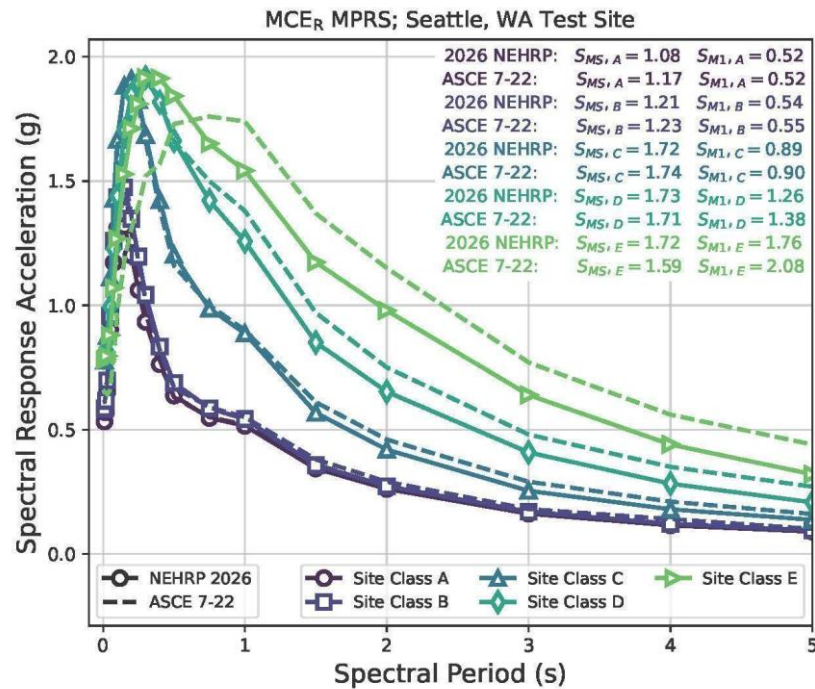


Figure 7. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Seattle, WA, PNW (refer to Table A1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

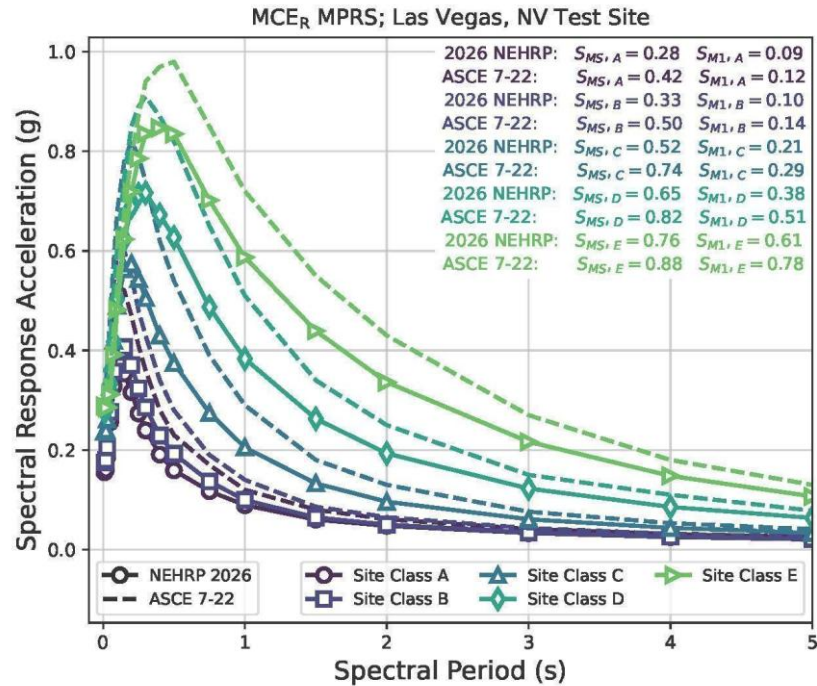


Figure 8. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Las Vegas, NV (refer to Table A1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

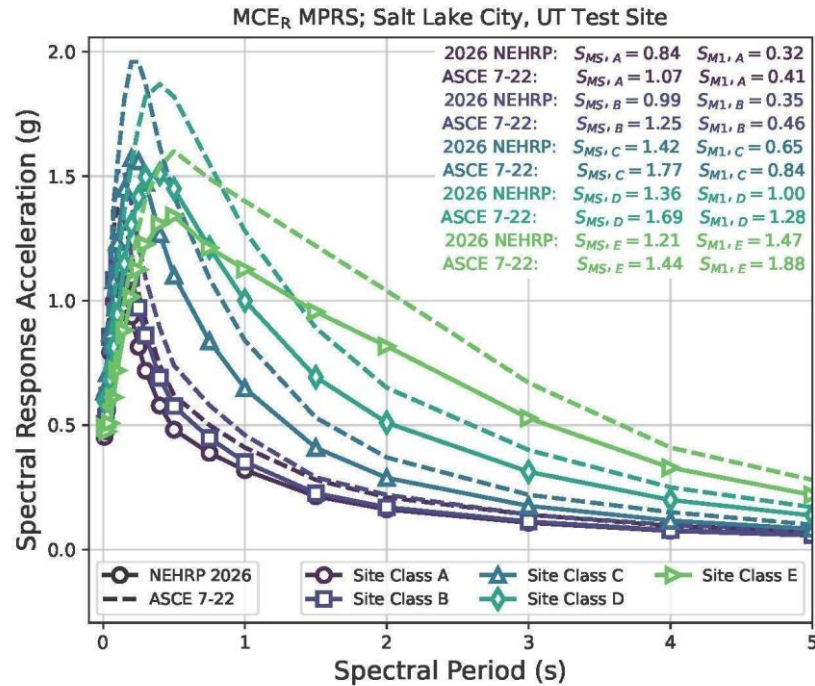


Figure 9. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Salt Lake City, UT, IMW (refer to Table A1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

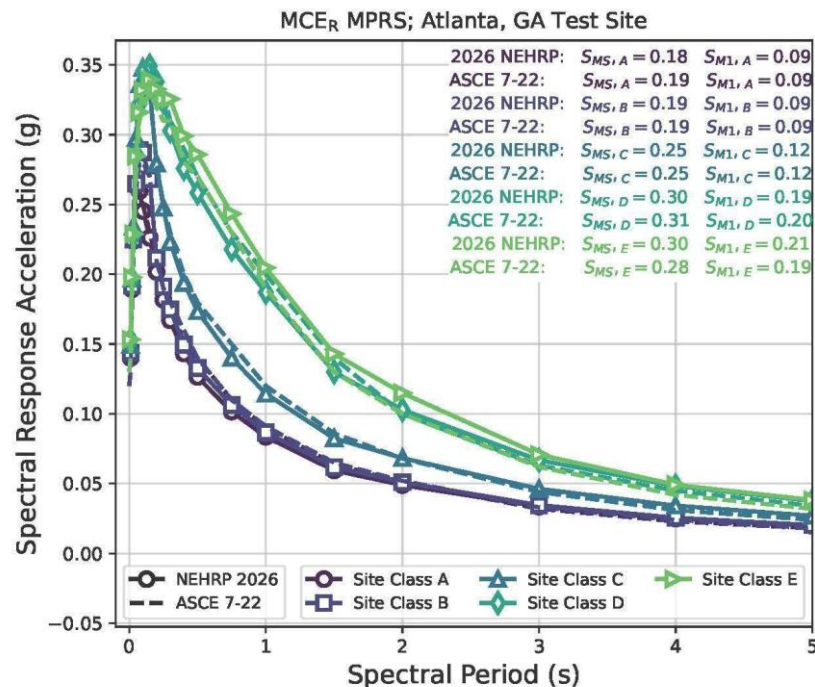


Figure 10. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Atlanta, GA, CEUS (refer to Table A1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

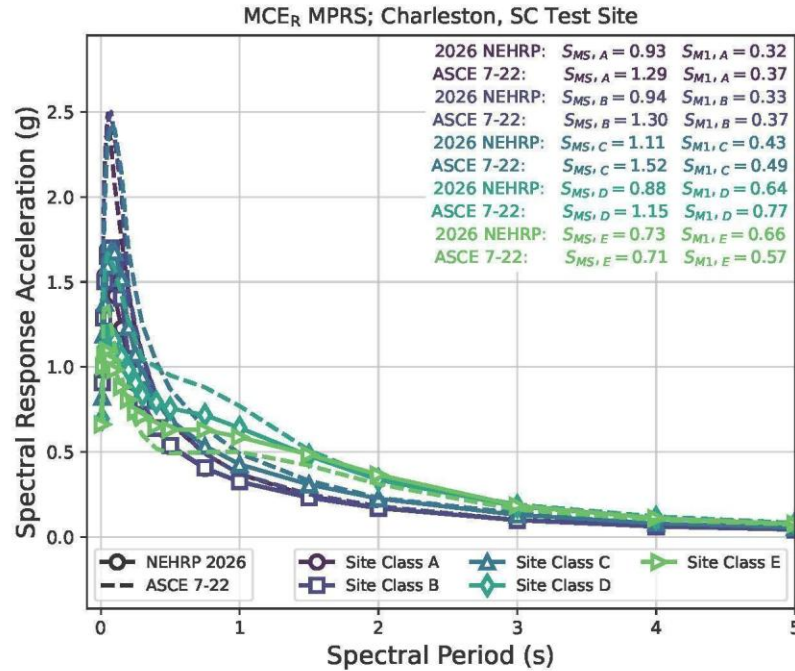


Figure 11. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Charleston, SC, CEUS (refer to Table A1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

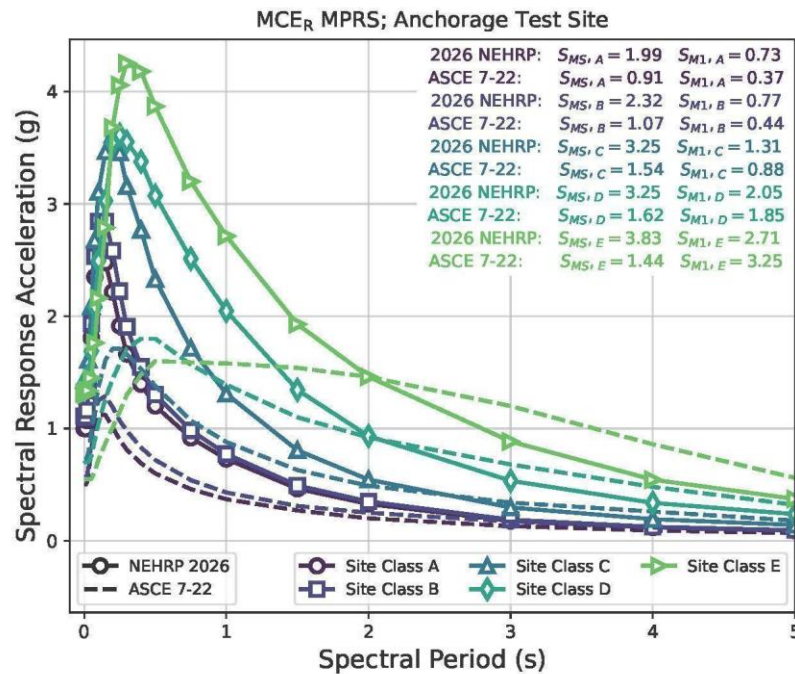


Figure 12. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Anchorage, AK (refer to Table B1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

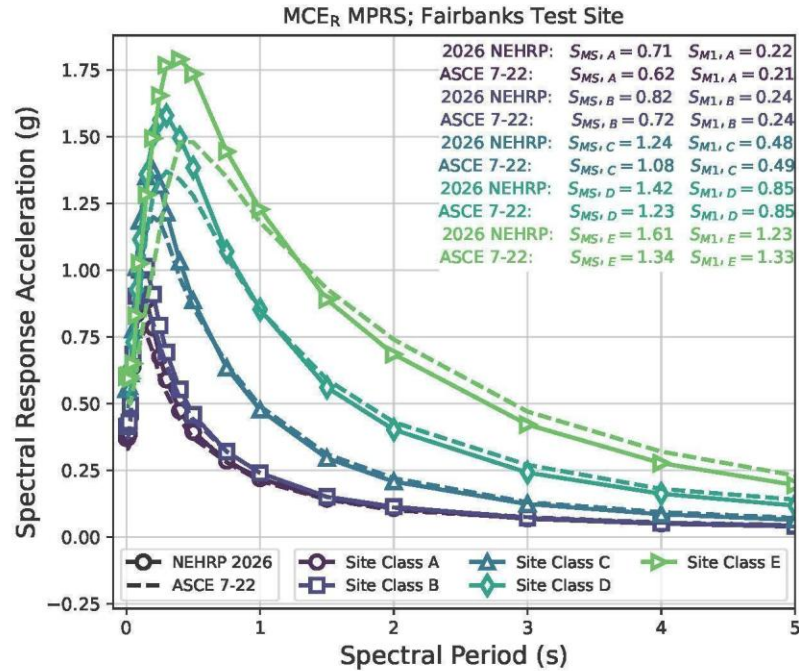


Figure 13. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Fairbanks, AK (refer to Table B1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

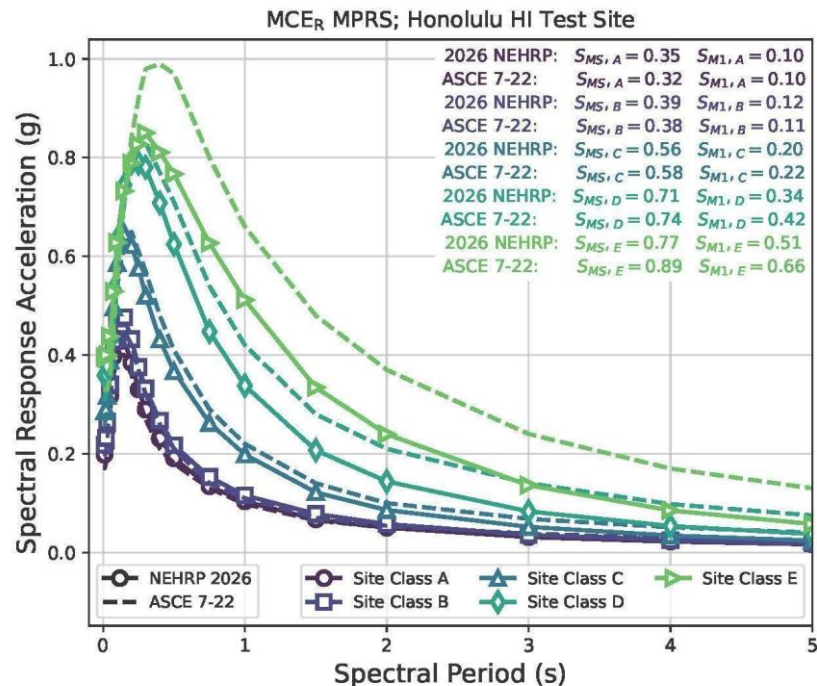


Figure 14. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Honolulu, HI (refer to Table C1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

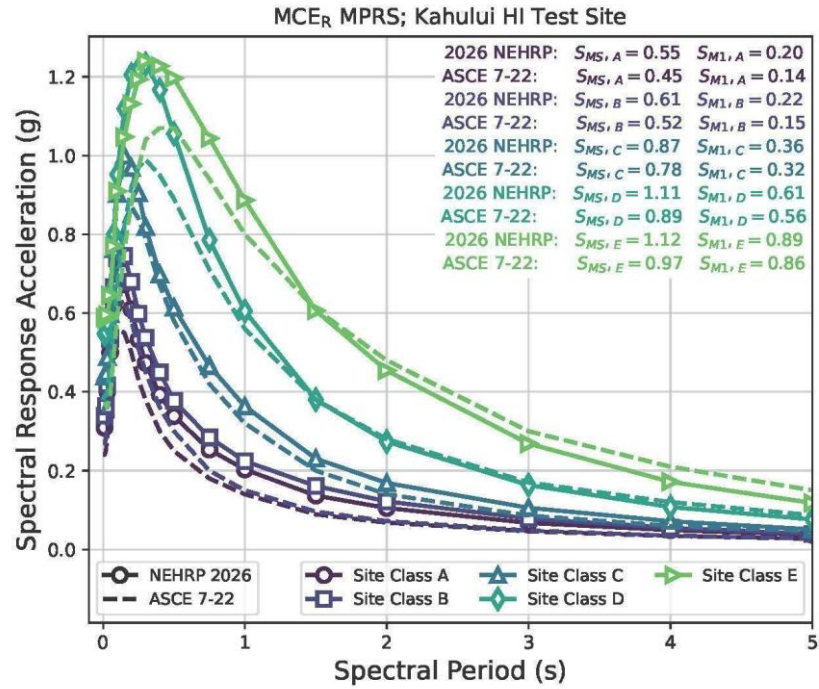


Figure 15. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Kahului, HI (refer to Table C1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

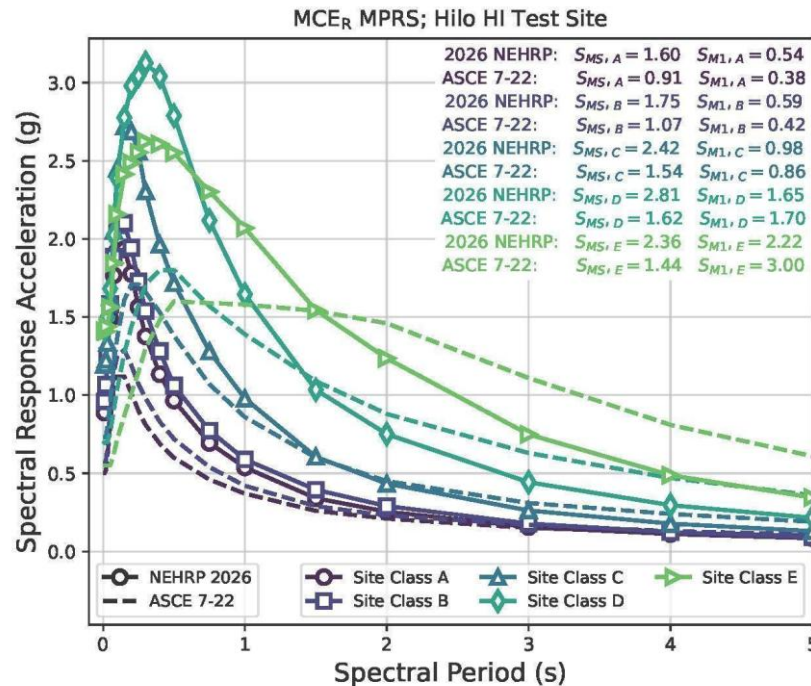


Figure 16. Risk-targeted maximum considered earthquake multi-period response spectra for the 2026 NEHRP and ASCE 7-22 for Site Classes A, B, C, D, and E for Hilo, HI (refer to Table C1). Corresponding values of S_{MS} and S_{M1} are also provided as complementary information.

RP3-5.0 Seismic Design Category (SDC) Change Maps for the Changing MCE_R Definition and the New CONUS Hazard Model

This section presents maps of seismic design category (SDC) changes as they were defined in ASCE 7-22. At the time of writing this manuscript, several potential proposals for changing the definition of SDC are under consideration by the BSSC PUC of the present, 2026, NEHRP Provisions. These potential changes are not considered in the results presented in this section, to keep the focus on the consequences of the two factors discussed in this paper: 1) the changes to the MCE_R definition from the deterministic capped ground motions of ASCE 7-22 to the increased 1.5%-in-50 years target risk of collapse in the 2026 NEHRP Provisions, and 2) the update to hazard model from 2018 NSHM to 2023 NSHM in CONUS. These hypothetical maps demonstrate changes over the last cycle from ASCE 7-22 to 2026 NEHRP without any changes to SDC definitions of ASCE 7-22. Furthermore, additional restrictions are being considered to be imposed on CEUS ground motions for Site Classes DE and E and therefore caution is advised when referring to the results in Figures 24 and 25, respectively for Site Classes DE and E.

The SDC changes across the CONUS for default site conditions (Figure 17) demonstrate that although the replacement of deterministic capping with the increased 1.5%-in-50 year target risk conceptually decreases the SDC across most of the country (i.e., locations that were not deterministically capped), when combined with the update to the 2023 USGS NSHM, the SDC changes in few geographic

areas, mainly in narrow regions indicating areas affected by crossing of artificial judgement-based thresholds such as those in determining deterministic events, lower limits, or SDC boundaries.

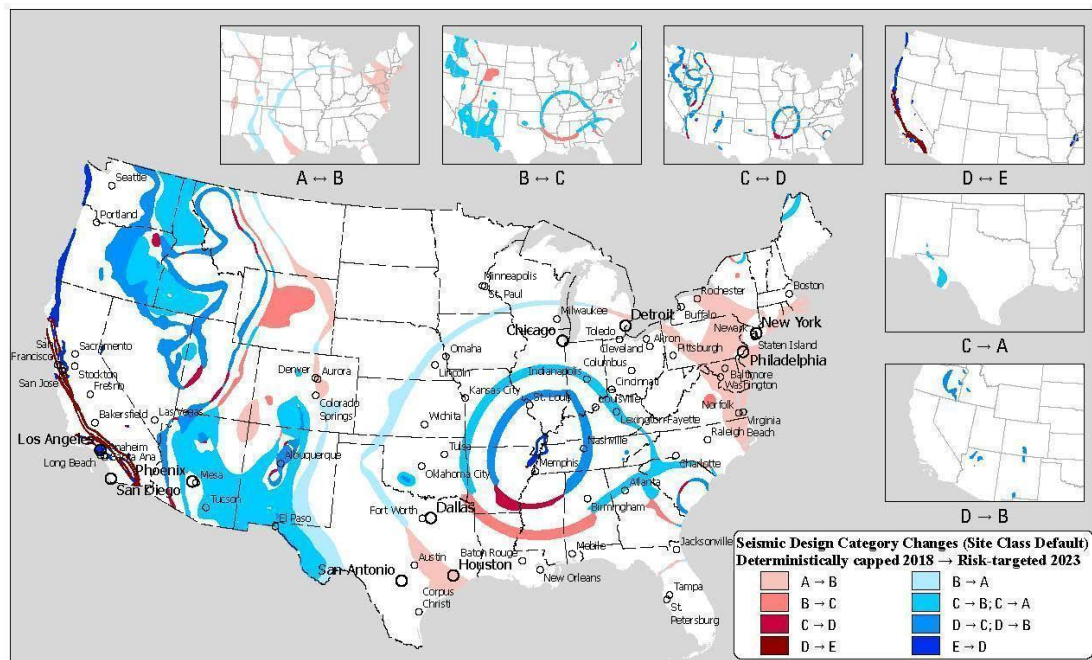


Figure 17. Changes in SDC for default site conditions from the deterministic capping of ASCE 7-22, using the 2018 USGS NSHM, to the increased 1.5%-in-50yrs target risk of collapse in the 2026 NEHRP Provisions, using the 2023 USGS NSHM, with decreases in blue and increases in red. SDC definition is according to ASCE 7-22.

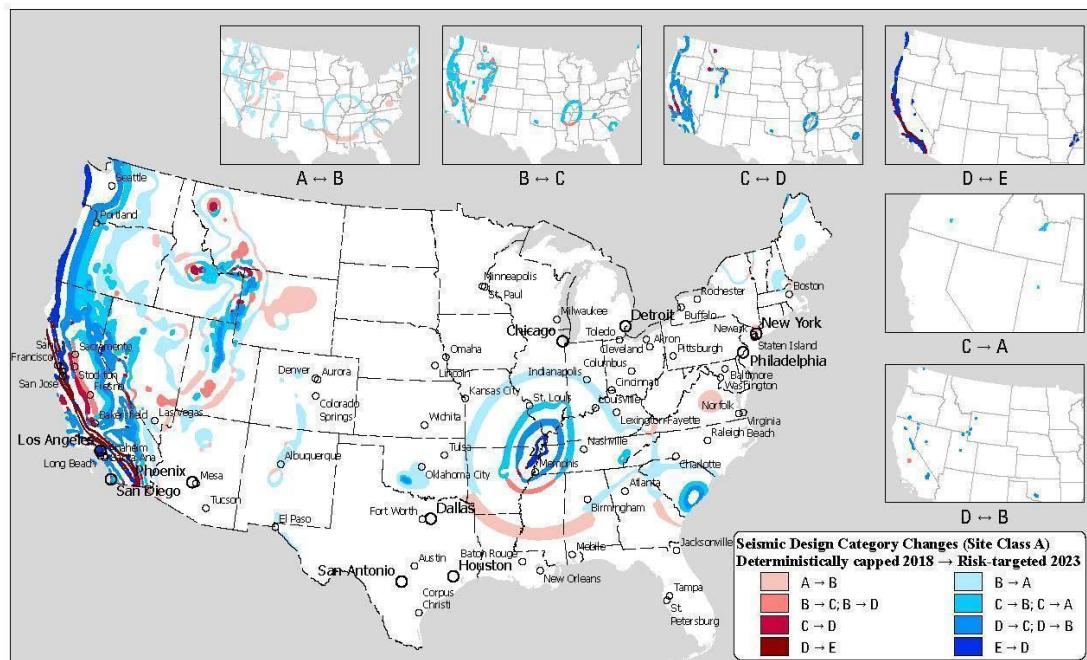


Figure 18. Changes in SDC for Site Class A from the deterministic capping of ASCE 7-22, using the 2018 USGS NSHM, to the increased 1.5%-in-50yrs target risk of collapse in the 2026 NEHRP Provisions, using the 2023 USGS NSHM, with decreases in blue and increases in red. SDC definition is according to ASCE 7-22.

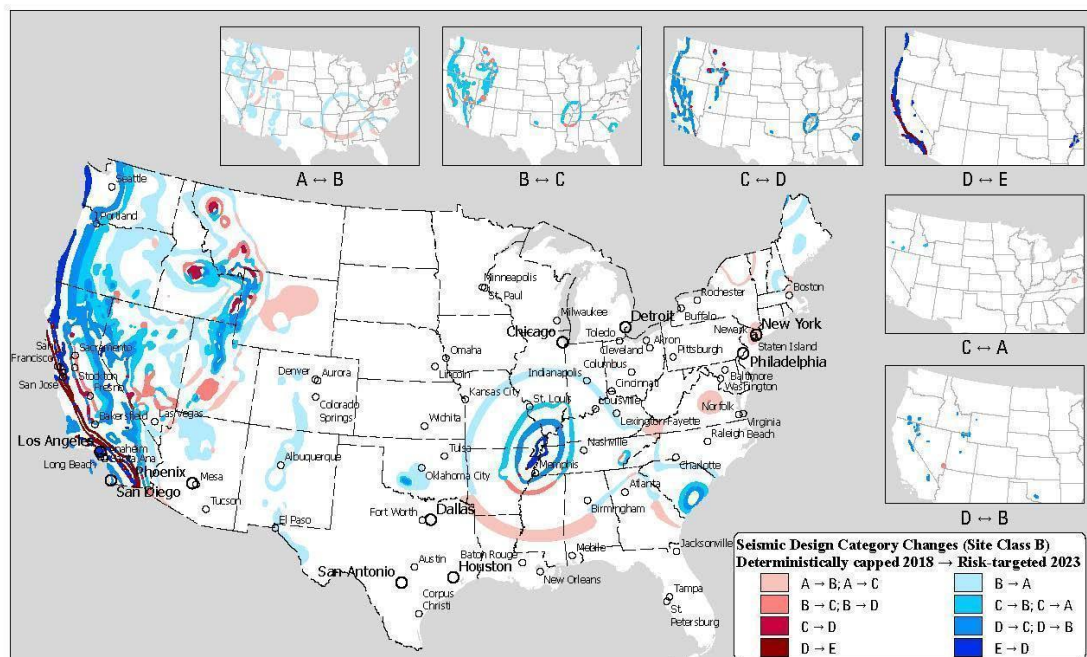


Figure 19. Changes in SDC for Site Class B from the deterministic capping of ASCE 7-22, using the 2018 USGS NSHM, to the increased 1.5%-in-50yrs target risk of collapse in the 2026 NEHRP Provisions, using the 2023 USGS NSHM, with decreases in blue and increases in red. SDC definition is according to ASCE 7-22.

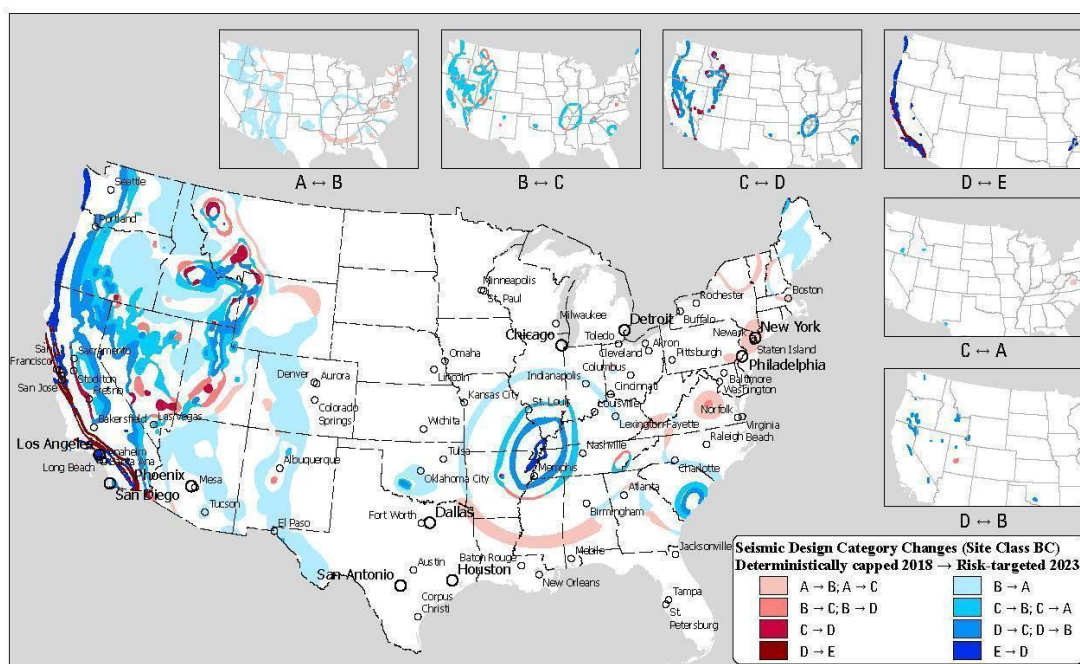


Figure 20. Changes in SDC for Site Class BC from the deterministic capping of ASCE 7-22, using the 2018 USGS NSHM, to the increased 1.5%-in-50yrs target risk of collapse in the 2026 NEHRP Provisions, using the 2023 USGS NSHM, with decreases in blue and increases in red. SDC definition is according to ASCE 7-22.

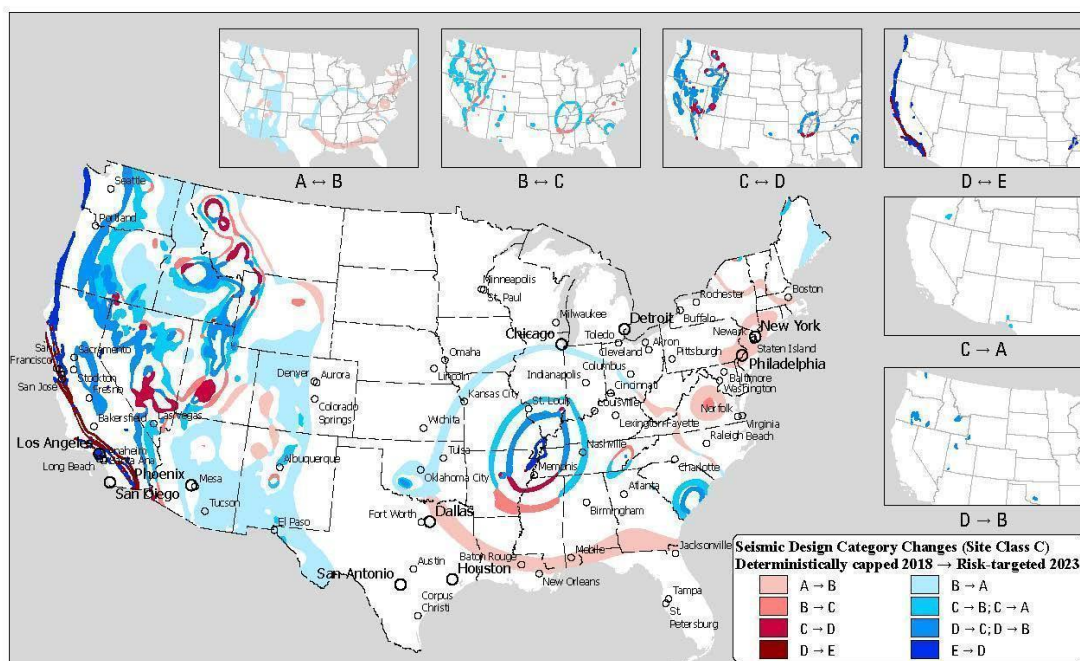


Figure 21. Changes in SDC for Site Class C from the deterministic capping of ASCE 7-22, using the 2018 USGS NSHM, to the increased 1.5%-in-50yrs target risk of collapse in the 2026 NEHRP

Provisions, using the 2023 USGS NSHM, with decreases in blue and increases in red. SDC definition is according to ASCE 7-22.

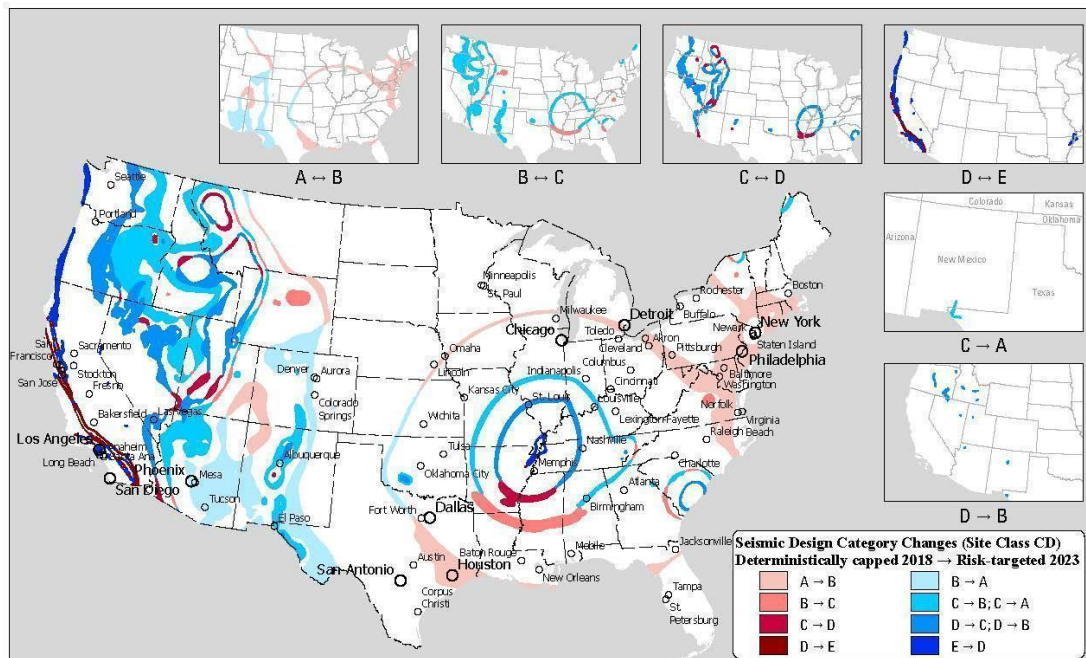


Figure 22. Changes in SDC for Site Class CD from the deterministic capping of ASCE 7-22, using the 2018 USGS NSHM, to the increased 1.5%-in-50yrs target risk of collapse in the 2026 NEHRP Provisions, using the 2023 USGS NSHM, with decreases in blue and increases in red. SDC definition is according to ASCE 7-22.

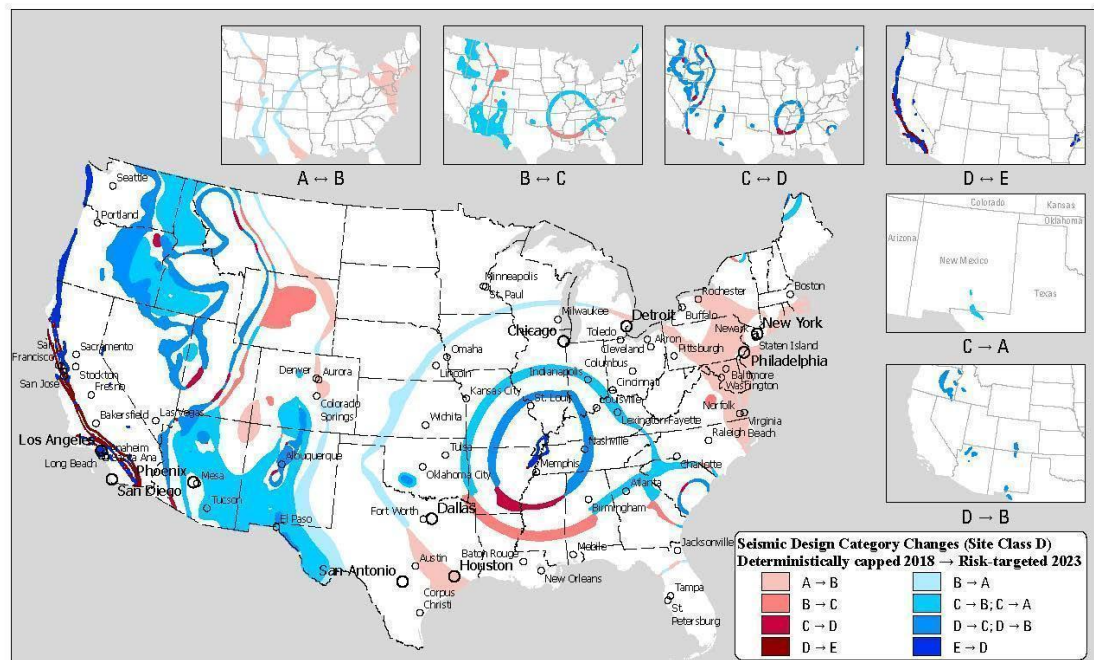


Figure 23. Changes in SDC for Site Class D from the deterministic capping of ASCE 7-22, using the 2018 USGS NSHM, to the increased 1.5%-in-50yrs target risk of collapse in the 2026 NEHRP Provisions, using the 2023 USGS NSHM, with decreases in blue and increases in red. SDC definition is according to ASCE 7-22.

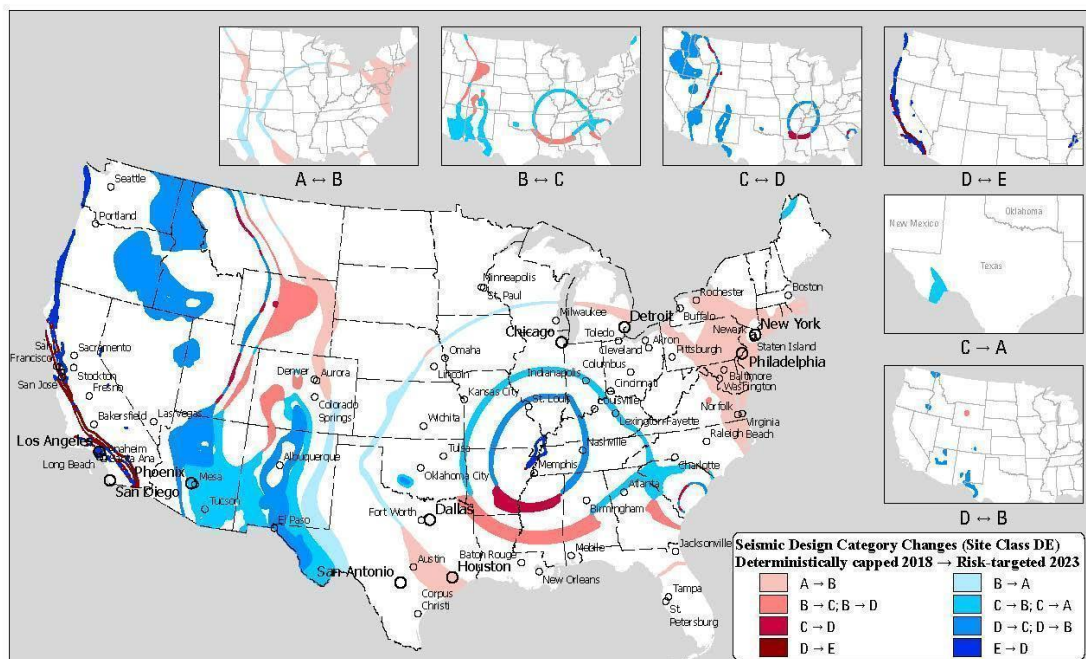


Figure 24.

Changes in SDC for Site Class DE from the deterministic capping of ASCE 7-22, using the 2018 USGS NSHM, to the increased 1.5%-in-50yrs target risk of collapse in the 2026 NEHRP Provisions, using the

2023 USGS NSHM, with decreases in blue and increases in red. SDC definition is according to ASCE 7-22.

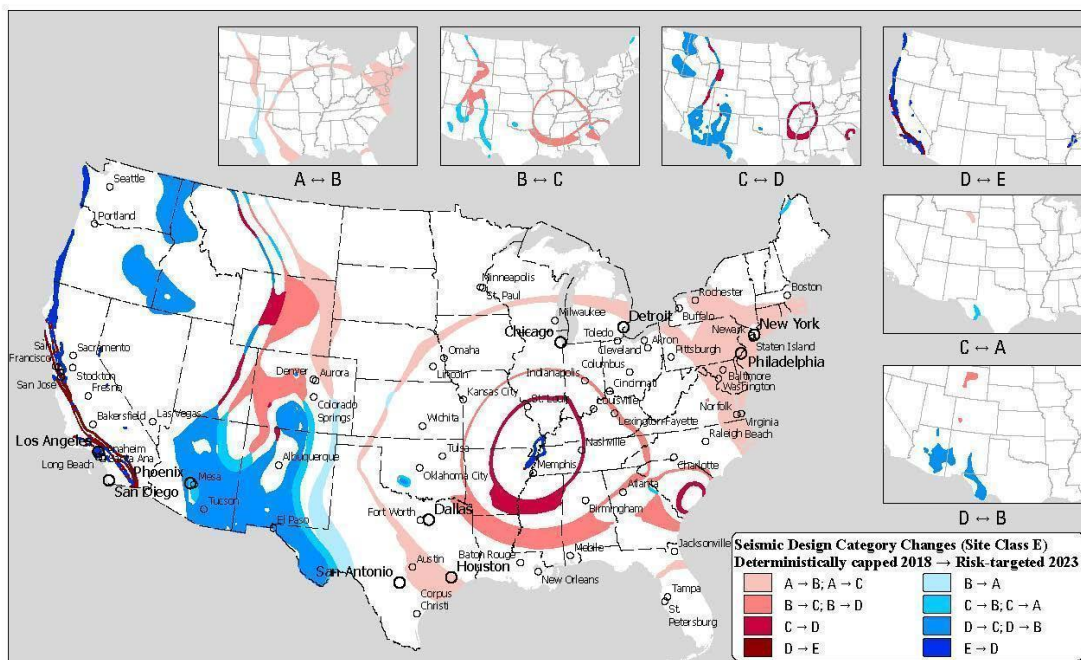


Figure 25. Changes in SDC for Site Class E from the deterministic capping of ASCE 7-22, using the 2018 USGS NSHM, to the increased 1.5%-in-50yrs target risk of collapse in the 2026 NEHRP Provisions, using the 2023 USGS NSHM, with decreases in blue and increases in red. SDC definition is according to ASCE 7-22.

RP3-6.0 Discussion: Comparing the 2026 NEHRP Values to the Intended Values in 1997 NEHRP in High Seismic Areas

For locations in high seismic regions, the 2026 NEHRP Provisions 1.5% in 50 years uniform risk targeted ground motions are remarkably close to the 10% in 50 years uniform hazard (475-year return period) ground motions of the 1997 NEHRP Provisions if they were to be multiplied by 1.5. The multiplication by 1.5 reflects the intention of the 1997 NEHRP Provisions (pages 288 - 293 of commentary) in establishing the definition of the MCE deterministically capped motions at the time. If one only considered the values of design parameters at the high seismic areas of Table 3, the new 1.5% in 50 years uniform risk targeted motions appear to very closely achieve the goals of the 1997 NEHRP Provisions but with the added bonus that they also provide a uniform procedure for determining the ground motions for the entire U.S. Table 3 illustrates this by comparing the design ground motions as described above for default site conditions for the original NEHRP 34 cities using the same hazard model (i.e., the 2023 NSHM). In Table 3, it is shown that the first 24 cities, located in areas of high seismicity, exhibit design ground motion parameter values remarkably close to if 1.5% in 50 years uniform risk was to be used or if 10% in 50 years uniform hazard multiplied by 1.5 were to be used. This exercise does not hold the same conclusions for all other regions

and test sites, but it provides additional confidence that the selection of the 1.5% risk target is appropriate and consistent with past goals of the PUC in high seismic areas.

Table 3. The design ground motion parameters for the original 34 NEHRP cities for default site conditions calculated in two ways: 1) based on definitions of the 2026 NEHRP Provisions (1.5% risk of collapse in 50 years) and 2) 475-year return period uniform hazard ground motions times 1.5. Both versions use the same hazard model from USGS 2023 NSHM.

Site Number	Site Name	S_{MS} (g)		S_{M1} (g)	
		2026 NEHRP	475yr $\times$ 1.5	2026 NEHRP	475yr $\times$ 1.5
26	Los Angeles, CA	2.11	2.13	1.44	1.38
21	Century City, CA	2.04	2.08	1.46	1.4
28	Northridge, CA	2.21	2.34	1.5	1.5
25	Long Beach, CA	1.84	1.85	1.41	1.26
24	Irvine, CA	1.51	1.58	1.01	0.99
32	Riverside, CA	1.97	2.12	1.36	1.42
33	San Bernardino, CA	3.03	3.15	2.54	2.4
35	San Luis Obispo, CA	1.29	1.31	0.91	0.86
34	San Diego, CA	1.72	1.61	1.2	1.05
37	Santa Barbara, CA	2.45	2.32	1.77	1.6
38	Ventura, CA	2.9	2.88	2.29	2.17
5	Oakland, CA	2.59	2.71	1.73	1.76
1	Concord, CA	2.17	2.3	1.45	1.46
17	Monterey, CA	1.75	1.82	1.21	1.16
7	Sacramento, CA	0.87	0.89	0.58	0.6
8	San Francisco, CA	2.1	2.23	1.42	1.44
10	San Mateo, CA	2.43	2.48	1.86	1.76
9	San Jose, CA	2.35	2.51	1.69	1.76
18	Santa Cruz, CA	1.97	2.11	1.35	1.38

12	Vallejo, CA	1.98	2.11	1.4	1.42
11	Santa Rosa, CA	2.95	3.04	2.1	2.11
57	Seattle, WA	1.75	1.77	1.26	1.18
59	Tacoma, WA	1.77	1.77	0.92	0.92
54	Everett, WA	1.4	1.42	0.8	0.76
49	Portland, OR	1.12	0.97	0.77	0.6
93	Salt Lake City, UT	1.46	1.17	1	0.69
117	Boise, ID	0.29	0.27	0.18	0.18
64	Reno, NV	1.51	1.58	0.97	0.92
63	Las Vegas, NV	0.65	0.59	0.38	0.34
137	St Louis, MO	0.63	0.57	0.34	0.26
166	Memphis, TN	1.05	0.86	0.73	0.46
178	Charleston, SC	1.11	0.68	0.64	0.29
119	Chicago, IL	0.19	0.16	0.14	0.11
149	New York, NY	0.37	0.3	0.14	0.1

RP3-7 Electronic Supplement

Additional data can be found through the electronic supplement available at <https://doi.org/10.5066/P14WDBAU>

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RP3-Appendix A: Section 3 data tables for CONUS

Table A1. The 178 CONUS test sites. Regions are approximately defined as northern California (NorCA) for latitudes above 37 degrees in the state of CA, central California (CntCA) for latitudes between 36 and 37 degrees in the state of CA, southern California (SoCA) for latitudes below 36 degrees in the state of CA, Pacific northwest (PNW) for locations in the states of WA and OR, Nevada (NV) for locations in the state of NV, intermountain west (IMW) for longitudes between -115 and -100 degrees, and central and eastern

U.S. (CEUS) for locations east of longitude -100 degrees.

Site Number	Site Name	Latitude (degrees)	Longitude (degrees)	Region
1	Concord, CA	37.95	-122.00	NorCA
2	Eureka, CA	40.80	-124.20	NorCA
3	Mammoth Lakes, CA	37.65	-119.00	NorCA
4	Morgan Hill, CA	37.15	-121.65	NorCA
5	Oakland, CA	37.80	-122.25	NorCA
6	Redding, CA	40.60	-122.40	NorCA
7	Sacramento, CA	38.60	-121.50	NorCA
8	San Francisco, CA	37.75	-122.40	NorCA
9	San Jose, CA	37.35	-121.90	NorCA
10	San Mateo, CA	37.55	-122.30	NorCA
11	Santa Rosa, CA	38.45	-122.70	NorCA
12	Vallejo, CA	38.10	-122.25	NorCA
13	Big Sur, CA	36.25	-121.75	CntCA
14	Coalinga, CA	36.15	-120.40	CntCA
15	Death Valley, CA	36.35	-116.85	CntCA
16	Fresno, CA	36.75	-119.75	CntCA
17	Monterey, CA	36.60	-121.90	CntCA
18	Santa Cruz, CA	36.95	-122.05	CntCA
19	Bakersfield, CA	35.35	-119.35	SoCA
20	Brawley, CA	33.00	-115.55	SoCA
21	Century City, CA	34.05	-118.40	SoCA
22	Cucamonga, CA	34.20	-117.55	SoCA
23	Diablo Canyon, CA	35.20	-120.85	SoCA
24	Irvine, CA	33.65	-117.80	SoCA
25	Long Beach, CA	33.80	-118.20	SoCA
26	Los Angeles, CA	34.05	-118.25	SoCA
27	Malibu West, CA	34.05	-118.95	SoCA
28	Northridge, CA	34.20	-118.55	SoCA
29	Palm Springs, CA	33.85	-116.55	SoCA
30	Palmdale, CA	34.50	-118.00	SoCA
31	Pasadena, CA	34.15	-118.15	SoCA
32	Riverside, CA	33.95	-117.40	SoCA
33	San Bernardino, CA	34.10	-117.30	SoCA
34	San Diego, CA	32.70	-117.15	SoCA
35	San Luis Obispo, CA	35.30	-120.65	SoCA

Site Number	Site Name	Latitude (degrees)	Longitude (degrees)	Region
36	San Onofre, CA	33.40	-117.55	SoCA
37	Santa Barbara, CA	34.45	-119.70	SoCA
38	Ventura, CA	34.30	-119.30	SoCA
39	Astoria, OR	46.20	-123.85	PNW
40	Bend, OR	44.05	-121.30	PNW
41	Brookings, OR	42.05	-124.25	PNW
42	Coos Bay, OR	43.40	-124.20	PNW
43	Eugene, OR	44.05	-123.10	PNW
44	Hillsboro, OR	45.50	-123.00	PNW
45	Klamath Falls, OR	42.20	-121.80	PNW
46	Medford, OR	42.35	-122.90	PNW
47	Newport, OR	44.65	-124.05	PNW
48	Pendleton, OR	45.70	-118.80	PNW
49	Portland, OR	45.50	-122.65	PNW
50	Salem, OR	44.95	-123.05	PNW
51	Aberdeen, WA	47.00	-123.80	PNW
52	Bellingham, WA	48.75	-122.50	PNW
53	Ellensburg, WA	47.00	-120.55	PNW
54	Everett, WA	48.00	-122.20	PNW
55	Hanford Site, WA	46.55	-119.60	PNW
56	Olympia, WA	47.05	-122.90	PNW
57	Seattle, WA	47.60	-122.30	PNW
58	Spokane, WA	47.65	-117.40	PNW
59	Tacoma, WA	47.25	-122.45	PNW
60	Yakima, WA	46.60	-120.50	PNW
61	Carson City, NV	39.15	-119.75	NV
62	Elko, NV	40.85	-115.75	NV
63	Las Vegas, NV	36.20	-115.15	NV
64	Reno, NV	39.55	-119.80	NV
65	Grand Canyon, Village, AZ	36.05	-112.15	IMW
66	Palo Verde, AZ	33.40	-112.85	IMW
67	Phoenix, AZ	33.45	-112.10	IMW
68	Tucson, AZ	32.20	-110.95	IMW
69	Denver, CO	39.75	-105.00	IMW
70	La Junta, CO	38.00	-103.55	IMW
71	Paradox, CO	38.40	-108.95	IMW
72	Rangely, CO	40.10	-108.80	IMW
73	Trinidad, CO	37.20	-104.50	IMW
74	Idaho National Lab, ID	43.60	-112.85	IMW
75	Billings, MT	45.80	-108.50	IMW
76	Bozeman, MT	45.70	-111.05	IMW
77	Lima, MT	44.65	-112.60	IMW
78	Missoula, MT	46.90	-114.00	IMW
79	Albuquerque, NM	35.10	-106.60	IMW
80	Artesia, NM	32.85	-104.40	IMW

Site Number	Site Name	Latitude (degrees)	Longitude (degrees)	Region
81	Crownpoint, NM	35.70	-108.15	IMW
82	Las Cruces, NM	32.30	-106.75	IMW
83	Los Alamos National Lab, NM	35.85	-106.30	IMW
84	Edgemont, SD	43.30	-103.85	IMW
85	Amarillo, TX	35.20	-101.85	IMW
86	Kermit, TX	31.85	-103.10	IMW
87	Snyder, TX	32.70	-100.90	IMW
88	Angle, UT	38.25	-111.95	IMW
89	Brigham City, UT	41.50	-112.00	IMW
90	Cedar City, UT	37.70	-113.05	IMW
91	Green River, UT	39.00	-110.15	IMW
92	Provo, UT	40.25	-111.65	IMW
93	Salt Lake City, UT	40.75	-111.90	IMW
94	Casper, WY	42.85	-106.30	IMW
95	Cheyenne, WY	41.15	-104.80	IMW
96	Jackson, WY	43.50	-110.75	IMW
97	Kaycee, WY	43.70	-106.65	IMW
98	Sunday Peak, WY	43.50	-110.20	IMW
99	Yellowstone, WY	44.40	-110.55	IMW
100	Atmore, AL	31.00	-87.50	CEUS
101	Birmingham, AL	33.50	-86.80	CEUS
102	El Dorado, AR	33.20	-92.70	CEUS
103	Enola, AR	35.20	-92.20	CEUS
104	Greenbrier, AR	35.20	-92.40	CEUS
105	Little Rock, AR	34.75	-92.30	CEUS
106	Hartford, CT	41.75	-72.70	CEUS
107	Washington, DC	38.90	-77.05	CEUS
108	Wilmington, DE	39.75	-75.55	CEUS
109	Jacksonville, FL	30.35	-81.65	CEUS
110	Miami, FL	25.75	-80.20	CEUS
111	Orlando, FL	28.55	-81.40	CEUS
112	Tampa, FL	27.95	-82.45	CEUS
113	Atlanta, GA	33.75	-84.40	CEUS
114	Lincolnton, GA	33.80	-82.50	CEUS
115	Savannah, GA	32.10	-81.10	CEUS
116	Des Moines, IA	41.60	-93.60	CEUS
117	Boise, ID	43.60	-116.20	CEUS
118	Central, IL	40.00	-90.00	CEUS
119	Chicago, IL	41.85	-87.65	CEUS
120	Evansville, IN	38.00	-87.60	CEUS
121	Indianapolis, IN	39.80	-86.15	CEUS
122	Topeka, KS	39.05	-95.70	CEUS
123	Wichita, KS	37.70	-97.35	CEUS
124	Louisville, KY	38.25	-85.75	CEUS
125	New Orleans, LA	29.95	-90.05	CEUS

Site Number	Site Name	Latitude (degrees)	Longitude (degrees)	Region
126	Boston, MA	42.35	-71.05	CEUS
127	Baltimore, MD	39.30	-76.60	CEUS
128	Augusta, ME	44.30	-69.80	CEUS
129	Bangor, ME	44.80	-68.80	CEUS
130	Portland, ME	43.65	-70.25	CEUS
131	Detroit, MI	42.35	-83.05	CEUS
132	Grand Rapids, MI	42.95	-85.65	CEUS
133	Minneapolis, MN	45.00	-93.30	CEUS
134	Cape Girardeau, MO	37.30	-89.50	CEUS
135	Kansas City, MO	39.10	-94.60	CEUS
136	New Madrid, MO	36.60	-89.55	CEUS
137	St Louis, MO	38.60	-90.20	CEUS
138	Jackson, MS	32.30	-90.20	CEUS
139	Asheville, NC	35.60	-82.55	CEUS
140	Charlotte, NC	35.25	-80.85	CEUS
141	Greenville, NC	35.60	-77.35	CEUS
142	Raleigh, NC	35.80	-78.65	CEUS
143	Fargo, ND	46.90	-96.80	CEUS
144	Omaha, NE	41.25	-96.00	CEUS
145	Manchester, NH	43.00	-71.45	CEUS
146	Trenton, NJ	40.20	-74.75	CEUS
147	Batavia, NY	43.00	-78.20	CEUS
148	Malone, NY	44.85	-74.30	CEUS
149	New York, NY	40.75	-74.00	CEUS
150	Cincinnati, OH	39.15	-84.50	CEUS
151	Cleveland, OH	41.50	-81.70	CEUS
152	Columbus, OH	39.95	-83.00	CEUS
153	Sidney, OH	40.30	-84.15	CEUS
154	Youngstown, OH	41.10	-80.65	CEUS
155	Elgin, OK	34.80	-98.30	CEUS
156	Oklahoma City, OK	35.50	-97.50	CEUS
157	Philadelphia, PA	39.95	-75.15	CEUS
158	Pittsburg, PA	40.45	-80.00	CEUS
159	Providence, RI	41.80	-71.40	CEUS
160	Charleston, SC	32.80	-79.95	CEUS
161	Platte, SD	43.40	-98.85	CEUS
162	Sioux Falls, SD	43.55	-96.75	CEUS
163	Chattanooga, TN	35.05	-85.25	CEUS
164	Knoxville, TN	35.95	-83.90	CEUS
165	Maryville, TN	35.75	-84.00	CEUS
166	Memphis, TN	35.15	-90.05	CEUS
167	Nashville, TN	36.15	-86.80	CEUS
168	Austin, TX	30.25	-97.75	CEUS
169	Dallas, TX	32.80	-96.80	CEUS
170	Houston, TX	29.75	-95.35	CEUS

Site Number	Site Name	Latitude (degrees)	Longitude (degrees)	Region
171	San Antonio, TX	29.40	-98.50	CEUS
172	Blacksburg, VA	37.25	-80.40	CEUS
173	Michaux, VA	37.65	-77.90	CEUS
174	Richmond, VA	37.55	-77.45	CEUS
175	Virginia Beach, VA	36.85	-76.00	CEUS
176	Burlington, VT	44.50	-73.20	CEUS
177	Milwaukee, WI	43.05	-87.90	CEUS
178	Charleston, WV	38.35	-81.65	CEUS

Table A2. Comparisons between 2026 NEHRP and previous ASCE 7 Standards for S_s and S_1 (reference site class, B or BC depending on the edition, at 0.2 and 1.0 Sec) values for the 178 CONUS test sites. All values are rounded to two decimal places.

Site Name	S_s (g)					S_1 (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Concord, CA	1.85	2.08	2.22	2.76	2.01	0.65	0.74	0.67	0.91	0.67
Eureka, CA	2.53	2.97	2.81	3.42	3.19	1.04	1.19	1.09	1.20	0.96
Mammoth Lakes, CA	1.62	1.53	1.52	2.05	1.86	0.56	0.49	0.49	0.58	0.51
Morgan Hill, CA	1.50	1.64	1.77	1.96	2.38	0.60	0.60	0.65	0.68	0.77
Oakland, CA	1.60	1.86	1.88	2.07	2.46	0.60	0.75	0.72	0.72	0.81
Redding, CA	0.74	0.70	1.05	1.26	0.85	0.28	0.32	0.44	0.43	0.31
Sacramento, CA	0.59	0.67	0.57	0.66	0.57	0.24	0.29	0.25	0.24	0.26
San Francisco, CA	1.50	1.50	1.50	1.57	1.91	0.68	0.64	0.60	0.60	0.64
San Jose, CA	1.50	1.50	1.50	1.50	2.11	0.60	0.60	0.60	0.61	0.81
San Mateo, CA	1.75	1.85	1.80	2.10	2.26	0.90	0.86	0.74	0.81	0.85
Santa Rosa, CA	2.05	2.51	2.41	2.81	2.83	0.83	1.04	0.94	1.15	1.05
Vallejo, CA	1.50	1.50	1.50	2.33	1.76	0.60	0.60	0.60	0.89	0.67
Big Sur, CA	1.25	1.21	1.27	1.43	1.32	0.53	0.46	0.48	0.48	0.42
Coalinga, CA	1.50	2.02	1.79	2.09	2.01	0.54	0.69	0.60	0.60	0.60
Death Valley, CA	1.70	1.83	1.07	1.32	1.18	0.75	0.69	0.42	0.44	0.39
Fresno, CA	0.51	0.63	0.58	0.70	0.73	0.22	0.26	0.23	0.23	0.26
Monterey, CA	1.46	1.53	1.33	1.50	1.53	0.61	0.56	0.50	0.49	0.49
Santa Cruz, CA	1.50	1.52	1.59	1.65	1.74	0.60	0.60	0.60	0.59	0.60
Bakersfield, CA	1.09	1.09	0.98	1.13	1.05	0.47	0.43	0.37	0.37	0.40
Brawley, CA	1.50	1.50	1.50	1.78	2.93	0.60	0.60	0.60	0.71	1.01
Century City, CA	1.83	2.17	2.11	2.37	1.87	0.72	0.80	0.75	0.92	0.72
Cucamonga, CA	2.43	3.39	2.28	2.53	2.52	0.89	1.09	0.84	0.90	0.87
Diablo Canyon, CA	1.64	1.41	1.17	1.31	1.11	0.63	0.52	0.42	0.42	0.38
Irvine, CA	1.50	1.55	1.25	1.43	1.23	0.53	0.57	0.45	0.45	0.40
Long Beach, CA	1.80	1.64	1.68	1.90	1.69	0.70	0.62	0.61	0.75	0.66
Los Angeles, CA	2.16	2.40	1.97	2.25	1.97	0.72	0.84	0.70	0.72	0.67
Malibu West, CA	2.33	2.28	1.52	1.74	1.63	0.95	0.78	0.53	0.54	0.54
Northridge, CA	1.63	1.69	1.74	2.09	2.03	0.61	0.60	0.60	0.70	0.68
Palm Springs, CA	1.50	1.99	1.91	2.10	2.21	0.60	0.96	0.80	0.79	0.74
Palmdale, CA	2.21	2.64	2.50	2.88	3.41	1.22	1.31	1.07	1.18	1.30
Pasadena, CA	2.50	2.88	2.09	2.36	2.16	0.86	1.00	0.76	0.76	0.71

Site Name	S _s (g)					S ₁ (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Riverside, CA	1.50	1.50	1.50	1.50	1.67	0.60	0.60	0.58	0.58	0.54
San Bernardino, CA	1.70	2.37	2.33	2.78	2.93	0.62	1.08	0.93	1.07	1.04
San Diego, CA	1.61	1.25	1.58	1.74	1.68	0.64	0.48	0.53	0.53	0.51
San Luis, Obispo, CA	1.23	1.12	1.09	1.23	1.03	0.47	0.43	0.40	0.40	0.36
San Onofre, CA	1.21	1.16	1.05	1.20	1.02	0.45	0.45	0.39	0.39	0.34
Santa Barbara, CA	2.07	2.83	2.12	2.37	2.41	0.81	0.99	0.78	0.87	0.78
Ventura, CA	2.46	2.38	2.02	2.25	2.86	0.86	0.90	0.76	1.01	1.15
Astoria, OR	1.33	1.31	1.28	1.65	1.44	0.66	0.67	0.67	0.70	0.50
Bend, OR	0.39	0.40	0.38	0.41	0.25	0.16	0.21	0.20	0.16	0.12
Brookings, OR	1.50	1.49	2.02	2.62	2.45	0.72	0.71	0.95	1.02	0.74
Coos Bay, OR	1.50	1.38	1.56	2.03	1.85	0.70	0.70	0.80	0.84	0.61
Eugene, OR	0.68	0.78	0.71	0.84	0.65	0.33	0.41	0.40	0.38	0.28
Hillsboro, OR	0.90	0.99	0.89	0.98	0.91	0.35	0.45	0.43	0.40	0.42
Klamath Falls, OR	0.89	0.90	1.07	1.21	0.75	0.35	0.35	0.40	0.39	0.25
Medford, OR	0.58	0.62	0.64	0.75	0.58	0.27	0.33	0.37	0.34	0.25
Newport, OR	1.82	1.74	1.68	2.09	1.98	0.69	0.76	0.78	0.80	0.63
Pendleton, OR	0.40	0.35	0.37	0.37	0.27	0.12	0.13	0.14	0.11	0.08
Portland, OR	0.99	0.98	0.89	0.94	1.01	0.34	0.42	0.39	0.36	0.34
Salem, OR	0.80	0.93	0.83	0.96	0.82	0.34	0.44	0.42	0.39	0.30
Aberdeen, WA	1.43	1.47	1.53	1.89	1.78	0.65	0.70	0.69	0.71	0.54
Bellingham, WA	0.99	0.96	1.01	1.12	1.07	0.33	0.38	0.35	0.32	0.29
Ellensburg, WA	0.54	0.53	0.50	0.54	0.52	0.18	0.22	0.21	0.17	0.16
Everett, WA	1.15	1.27	1.20	1.36	1.26	0.40	0.48	0.43	0.47	0.42
Hanford, Site, WA	0.46	0.42	0.41	0.45	0.40	0.15	0.17	0.16	0.14	0.12
Olympia, WA	1.16	1.33	1.41	1.50	1.69	0.44	0.55	0.52	0.50	0.48
Seattle, WA	1.45	1.37	1.40	1.56	1.64	0.49	0.53	0.49	0.66	0.66
Spokane, WA	0.40	0.33	0.31	0.32	0.19	0.11	0.12	0.11	0.09	0.06
Tacoma, WA	1.21	1.30	1.36	1.50	1.63	0.42	0.51	0.47	0.46	0.49
Yakima, WA	0.52	0.51	0.47	0.51	0.52	0.18	0.21	0.20	0.16	0.16
Carson City, NV	1.66	2.28	2.01	2.33	1.64	0.60	0.79	0.73	0.74	0.49
Elko, NV	0.49	0.52	0.49	0.52	0.43	0.17	0.16	0.16	0.15	0.12
Las Vegas, NV	0.57	0.50	0.65	0.70	0.47	0.18	0.17	0.21	0.20	0.14
Reno, NV	1.50	1.50	1.47	1.54	1.33	0.59	0.52	0.52	0.54	0.39
Grand Canyon Village, AZ	0.60	0.46	0.37	0.43	0.24	0.17	0.13	0.11	0.11	0.07
Palo Verde, AZ	0.17	0.15	0.17	0.16	0.09	0.06	0.05	0.08	0.08	0.07
Phoenix, AZ	0.18	0.17	0.18	0.18	0.10	0.06	0.06	0.06	0.06	0.04

Site Name	S _s (g)					S ₁ (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Tucson, AZ	0.29	0.27	0.27	0.29	0.13	0.08	0.08	0.08	0.08	0.04
Denver, CO	0.22	0.18	0.21	0.19	0.15	0.06	0.06	0.06	0.04	0.04
La Junta, CO	0.19	0.19	0.17	0.17	0.13	0.06	0.07	0.06	0.05	0.04
Paradox, CO	0.24	0.19	0.20	0.18	0.20	0.06	0.06	0.06	0.04	0.05
Rangely, CO	0.30	0.26	0.31	0.28	0.20	0.07	0.08	0.08	0.06	0.05
Trinidad, CO	0.24	0.33	0.23	0.17	0.16	0.07	0.09	0.07	0.05	0.05
Idaho National Lab, ID	0.33	0.33	0.28	0.30	0.22	0.13	0.13	0.12	0.11	0.09
Billings, MT	0.14	0.12	0.13	0.13	0.13	0.05	0.05	0.06	0.05	0.05
Bozeman, MT	0.75	0.72	0.70	0.66	0.74	0.23	0.21	0.22	0.18	0.20
Lima, MT	1.14	1.34	1.19	1.26	0.83	0.40	0.40	0.39	0.37	0.22
Missoula, MT	0.54	0.50	0.44	0.46	0.43	0.17	0.15	0.15	0.13	0.12
Albuquerque, NM	0.56	0.45	0.41	0.42	0.37	0.17	0.13	0.13	0.12	0.10
Artesia, NM	0.16	0.11	0.09	0.09	0.06	0.05	0.04	0.04	0.04	0.03
Crownpoint, NM	0.27	0.22	0.18	0.17	0.27	0.07	0.07	0.06	0.05	0.06
Las Cruces, NM	0.31	0.29	0.28	0.30	0.17	0.10	0.09	0.09	0.09	0.06
Los Alamos National Lab, NM	0.48	0.56	0.52	0.55	0.37	0.16	0.17	0.17	0.16	0.10
Edgemont, SD	0.21	0.16	0.22	0.20	0.18	0.05	0.05	0.05	0.04	0.04
Amarillo, TX	0.18	0.15	0.20	0.19	0.19	0.04	0.05	0.05	0.04	0.04
Kermit, TX	0.25	0.20	0.18	0.12	0.09	0.05	0.05	0.04	0.03	0.02
Snyder, TX	0.10	0.07	0.07	0.08	0.07	0.03	0.03	0.03	0.03	0.03
Angle, UT	0.69	0.59	0.72	0.56	0.78	0.20	0.17	0.22	0.15	0.18
Brigham City, UT	1.44	1.47	1.40	1.50	1.09	0.57	0.54	0.50	0.53	0.39
Cedar City, UT	0.72	0.68	0.76	0.67	0.59	0.23	0.21	0.25	0.20	0.16
Green River, UT	0.27	0.28	0.33	0.28	0.31	0.08	0.09	0.09	0.07	0.08
Provo, UT	1.27	1.21	1.39	1.51	1.04	0.54	0.45	0.52	0.57	0.37
Salt Lake City, UT	1.72	1.54	1.55	1.76	1.39	0.70	0.56	0.56	0.62	0.48
Casper, WY	0.37	0.28	0.31	0.29	0.29	0.08	0.07	0.07	0.05	0.07
Cheyenne, WY	0.19	0.16	0.16	0.17	0.14	0.05	0.05	0.05	0.04	0.04
Jackson, WY	1.14	1.32	1.04	1.16	0.98	0.42	0.40	0.35	0.34	0.27
Kaycee, WY	0.37	0.29	0.33	0.30	0.50	0.08	0.07	0.07	0.05	0.11
Sunday Peak, WY	0.71	0.73	0.89	0.73	0.95	0.21	0.22	0.26	0.20	0.22
Yellowstone, WY	1.24	1.29	1.24	1.29	1.04	0.41	0.38	0.41	0.38	0.28
Atmore, AL	0.13	0.12	0.10	0.13	0.12	0.06	0.06	0.06	0.07	0.07
Birmingham, AL	0.30	0.26	0.30	0.33	0.30	0.10	0.11	0.10	0.12	0.10
El Dorado, AR	0.23	0.19	0.16	0.25	0.25	0.10	0.10	0.09	0.11	0.11

Site Name	S _s (g)					S ₁ (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Enola, AR	0.62	0.49	0.78	0.84	0.96	0.19	0.19	0.19	0.20	0.20
Greenbrier, AR	0.56	0.44	0.53	0.64	0.66	0.17	0.17	0.17	0.18	0.17
Little Rock, AR	0.50	0.40	0.39	0.54	0.50	0.16	0.16	0.15	0.17	0.15
Hartford, CT	0.24	0.18	0.19	0.21	0.25	0.06	0.06	0.06	0.05	0.06
Washington, DC	0.15	0.12	0.13	0.15	0.22	0.05	0.05	0.04	0.04	0.06
Wilmington, DE	0.26	0.20	0.18	0.23	0.28	0.06	0.06	0.05	0.05	0.06
Jacksonville, FL	0.14	0.11	0.11	0.16	0.13	0.06	0.06	0.06	0.06	0.06
Miami, FL	0.05	0.04	0.04	0.05	0.04	0.02	0.02	0.02	0.02	0.02
Orlando, FL	0.10	0.08	0.06	0.09	0.07	0.04	0.04	0.03	0.04	0.04
Tampa, FL	0.08	0.06	0.05	0.07	0.06	0.03	0.03	0.03	0.03	0.03
Atlanta, GA	0.23	0.19	0.18	0.24	0.25	0.09	0.09	0.09	0.09	0.09
Lincolnton, GA	0.34	0.28	0.47	0.49	0.37	0.11	0.11	0.11	0.11	0.10
Savannah, GA	0.41	0.31	0.32	0.46	0.36	0.12	0.12	0.11	0.12	0.12
Des Moines, IA	0.08	0.07	0.06	0.09	0.08	0.04	0.05	0.05	0.06	0.05
Boise, ID	0.31	0.31	0.31	0.33	0.16	0.10	0.11	0.11	0.10	0.06
Central, IL	0.23	0.18	0.17	0.27	0.25	0.10	0.10	0.09	0.11	0.10
Chicago, IL	0.17	0.14	0.12	0.15	0.15	0.06	0.06	0.06	0.07	0.06
Evansville, IN	0.67	0.58	0.55	0.70	0.66	0.19	0.20	0.18	0.20	0.18
Indianapolis, IN	0.19	0.16	0.18	0.26	0.24	0.08	0.09	0.08	0.10	0.09
Topeka, KS	0.16	0.13	0.10	0.13	0.12	0.05	0.06	0.06	0.07	0.06
Wichita, KS	0.14	0.11	0.09	0.12	0.10	0.05	0.05	0.06	0.06	0.05
Louisville, KY	0.25	0.21	0.20	0.31	0.28	0.10	0.11	0.10	0.12	0.11
New Orleans, LA	0.11	0.10	0.09	0.11	0.10	0.05	0.05	0.05	0.06	0.05
Boston, MA	0.28	0.22	0.27	0.26	0.29	0.07	0.07	0.07	0.06	0.06
Baltimore, MD	0.17	0.13	0.14	0.16	0.22	0.05	0.05	0.04	0.04	0.06
Augusta, ME	0.29	0.23	0.32	0.31	0.28	0.08	0.08	0.08	0.07	0.07
Bangor, ME	0.26	0.21	0.30	0.31	0.25	0.07	0.08	0.08	0.07	0.07
Portland, ME	0.31	0.24	0.28	0.28	0.28	0.08	0.08	0.07	0.06	0.07
Detroit, MI	0.12	0.10	0.10	0.11	0.12	0.05	0.05	0.05	0.05	0.05
Grand Rapids, MI	0.09	0.07	0.08	0.11	0.10	0.04	0.04	0.05	0.05	0.05
Minneapolis, MN	0.06	0.05	0.05	0.05	0.04	0.03	0.03	0.03	0.03	0.03
Cape Girardeau, MO	1.50	1.31	1.18	1.50	1.25	0.56	0.46	0.41	0.41	0.34
Kansas City, MO	0.13	0.11	0.10	0.14	0.13	0.06	0.06	0.07	0.08	0.07
New Madrid, MO	3.34	2.91	3.20	4.25	3.52	1.30	1.15	1.28	1.13	1.07
St Louis, MO	0.59	0.44	0.46	0.63	0.59	0.17	0.17	0.16	0.18	0.16
Jackson, MS	0.19	0.16	0.14	0.23	0.20	0.08	0.09	0.08	0.10	0.09

Site Name	S _s (g)					S ₁ (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Asheville, NC	0.39	0.31	0.30	0.36	0.36	0.11	0.11	0.09	0.10	0.10
Charlotte, NC	0.31	0.24	0.20	0.27	0.26	0.11	0.10	0.08	0.09	0.08
Greenville, NC	0.16	0.12	0.10	0.14	0.14	0.07	0.06	0.05	0.05	0.06
Raleigh, NC	0.20	0.15	0.12	0.17	0.17	0.08	0.08	0.06	0.07	0.06
Fargo, ND	0.07	0.05	0.06	0.06	0.05	0.02	0.02	0.02	0.02	0.01
Omaha, NE	0.12	0.10	0.07	0.09	0.08	0.04	0.05	0.05	0.05	0.05
Manchester, NH	0.35	0.27	0.43	0.39	0.34	0.08	0.08	0.09	0.07	0.07
Trenton, NJ	0.29	0.22	0.21	0.22	0.29	0.06	0.06	0.05	0.05	0.06
Batavia, NY	0.27	0.21	0.18	0.19	0.16	0.06	0.06	0.05	0.04	0.05
Malone, NY	0.63	0.54	0.52	0.49	0.50	0.13	0.14	0.11	0.09	0.10
New York, NY	0.36	0.28	0.29	0.28	0.34	0.07	0.07	0.06	0.05	0.07
Cincinnati, OH	0.18	0.14	0.15	0.22	0.20	0.08	0.08	0.08	0.09	0.08
Cleveland, OH	0.19	0.17	0.14	0.14	0.14	0.05	0.06	0.05	0.05	0.05
Columbus, OH	0.15	0.12	0.12	0.16	0.15	0.06	0.06	0.06	0.07	0.06
Sidney, OH	0.26	0.21	0.36	0.36	0.34	0.07	0.07	0.08	0.08	0.08
Youngstown, OH	0.15	0.16	0.11	0.12	0.12	0.05	0.06	0.05	0.05	0.05
Elgin, OK	0.43	0.46	0.82	0.89	0.57	0.09	0.13	0.22	0.19	0.13
Oklahoma City, OK	0.33	0.26	0.31	0.34	0.22	0.07	0.08	0.08	0.08	0.07
Philadelphia, PA	0.27	0.20	0.18	0.21	0.27	0.06	0.06	0.05	0.05	0.06
Pittsburg, PA	0.13	0.11	0.09	0.11	0.12	0.05	0.05	0.05	0.05	0.05
Providence, RI	0.23	0.18	0.20	0.21	0.24	0.06	0.06	0.06	0.05	0.06
Charleston, SC	1.51	1.15	1.42	1.66	1.21	0.37	0.37	0.41	0.38	0.33
Platte, SD	0.18	0.15	0.18	0.17	0.12	0.04	0.04	0.04	0.03	0.03
Sioux Falls, SD	0.11	0.09	0.09	0.09	0.09	0.03	0.04	0.04	0.04	0.03
Chattanooga, TN	0.47	0.38	0.48	0.51	0.48	0.12	0.13	0.12	0.13	0.12
Knoxville, TN	0.52	0.42	0.64	0.57	0.59	0.12	0.13	0.14	0.13	0.13
Maryville, TN	0.53	0.43	0.92	0.81	0.67	0.12	0.13	0.16	0.14	0.14
Memphis, TN	1.40	1.01	1.03	1.32	1.10	0.38	0.35	0.35	0.35	0.34
Nashville, TN	0.35	0.30	0.28	0.47	0.40	0.13	0.15	0.14	0.17	0.14
Austin, TX	0.08	0.06	0.05	0.06	0.06	0.03	0.03	0.03	0.04	0.04
Dallas, TX	0.12	0.10	0.10	0.13	0.11	0.05	0.05	0.05	0.06	0.06
Houston, TX	0.09	0.07	0.07	0.08	0.07	0.04	0.04	0.04	0.04	0.04
San Antonio, TX	0.11	0.08	0.05	0.06	0.06	0.03	0.03	0.02	0.02	0.03
Blacksburg, VA	0.31	0.24	0.23	0.29	0.27	0.08	0.08	0.07	0.08	0.08
Michaux, VA	0.27	0.24	0.37	0.35	0.56	0.07	0.07	0.07	0.06	0.10
Richmond, VA	0.22	0.19	0.24	0.25	0.38	0.06	0.06	0.06	0.05	0.08

Site Name	S _s (g)					S ₁ (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Virginia Beach, VA	0.11	0.09	0.08	0.10	0.13	0.05	0.05	0.04	0.04	0.05
Burlington, VT	0.38	0.36	0.35	0.35	0.36	0.10	0.11	0.09	0.07	0.08
Milwaukee, WI	0.11	0.09	0.08	0.10	0.09	0.04	0.05	0.05	0.05	0.05
Charleston, WV	0.19	0.15	0.15	0.20	0.20	0.07	0.07	0.06	0.07	0.07

Table A3. Comparisons between 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} (default site conditions as defined in each edition: in 2026 NEHRP and ASCE 7-22, “Default” is the maximum of C, CD, and D; in ASCE 7-16, “D-default” is per Section 11.4.4 equivalent to the greater of C and D; in ASCE 7-10 and ASCE -05, default is D) values for the 178 CONUS test sites. Where values are listed as NaN, in ASCE 7-16, site-specific procedures were required due to known problems with the F_v parameter. All values are rounded to two decimal places.

Site Name	S_{MS} (g)					S_{M1} (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Concord, CA	1.85	2.08	2.67	2.84	2.17	0.98	1.10	NaN	1.91	1.45
Eureka, CA	2.53	2.97	3.38	3.65	3.12	1.56	1.79	NaN	2.47	2.00
Mammoth Lakes, CA	1.62	1.53	1.82	2.22	2.05	0.84	0.74	NaN	1.41	1.25
Morgan Hill, CA	1.50	1.64	2.13	2.14	2.57	0.90	0.90	NaN	1.70	1.68
Oakland, CA	1.60	1.86	2.26	2.21	2.59	0.90	1.12	NaN	1.70	1.73
Redding, CA	0.90	0.87	1.26	1.52	1.10	0.51	0.56	NaN	0.97	0.73
Sacramento, CA	0.78	0.85	0.76	0.97	0.87	0.47	0.53	NaN	0.63	0.58
San Francisco, CA	1.50	1.50	1.80	1.81	2.10	1.02	0.96	NaN	1.75	1.42
San Jose, CA	1.50	1.50	1.80	1.65	2.35	0.90	0.90	NaN	1.70	1.69
San Mateo, CA	1.75	1.85	2.16	2.30	2.43	1.34	1.29	NaN	2.30	1.86
Santa Rosa, CA	2.05	2.51	2.90	2.93	2.95	1.25	1.55	NaN	2.40	2.10
Vallejo, CA	1.50	1.50	1.80	2.42	1.98	0.90	0.90	NaN	1.80	1.40
Big Sur, CA	1.25	1.23	1.53	1.56	1.52	0.80	0.70	NaN	1.14	1.04
Coalinga, CA	1.50	2.02	2.14	2.32	2.23	0.81	1.04	NaN	1.48	1.44
Death Valley, CA	1.70	1.83	1.29	1.43	1.28	1.12	1.04	NaN	1.05	0.93
Fresno, CA	0.71	0.82	0.78	0.98	1.00	0.44	0.48	NaN	0.62	0.62
Monterey, CA	1.46	1.53	1.59	1.75	1.75	0.91	0.84	NaN	1.21	1.21
Santa Cruz, CA	1.50	1.52	1.91	1.86	1.97	0.90	0.90	NaN	1.47	1.35
Bakersfield, CA	1.16	1.16	1.17	1.44	1.39	0.72	0.68	NaN	0.96	0.94
Brawley, CA	1.50	1.50	1.80	1.91	3.03	0.90	0.90	NaN	1.94	2.35
Century City, CA	1.83	2.17	2.53	2.49	2.04	1.09	1.21	NaN	1.84	1.46
Cucamonga, CA	2.43	3.39	2.74	2.71	2.70	1.33	1.63	NaN	2.34	2.15
Diablo Canyon, CA	1.64	1.41	1.40	1.43	1.28	0.95	0.78	NaN	1.02	0.93
Irvine, CA	1.50	1.55	1.50	1.68	1.51	0.80	0.86	NaN	1.12	1.01
Long Beach, CA	1.80	1.64	2.02	2.03	1.84	1.05	0.93	NaN	1.58	1.41
Los Angeles, CA	2.16	2.40	2.36	2.37	2.11	1.09	1.26	NaN	1.66	1.44
Malibu West, CA	2.33	2.28	1.83	1.90	1.79	1.43	1.17	NaN	1.29	1.19
Northridge, CA	1.63	1.69	2.08	2.26	2.21	0.91	0.90	NaN	1.65	1.50
Palm Springs, CA	1.50	1.99	2.29	2.29	2.41	0.90	1.44	NaN	2.13	1.76

Site Name	S _M s (g)					S _M 1 (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Palmdale, CA	2.21	2.64	3.00	3.07	3.48	1.83	1.96	NaN	3.28	3.21
Pasadena, CA	2.50	2.88	2.50	2.47	2.26	1.29	1.50	NaN	1.76	1.51
Riverside, CA	1.50	1.50	1.80	1.67	1.97	0.90	0.90	NaN	1.45	1.36
San Bernardino, CA	1.70	2.37	2.79	2.97	3.03	0.93	1.63	NaN	2.83	2.54
San Diego, CA	1.61	1.25	1.89	1.80	1.72	0.96	0.73	NaN	1.23	1.20
San Luis Obispo, CA	1.24	1.18	1.31	1.45	1.29	0.72	0.67	NaN	1.00	0.91
San Onofre, CA	1.23	1.20	1.26	1.48	1.32	0.70	0.69	NaN	0.98	0.88
Santa Barbara, CA	2.07	2.83	2.54	2.44	2.45	1.21	1.49	NaN	1.89	1.77
Ventura, CA	2.46	2.38	2.42	2.38	2.90	1.28	1.35	NaN	2.10	2.29
Astoria, OR	1.33	1.31	1.54	2.05	1.52	1.00	1.01	NaN	1.44	1.01
Bend, OR	0.58	0.60	0.56	0.58	0.40	0.35	0.41	0.43	0.44	0.30
Brookings, OR	1.50	1.49	2.42	2.99	2.37	1.08	1.06	NaN	2.07	1.45
Coos Bay, OR	1.50	1.38	1.87	2.48	1.85	1.06	1.04	NaN	1.71	1.22
Eugene, OR	0.85	0.93	0.87	1.14	0.86	0.58	0.65	NaN	0.83	0.61
Hillsboro, OR	1.03	1.09	1.06	1.25	1.11	0.60	0.69	NaN	0.86	0.79
Klamath Falls, OR	1.02	1.02	1.28	1.31	0.88	0.59	0.60	NaN	0.91	0.59
Medford, OR	0.78	0.80	0.82	1.03	0.79	0.50	0.58	NaN	0.76	0.57
Newport, OR	1.82	1.74	2.01	2.40	1.99	1.04	1.15	NaN	1.65	1.31
Pendleton, OR	0.60	0.53	0.55	0.52	0.41	0.29	0.30	0.32	0.29	0.23
Portland, OR	1.09	1.09	1.07	1.15	1.12	0.59	0.66	NaN	0.78	0.77
Salem, OR	0.94	1.05	1.00	1.22	1.03	0.59	0.69	NaN	0.84	0.67
Aberdeen, WA	1.43	1.47	1.83	2.22	1.85	0.98	1.05	NaN	1.45	1.08
Bellingham, WA	1.09	1.07	1.21	1.32	1.22	0.57	0.62	NaN	0.72	0.66
Ellensburg, WA	0.74	0.73	0.70	0.72	0.69	0.38	0.43	NaN	0.42	0.41
Everett, WA	1.20	1.27	1.44	1.55	1.40	0.64	0.73	NaN	0.97	0.80
Hanford Site, WA	0.66	0.61	0.60	0.62	0.54	0.33	0.35	0.37	0.35	0.32
Olympia, WA	1.20	1.33	1.70	1.86	1.81	0.68	0.82	NaN	1.06	0.94
Seattle, WA	1.45	1.37	1.68	1.76	1.75	0.74	0.79	NaN	1.38	1.26
Spokane, WA	0.60	0.51	0.48	0.47	0.29	0.27	0.27	0.27	0.25	0.17
Tacoma, WA	1.23	1.30	1.63	1.62	1.77	0.66	0.76	NaN	0.96	0.92
Yakima, WA	0.72	0.71	0.67	0.69	0.68	0.37	0.42	0.43	0.41	0.41
Carson City, NV	1.66	2.28	2.41	2.44	1.74	0.90	1.19	NaN	1.73	1.17
Elko, NV	0.69	0.72	0.68	0.71	0.60	0.36	0.35	0.37	0.40	0.32
Las Vegas, NV	0.76	0.70	0.83	0.82	0.65	0.37	0.36	NaN	0.51	0.38
Reno, NV	1.50	1.50	1.76	1.74	1.51	0.89	0.78	NaN	1.32	0.97
Grand Canyon Village, AZ	0.80	0.66	0.56	0.59	0.32	0.36	0.30	0.27	0.30	0.17

Site Name	S _{MS} (g)					S _{M1} (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Palo Verde, AZ	0.27	0.25	0.27	0.28	0.24	0.15	0.13	0.20	0.22	0.20
Phoenix, AZ	0.28	0.27	0.28	0.28	0.17	0.15	0.14	0.16	0.16	0.11
Tucson, AZ	0.45	0.43	0.43	0.42	0.21	0.19	0.19	0.20	0.21	0.12
Denver, CO	0.35	0.29	0.34	0.24	0.21	0.14	0.14	0.14	0.10	0.11
La Junta, CO	0.30	0.30	0.28	0.20	0.16	0.14	0.16	0.14	0.11	0.09
Paradox, CO	0.38	0.31	0.32	0.23	0.27	0.15	0.15	0.14	0.11	0.14
Rangely, CO	0.47	0.41	0.48	0.32	0.27	0.17	0.18	0.18	0.13	0.14
Trinidad, CO	0.39	0.51	0.36	0.23	0.23	0.16	0.21	0.16	0.13	0.12
Idaho National Lab, ID	0.50	0.51	0.44	0.49	0.37	0.30	0.29	0.28	0.31	0.25
Billings, MT	0.22	0.19	0.21	0.18	0.18	0.12	0.12	0.14	0.13	0.13
Bozeman, MT	0.90	0.88	0.86	0.88	0.95	0.45	0.41	NaN	0.49	0.52
Lima, MT	1.19	1.34	1.43	1.33	1.00	0.64	0.64	NaN	0.88	0.58
Missoula, MT	0.74	0.70	0.64	0.64	0.61	0.35	0.33	0.34	0.36	0.33
Albuquerque, NM	0.76	0.64	0.61	0.57	0.51	0.36	0.30	0.31	0.32	0.27
Artesia, NM	0.25	0.18	0.14	0.14	0.09	0.11	0.10	0.09	0.10	0.07
Crownpoint, NM	0.43	0.35	0.29	0.24	0.34	0.17	0.16	0.14	0.13	0.16
Las Cruces, NM	0.49	0.45	0.44	0.45	0.26	0.24	0.22	0.22	0.24	0.17
Los Alamos National Lab, NM	0.68	0.75	0.72	0.68	0.51	0.34	0.35	0.38	0.41	0.28
Edgemont, SD	0.34	0.25	0.35	0.24	0.23	0.12	0.12	0.13	0.08	0.09
Amarillo, TX	0.28	0.25	0.31	0.22	0.22	0.10	0.11	0.11	0.08	0.08
Kermit, TX	0.40	0.32	0.29	0.16	0.11	0.11	0.11	0.10	0.09	0.06
Snyder, TX	0.15	0.12	0.11	0.11	0.10	0.08	0.08	0.07	0.06	0.07
Angle, UT	0.86	0.79	0.88	0.73	0.86	0.40	0.36	NaN	0.39	0.46
Brigham City, UT	1.44	1.47	1.68	1.58	1.19	0.85	0.81	NaN	1.12	0.84
Cedar City, UT	0.88	0.85	0.91	0.84	0.75	0.44	0.41	NaN	0.51	0.43
Green River, UT	0.42	0.44	0.51	0.34	0.39	0.20	0.21	0.21	0.17	0.19
Provo, UT	1.27	1.23	1.66	1.59	1.14	0.81	0.70	NaN	1.16	0.80
Salt Lake City, UT	1.72	1.54	1.85	1.81	1.46	1.05	0.84	NaN	1.28	1.00
Casper, WY	0.56	0.44	0.49	0.34	0.37	0.19	0.18	0.17	0.11	0.17
Cheyenne, WY	0.31	0.25	0.26	0.22	0.19	0.13	0.13	0.12	0.08	0.09
Jackson, WY	1.19	1.32	1.25	1.31	1.14	0.66	0.64	NaN	0.84	0.69
Kaycee, WY	0.56	0.46	0.51	0.34	0.56	0.18	0.18	0.17	0.11	0.26
Sunday Peak, WY	0.87	0.89	1.06	0.93	1.03	0.42	0.43	NaN	0.52	0.54
Yellowstone, WY	1.25	1.29	1.49	1.52	1.26	0.65	0.63	NaN	0.97	0.73
Atmore, AL	0.21	0.19	0.16	0.18	0.16	0.14	0.15	0.15	0.15	0.17

Site Name	S _M s (g)					S _M 1 (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Birmingham, AL	0.47	0.42	0.47	0.39	0.36	0.23	0.25	0.24	0.24	0.22
El Dorado, AR	0.38	0.30	0.26	0.32	0.30	0.23	0.23	0.21	0.22	0.25
Enola, AR	0.81	0.69	0.93	0.84	0.92	0.38	0.38	0.43	0.42	0.41
Greenbrier, AR	0.76	0.64	0.73	0.68	0.69	0.36	0.36	0.38	0.38	0.35
Little Rock, AR	0.70	0.60	0.58	0.59	0.54	0.35	0.35	0.34	0.35	0.32
Hartford, CT	0.38	0.29	0.30	0.25	0.30	0.15	0.15	0.13	0.10	0.13
Washington, DC	0.25	0.19	0.21	0.19	0.26	0.12	0.12	0.10	0.09	0.12
Wilmington, DE	0.42	0.31	0.29	0.26	0.32	0.14	0.14	0.11	0.10	0.12
Jacksonville, FL	0.23	0.18	0.17	0.20	0.17	0.15	0.14	0.13	0.13	0.14
Miami, FL	0.08	0.07	0.06	0.07	0.06	0.05	0.05	0.05	0.04	0.05
Orlando, FL	0.15	0.13	0.10	0.11	0.10	0.09	0.09	0.08	0.08	0.08
Tampa, FL	0.12	0.10	0.09	0.09	0.08	0.08	0.08	0.07	0.07	0.07
Atlanta, GA	0.36	0.30	0.29	0.31	0.30	0.21	0.22	0.20	0.20	0.19
Lincolnton, GA	0.52	0.45	0.67	0.52	0.41	0.25	0.25	0.26	0.23	0.21
Savannah, GA	0.60	0.48	0.49	0.49	0.39	0.29	0.28	0.27	0.26	0.24
Des Moines, IA	0.12	0.10	0.10	0.14	0.12	0.11	0.11	0.13	0.13	0.12
Boise, ID	0.48	0.48	0.48	0.50	0.29	0.25	0.25	0.26	0.28	0.18
Central, IL	0.36	0.29	0.27	0.34	0.31	0.23	0.23	0.22	0.24	0.21
Chicago, IL	0.26	0.22	0.19	0.20	0.19	0.14	0.15	0.15	0.15	0.14
Evansville, IN	0.85	0.77	0.75	0.74	0.70	0.39	0.40	0.41	0.41	0.37
Indianapolis, IN	0.31	0.25	0.29	0.31	0.29	0.20	0.20	0.20	0.21	0.19
Topeka, KS	0.25	0.21	0.16	0.17	0.16	0.13	0.14	0.15	0.15	0.14
Wichita, KS	0.22	0.18	0.15	0.16	0.14	0.12	0.13	0.13	0.13	0.12
Louisville, KY	0.39	0.33	0.32	0.38	0.35	0.25	0.25	0.24	0.25	0.23
New Orleans, LA	0.18	0.15	0.14	0.15	0.13	0.11	0.12	0.13	0.13	0.15
Boston, MA	0.44	0.35	0.42	0.30	0.33	0.16	0.17	0.16	0.12	0.13
Baltimore, MD	0.27	0.21	0.23	0.20	0.27	0.12	0.12	0.10	0.09	0.12
Augusta, ME	0.46	0.37	0.49	0.36	0.32	0.19	0.19	0.19	0.14	0.14
Bangor, ME	0.41	0.33	0.47	0.35	0.30	0.18	0.18	0.18	0.15	0.14
Portland, ME	0.49	0.38	0.44	0.33	0.33	0.18	0.19	0.17	0.13	0.14
Detroit, MI	0.19	0.15	0.16	0.14	0.16	0.11	0.11	0.11	0.10	0.11
Grand Rapids, MI	0.14	0.11	0.14	0.14	0.13	0.10	0.11	0.11	0.11	0.11
Minneapolis, MN	0.10	0.08	0.08	0.08	0.07	0.06	0.07	0.07	0.08	0.08
Cape Girardeau, MO	1.50	1.31	1.42	1.42	1.20	0.84	0.71	NaN	0.82	0.67
Kansas City, MO	0.20	0.18	0.15	0.20	0.18	0.14	0.15	0.17	0.16	0.15
New Madrid, MO	3.34	2.91	3.84	3.67	3.08	1.95	1.73	NaN	2.23	2.21

Site Name	S _{MS} (g)					S _{M1} (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
St Louis, MO	0.78	0.64	0.66	0.68	0.63	0.36	0.36	0.37	0.38	0.34
Jackson, MS	0.31	0.25	0.22	0.30	0.25	0.20	0.21	0.20	0.22	0.24
Asheville, NC	0.58	0.48	0.47	0.41	0.41	0.25	0.26	0.23	0.21	0.20
Charlotte, NC	0.49	0.38	0.32	0.33	0.31	0.25	0.25	0.20	0.19	0.18
Greenville, NC	0.25	0.20	0.15	0.18	0.18	0.16	0.15	0.12	0.11	0.13
Raleigh, NC	0.32	0.25	0.19	0.22	0.22	0.19	0.18	0.14	0.14	0.14
Fargo, ND	0.12	0.09	0.09	0.08	0.07	0.05	0.05	0.05	0.03	0.03
Omaha, NE	0.20	0.16	0.12	0.12	0.11	0.10	0.11	0.11	0.11	0.11
Manchester, NH	0.53	0.42	0.63	0.42	0.38	0.19	0.19	0.21	0.14	0.15
Trenton, NJ	0.45	0.35	0.33	0.25	0.33	0.15	0.15	0.12	0.10	0.13
Batavia, NY	0.42	0.33	0.29	0.23	0.20	0.14	0.15	0.12	0.09	0.10
Malone, NY	0.82	0.74	0.71	0.52	0.53	0.29	0.31	0.26	0.18	0.20
New York, NY	0.55	0.44	0.46	0.31	0.37	0.17	0.17	0.14	0.11	0.14
Cincinnati, OH	0.28	0.23	0.23	0.27	0.26	0.18	0.19	0.18	0.19	0.17
Cleveland, OH	0.30	0.27	0.23	0.17	0.18	0.12	0.14	0.12	0.11	0.11
Columbus, OH	0.24	0.19	0.19	0.21	0.19	0.14	0.15	0.14	0.14	0.14
Sidney, OH	0.41	0.33	0.54	0.37	0.36	0.17	0.18	0.19	0.18	0.17
Youngstown, OH	0.24	0.26	0.18	0.15	0.16	0.12	0.14	0.11	0.11	0.11
Elgin, OK	0.62	0.66	0.98	0.79	0.52	0.22	0.29	NaN	0.38	0.26
Oklahoma City, OK	0.51	0.42	0.48	0.37	0.26	0.18	0.19	0.19	0.17	0.14
Philadelphia, PA	0.43	0.32	0.29	0.25	0.31	0.14	0.15	0.11	0.10	0.13
Pittsburg, PA	0.20	0.18	0.14	0.14	0.16	0.12	0.13	0.11	0.11	0.11
Providence, RI	0.37	0.28	0.32	0.25	0.28	0.15	0.15	0.13	0.10	0.12
Charleston, SC	1.51	1.20	1.70	1.52	1.11	0.61	0.61	NaN	0.77	0.64
Platte, SD	0.29	0.24	0.29	0.20	0.16	0.09	0.09	0.09	0.06	0.05
Sioux Falls, SD	0.18	0.14	0.15	0.12	0.11	0.08	0.08	0.09	0.08	0.08
Chattanooga, TN	0.66	0.57	0.68	0.55	0.52	0.27	0.29	0.29	0.27	0.26
Knoxville, TN	0.72	0.61	0.82	0.60	0.61	0.28	0.29	0.32	0.26	0.26
Maryville, TN	0.73	0.62	1.10	0.78	0.68	0.28	0.29	0.37	0.30	0.28
Memphis, TN	1.40	1.11	1.23	1.25	1.05	0.62	0.60	NaN	0.72	0.73
Nashville, TN	0.53	0.47	0.45	0.54	0.47	0.30	0.32	0.33	0.34	0.30
Austin, TX	0.13	0.10	0.08	0.09	0.09	0.08	0.08	0.07	0.09	0.10
Dallas, TX	0.19	0.15	0.16	0.16	0.14	0.12	0.12	0.13	0.13	0.14
Houston, TX	0.14	0.12	0.11	0.11	0.10	0.09	0.09	0.09	0.10	0.12
San Antonio, TX	0.18	0.13	0.08	0.08	0.08	0.07	0.07	0.05	0.05	0.08
Blacksburg, VA	0.48	0.38	0.37	0.32	0.32	0.20	0.20	0.17	0.16	0.16

Site Name	S_{MS} (g)					S_{MI} (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Michaux, VA	0.43	0.38	0.55	0.37	0.56	0.16	0.17	0.17	0.14	0.21
Richmond, VA	0.35	0.30	0.38	0.28	0.41	0.14	0.15	0.13	0.11	0.16
Virginia Beach, VA	0.17	0.14	0.13	0.14	0.16	0.11	0.11	0.09	0.08	0.11
Burlington, VT	0.57	0.54	0.54	0.40	0.41	0.23	0.25	0.21	0.16	0.17
Milwaukee, WI	0.17	0.14	0.12	0.13	0.12	0.11	0.11	0.12	0.12	0.11
Charleston, WV	0.30	0.23	0.25	0.25	0.25	0.16	0.17	0.15	0.15	0.15

Appendix B - Alaska Tables

Table B1. The 33 Alaska (AK) test sites.

Site Number	Site Name	Latitude (degrees)	Longitude (degrees)	Region
1	Adak	51.90	-176.70	AK
2	Anchorage	61.20	-149.90	AK
3	Barrow	71.30	-156.80	AK
4	Bethel	60.80	-161.80	AK
5	Chitina	61.50	-144.50	AK
6	Delta Junction	64.00	-145.70	AK
7	Dillingham	59.10	-158.50	AK
8	Evansville	66.90	-151.50	AK
9	Fairbanks	64.90	-147.70	AK
10	Gakona	62.30	-145.30	AK
11	Galena	64.80	-156.90	AK
12	Glennallen	62.10	-145.60	AK
13	Haines	59.30	-135.50	AK
14	Homer	59.70	-151.50	AK
15	Juneau	58.30	-134.40	AK
16	Kenai	60.60	-151.30	AK
17	Ketchikan	55.40	-131.70	AK
18	Knik-Fairview	61.50	-149.70	AK
19	Kodiak	57.80	-152.40	AK
20	Kotzebue	66.90	-162.60	AK
21	Lakes	61.60	-149.40	AK
22	McGrath	63.00	-155.60	AK
23	Nome	64.50	-165.40	AK
24	Paxson	63.10	-145.50	AK
25	Prudhoe Bay	70.30	-148.40	AK
26	Seward	60.10	-149.40	AK
27	Sitka	57.10	-135.40	AK
28	Sleetmute	61.70	-157.20	AK
29	Tok	63.30	-143.00	AK
30	Unalaska	53.90	-166.60	AK
31	Valdez	61.20	-146.40	AK
32	Wasilla	61.60	-149.50	AK
33	Yakutat	59.60	-139.70	AK

Table B2. Comparisons between 2026 NEHRP and previous ASCE 7 Standards for S_s and S_1 (reference site class, B or BC depending on the edition, at 0.2 and 1.0 Sec) values for the 33 AK test sites. All values are rounded to two decimal places.

Site Name	S_s (g)					S_1 (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Adak	1.74	1.50	1.50	1.50	3.41	0.69	0.60	0.60	0.60	0.86
Anchorage	1.49	1.50	1.50	1.50	3.27	0.55	0.68	0.68	0.65	0.95
Barrow	0.02	0.01	0.01	0.01	0.05	0.00	0.00	0.00	0.00	0.02
Bethel	0.30	0.30	0.30	0.33	0.07	0.09	0.13	0.13	0.12	0.04
Chitina	1.21	1.15	1.15	1.16	2.71	0.44	0.54	0.54	0.49	0.84
Delta Junction	0.62	0.73	0.73	0.74	0.92	0.27	0.37	0.37	0.33	0.34
Dillingham	0.26	0.30	0.30	0.30	0.36	0.13	0.17	0.17	0.16	0.14
Evansville	0.47	0.50	0.50	0.53	0.48	0.14	0.20	0.20	0.18	0.14
Fairbanks	1.10	0.98	0.98	1.02	1.15	0.30	0.37	0.37	0.34	0.33
Gakona	0.77	0.81	0.81	0.83	2.25	0.30	0.40	0.40	0.36	0.72
Galena	0.36	0.44	0.44	0.48	0.37	0.11	0.17	0.17	0.16	0.12
Glennallen	1.02	0.99	0.99	1.01	2.36	0.34	0.46	0.46	0.41	0.75
Haines	1.16	1.00	1.00	1.07	1.13	0.44	0.48	0.48	0.45	0.31
Homer	1.45	1.50	1.50	1.50	3.11	0.55	0.61	0.61	0.60	0.87
Juneau	0.56	0.53	0.53	0.54	0.42	0.27	0.36	0.36	0.31	0.17
Kenai	1.29	1.50	1.50	1.50	3.07	0.47	0.60	0.60	0.60	0.88
Ketchikan	0.26	0.30	0.30	0.29	0.23	0.17	0.25	0.25	0.22	0.12
Knik-Fairview	1.45	1.50	1.50	1.50	3.21	0.54	0.72	0.72	0.69	0.91
Kodiak	1.79	1.50	1.50	1.58	2.85	0.66	0.88	0.88	0.85	0.88
Kotzebue	0.43	0.42	0.42	0.45	0.45	0.13	0.16	0.16	0.15	0.13
Lakes	1.45	1.81	1.81	1.98	3.05	0.54	0.93	0.93	0.82	0.88
McGrath	0.36	0.41	0.41	0.44	0.43	0.13	0.19	0.19	0.18	0.16
Nome	0.51	0.54	0.54	0.58	0.57	0.14	0.19	0.19	0.18	0.15
Paxson	1.02	1.01	1.01	1.02	1.64	0.45	0.57	0.57	0.53	0.57
Prudhoe Bay	0.24	0.28	0.28	0.31	0.33	0.07	0.10	0.10	0.10	0.10
Seward	1.79	1.50	1.50	1.50	2.91	0.67	0.72	0.72	0.69	0.94
Sitka	1.03	0.95	0.95	0.93	0.74	0.51	0.60	0.60	0.57	0.28
Sleetmute	0.57	0.62	0.62	0.67	0.42	0.17	0.23	0.23	0.22	0.15
Tok	0.45	0.53	0.53	0.53	0.88	0.28	0.34	0.34	0.30	0.32
Unalaska	1.46	1.50	1.50	1.50	2.34	0.56	0.60	0.60	0.60	0.66
Valdez	1.43	1.50	1.50	1.57	2.67	0.56	0.77	0.77	0.74	0.83
Wasilla	1.48	2.00	2.00	2.13	3.10	0.55	0.95	0.95	0.85	0.89
Yakutat	1.78	1.63	1.63	1.76	1.37	0.67	0.76	0.76	0.73	0.48

Table B3. Comparisons between 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} (default site conditions as defined in each edition: in 2026 NEHRP and ASCE 7-22, “Default” is the maximum of C, CD, and D; in ASCE 7-16, “D-default” is per Section 11.4.4 equivalent to the greater of C and D; in ASCE 7-10 and ASCE -05, default is D) values for the 33 AK test sites. Where values are listed as NaN, in ASCE 7-16, site-specific procedures were required due to known problems with the F_v parameter. All values are rounded to two decimal places.

Site Name	S_{MS} (g)					S_{M1} (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Adak	1.74	1.50	1.80	1.65	3.79	1.03	0.90	NaN	1.70	1.98
Anchorage	1.49	1.50	1.80	1.65	3.27	0.83	1.02	NaN	1.85	2.05
Barrow	0.03	0.01	0.01	0.02	0.09	0.01	0.01	0.01	0.01	0.05
Bethel	0.46	0.47	0.47	0.46	0.13	0.23	0.29	0.29	0.34	0.12
Chitina	1.23	1.20	1.38	1.47	2.40	0.69	0.82	NaN	1.42	1.51
Delta Junction	0.81	0.89	0.89	1.05	1.23	0.51	0.61	NaN	0.83	0.84
Dillingham	0.41	0.46	0.46	0.42	0.65	0.29	0.36	0.39	0.46	0.41
Evansville	0.67	0.70	0.70	0.78	0.66	0.31	0.40	0.43	0.49	0.37
Fairbanks	1.17	1.08	1.17	1.23	1.42	0.54	0.62	NaN	0.85	0.85
Gakona	0.92	0.95	0.98	1.05	2.05	0.54	0.65	NaN	1.06	1.36
Galena	0.55	0.64	0.64	0.67	0.53	0.25	0.36	0.38	0.46	0.32
Glennallen	1.11	1.09	1.19	1.29	2.16	0.58	0.71	NaN	1.21	1.41
Haines	1.20	1.10	1.20	1.52	1.38	0.69	0.73	NaN	1.15	0.81
Homer	1.45	1.50	1.80	1.59	3.27	0.83	0.91	NaN	1.70	1.94
Juneau	0.76	0.73	0.73	0.81	0.65	0.50	0.60	NaN	0.85	0.46
Kenai	1.29	1.50	1.80	1.51	3.21	0.72	0.90	NaN	1.62	1.96
Ketchikan	0.41	0.46	0.46	0.44	0.42	0.35	0.47	NaN	0.60	0.33
Knik-Fairview	1.45	1.50	1.80	1.65	3.29	0.80	1.07	NaN	1.95	2.01
Kodiak	1.79	1.50	1.80	1.68	2.66	1.00	1.32	NaN	2.38	1.73
Kotzebue	0.62	0.61	0.61	0.67	0.64	0.29	0.34	0.36	0.41	0.36
Lakes	1.45	1.81	2.17	2.03	3.11	0.82	1.39	NaN	2.28	1.94
McGrath	0.55	0.61	0.61	0.61	0.72	0.29	0.39	0.43	0.52	0.44
Nome	0.71	0.74	0.74	0.71	0.75	0.31	0.39	0.42	0.46	0.40
Paxson	1.12	1.10	1.21	1.44	1.76	0.70	0.86	NaN	1.33	1.26
Prudhoe Bay	0.38	0.44	0.44	0.45	0.50	0.16	0.25	0.25	0.27	0.28
Seward	1.79	1.50	1.80	1.65	2.65	1.00	1.08	NaN	1.96	1.76
Sitka	1.12	1.07	1.14	1.32	1.06	0.77	0.90	NaN	1.44	0.73
Sleetmute	0.77	0.81	0.81	0.84	0.66	0.36	0.45	NaN	0.58	0.42
Tok	0.65	0.73	0.73	0.80	1.14	0.51	0.58	NaN	0.82	0.81
Unalaska	1.46	1.50	1.80	1.63	2.66	0.83	0.90	NaN	1.70	1.53
Valdez	1.43	1.50	1.80	1.70	2.42	0.84	1.15	NaN	1.96	1.54

Site Name	S _{MS} (g)					S _{M1} (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Wasilla	1.48	2.00	2.40	2.10	3.18	0.82	1.43	NaN	2.35	1.97
Yakutat	1.78	1.63	1.96	1.89	1.69	1.01	1.14	NaN	1.95	1.18

Appendix C - Hawaii Tables

Table C1. The 36 Hawaii (HI) test sites.

Site Number	Site Name	Latitude (degrees)	Longitude (degrees)	Region
1	Barbers Point	21.30	-158.10	HI
2	Barking Sands	22.06	-159.76	HI
3	Bradshaw Airfield	19.76	-155.56	HI
4	Diamond Head	21.26	-157.80	HI
5	Haleakala Crater	20.70	-156.26	HI
6	Hana	20.76	-155.98	HI
7	Hanapepe	21.90	-159.60	HI
8	Hawaiian Ocean View	19.10	-155.78	HI
9	Hilo Atc	19.70	-155.10	HI
10	Hilo	19.70	-155.06	HI
11	Honolulu	21.30	-157.86	HI
12	Kaheawa Wind	20.80	-156.56	HI
13	Kahuku	21.68	-157.94	HI
14	Kahului	20.90	-156.50	HI
15	Kailua Kona	19.66	-156.00	HI
16	Kaneohe	21.40	-157.80	HI
17	Kaunakakai	21.10	-157.00	HI
18	Kihei	20.78	-156.46	HI
19	Kilauea	19.40	-155.26	HI
20	Lahaina	20.88	-156.68	HI
21	Laie	21.66	-157.96	HI
22	Lanai City	20.80	-156.96	HI
23	Lihue	21.96	-159.36	HI
24	Marine Corps Base	21.46	-157.76	HI
25	Mauna Kea	19.80	-155.46	HI
26	Ocean View	19.10	-155.76	HI
27	Pearl Harbor	21.36	-157.96	HI
28	Pepeekeo	19.84	-155.10	HI
29	Princeville	22.22	-159.48	HI
30	Puuwai	21.90	-160.20	HI
31	Volcano	19.44	-155.22	HI
32	Wahiawa	21.50	-158.00	HI
33	Waianae	21.46	-158.20	HI
34	Waikoloa Village	19.94	-155.78	HI
35	Waimea	20.00	-155.70	HI
36	Waipahu	21.40	-158.00	HI

Table C2. Comparisons between 2026 NEHRP and previous ASCE 7 Standards for S_s and S_1 (reference site class, B or BC depending on the edition, at 0.2 and 1.0 Sec) values for the 36 HI test sites. All values are rounded to two decimal places.

Site Name	S_s (g)					S_1 (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Barbers Point	0.58	0.54	0.54	0.49	0.45	0.16	0.15	0.15	0.14	0.12
Barking Sands	0.19	0.17	0.17	0.17	0.12	0.06	0.05	0.05	0.05	0.04
Bradshaw Airfield	1.50	1.50	1.50	1.50	2.47	0.60	0.60	0.60	0.60	0.75
Diamond Head	0.62	0.58	0.58	0.53	0.55	0.18	0.17	0.17	0.15	0.15
Haleakala Crater	1.02	1.05	1.05	0.77	0.97	0.26	0.28	0.28	0.23	0.31
Hana	1.01	1.05	1.05	0.77	0.99	0.27	0.28	0.28	0.24	0.32
Hanapepe	0.22	0.20	0.20	0.19	0.13	0.06	0.06	0.06	0.06	0.04
Hawaiian Ocean View	2.10	2.10	2.49	2.49	2.94	0.88	0.88	1.06	1.06	0.98
Hilo Atc	1.50	1.50	1.50	1.50	2.39	0.60	0.60	0.60	0.60	0.71
Hilo	1.50	1.50	1.50	1.50	2.37	0.60	0.60	0.60	0.60	0.71
Honolulu	0.61	0.58	0.58	0.53	0.53	0.18	0.17	0.17	0.15	0.14
Kaheawa Wind	0.95	0.95	0.95	0.72	0.85	0.24	0.24	0.24	0.22	0.27
Kahuku	0.59	0.55	0.55	0.50	0.45	0.17	0.15	0.15	0.14	0.12
Kahului	0.97	0.98	0.98	0.73	0.83	0.25	0.25	0.25	0.22	0.27
Kailua Kona	1.92	1.92	2.31	2.23	2.41	0.88	0.88	1.01	0.82	0.75
Kaneohe	0.62	0.59	0.59	0.54	0.54	0.18	0.17	0.17	0.16	0.14
Kaunakakai	0.81	0.79	0.79	0.65	0.74	0.21	0.21	0.21	0.19	0.22
Kihei	0.98	0.99	0.99	0.74	0.88	0.25	0.26	0.26	0.22	0.28
Kilauea	2.67	2.67	3.21	3.21	5.83	1.23	1.23	1.47	1.47	2.12
Lahaina	0.91	0.91	0.91	0.70	0.81	0.23	0.23	0.23	0.21	0.26
Laie	0.59	0.55	0.55	0.50	0.45	0.17	0.15	0.15	0.14	0.12
Lanai City	0.76	0.75	0.75	0.63	0.75	0.21	0.21	0.21	0.19	0.24
Lihue	0.25	0.23	0.23	0.21	0.15	0.07	0.06	0.06	0.06	0.04
Marine Corps Base	0.63	0.59	0.59	0.55	0.54	0.18	0.17	0.17	0.16	0.15
Mauna Kea	1.50	1.50	1.50	1.50	2.41	0.60	0.60	0.60	0.60	0.73
Ocean View	2.22	2.22	2.65	2.65	3.13	0.94	0.94	1.14	1.14	1.06
Pearl Harbor	0.61	0.57	0.57	0.52	0.49	0.17	0.16	0.16	0.15	0.13
Pepeekeo	1.38	1.50	1.50	1.35	2.19	0.59	0.60	0.60	0.54	0.63
Princeville	0.21	0.19	0.19	0.19	0.12	0.06	0.06	0.06	0.05	0.04
Puuwai	0.17	0.15	0.15	0.14	0.11	0.05	0.04	0.04	0.04	0.03
Volcano	2.33	2.33	2.79	2.79	4.76	1.00	1.00	1.21	1.21	1.51
Wahiawa	0.60	0.56	0.56	0.51	0.46	0.17	0.16	0.16	0.15	0.12
Waianae	0.57	0.53	0.53	0.48	0.40	0.16	0.15	0.15	0.14	0.11

Site Name	S _s (g)					S ₁ (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Waikoloa Village	1.31	1.50	1.50	1.32	2.05	0.55	0.60	0.60	0.52	0.60
Waimea	1.28	1.50	1.50	1.21	1.99	0.48	0.56	0.56	0.46	0.58
Waipahu	0.60	0.56	0.56	0.51	0.47	0.17	0.16	0.16	0.15	0.13

Table C3. Comparisons between 2026 NEHRP and previous ASCE 7 Standards for S_{MS} and S_{M1} (default site conditions as defined in each edition: in 2026 NEHRP and ASCE 7-22, “Default” is the maximum of C, CD, and D; in ASCE 7-16, “D-default” is per Section 11.4.4 equivalent to the greater of C and D; in ASCE 7-10 and ASCE -05, default is D) values for the 36 HI test sites. Where values are listed as NaN, in ASCE 7-16, site-specific procedures were required due to known problems with the F_v parameter. All values are rounded to two decimal places.

Site Name	S_{MS} (g)					S_{M1} (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Barbers Point	0.78	0.74	0.74	0.69	0.62	0.35	0.33	0.35	0.39	0.29
Barking Sands	0.30	0.27	0.27	0.24	0.19	0.14	0.12	0.12	0.14	0.09
Bradshaw Airfield	1.50	1.50	1.80	1.65	2.88	0.90	0.90	NaN	1.70	1.75
Diamond Head	0.81	0.78	0.78	0.75	0.73	0.37	0.36	0.39	0.43	0.35
Haleakala Crater	1.11	1.13	1.26	0.93	1.29	0.49	0.51	NaN	0.60	0.71
Hana	1.11	1.13	1.26	0.93	1.31	0.50	0.51	NaN	0.60	0.73
Hanapepe	0.36	0.32	0.32	0.27	0.21	0.15	0.14	0.14	0.15	0.10
Hawaiian Ocean View	2.10	2.10	2.99	2.60	3.17	1.31	1.31	NaN	2.92	2.21
Hilo Atc	1.50	1.50	1.80	1.65	2.83	0.90	0.90	NaN	1.70	1.64
Hilo	1.50	1.50	1.80	1.65	2.81	0.90	0.90	NaN	1.70	1.65
Honolulu	0.80	0.77	0.77	0.74	0.71	0.37	0.36	0.38	0.42	0.34
Kaheawa Wind	1.06	1.07	1.15	0.87	1.14	0.46	0.47	NaN	0.55	0.62
Kahuku	0.78	0.75	0.75	0.70	0.62	0.35	0.34	0.35	0.39	0.28
Kahului	1.08	1.09	1.18	0.89	1.11	0.47	0.47	NaN	0.56	0.61
Kailua Kona	1.92	1.92	2.77	2.34	2.65	1.32	1.32	NaN	2.14	1.63
Kaneohe	0.81	0.78	0.78	0.76	0.72	0.38	0.36	0.39	0.43	0.34
Kaunakakai	0.95	0.94	0.95	0.79	0.96	0.42	0.41	NaN	0.49	0.51
Kihei	1.08	1.09	1.19	0.90	1.18	0.48	0.48	NaN	0.57	0.64
Kilauea	2.67	2.67	3.85	3.35	5.62	1.85	1.85	NaN	4.05	4.65
Lahaina	1.03	1.03	1.09	0.85	1.08	0.45	0.45	NaN	0.53	0.58
Laie	0.78	0.75	0.75	0.70	0.62	0.35	0.34	0.35	0.39	0.28
Lanai City	0.91	0.90	0.90	0.77	1.01	0.41	0.41	NaN	0.48	0.55
Lihue	0.40	0.36	0.36	0.30	0.23	0.17	0.16	0.16	0.17	0.11
Marine Corps Base	0.81	0.78	0.78	0.77	0.73	0.38	0.37	0.39	0.44	0.34
Mauna Kea	1.50	1.50	1.80	1.65	2.83	0.90	0.90	NaN	1.70	1.69
Ocean View	2.22	2.22	3.18	2.76	3.34	1.41	1.41	NaN	3.14	2.37
Pearl Harbor	0.80	0.76	0.76	0.73	0.67	0.37	0.35	0.37	0.41	0.32
Pepeekeo	1.38	1.50	1.80	1.56	2.58	0.89	0.90	NaN	1.49	1.41
Princeville	0.34	0.31	0.31	0.26	0.19	0.15	0.13	0.13	0.15	0.09
Puuwai	0.27	0.24	0.24	0.20	0.17	0.11	0.10	0.10	0.11	0.08
Volcano	2.33	2.33	3.34	2.91	4.79	1.50	1.50	NaN	3.32	3.27

Site Name	S_{MS} (g)					S_{M1} (g)				
	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP	ASCE 7-05	ASCE 7-10	ASCE 7-16	ASCE 7-22	2026 NEHRP
Wahiawa	0.79	0.76	0.76	0.71	0.64	0.36	0.34	0.36	0.40	0.30
Waianae	0.77	0.73	0.73	0.68	0.57	0.34	0.32	0.34	0.38	0.26
Waikoloa Village	1.31	1.50	1.80	1.54	2.45	0.82	0.90	NaN	1.43	1.37
Waimea	1.28	1.50	1.80	1.40	2.39	0.73	0.84	NaN	1.28	1.32
Waipahu	0.79	0.76	0.76	0.72	0.65	0.36	0.35	0.36	0.41	0.30

RESOURCE PAPER 4 TWO-STAGE ANALYSIS: CURRENT USE, DESIGN ISSUES, AND RECOMMENDATIONS FOR FUTURE WORK

RP4-1 Purpose and Introduction

This resource paper addresses the use of the two-stage analysis procedure, as found in Section 12.2.3.2 of ASCE/SEI 7-22 (ASCE, 2021). This procedure applies to structures that combine one or more flexible upper portions (towers) on a stiff lower portion (podium) and allows for separate analyses to be conducted of the tower and podium. During the development of the ASCE 7-22 seismic provisions, several questions regarding two-stage analysis were brought to the ASCE 7 Seismic Subcommittee. While some of the questions were addressed with modifications incorporated in ASCE 7-22, others required further study. As part of the Provisions Update Committee (PUC) work for the 2026 NEHRP Provisions, Issue Team 6 (IT6) was established to provide further study of two-stage analysis provisions. This resource paper is a product of the IT6 efforts. For purposes of this resource paper, the terms ‘tower’ and ‘podium’ will be used in place of the ASCE 7 terms ‘upper portion’ and ‘lower portion’; here, they have the same meaning.

Work undertaken by IT6 included a survey of current engineering practice using two-stage analysis; the survey indicated that both structures incorporating vertical combinations of SFRSs and the use of two-stage analyses are much more prevalent in the building stock and in current design practice than had been anticipated. During IT6 work, it was identified that a notable and growing portion of the building stock uses vertical combinations of SFRSs. Further, designers often prefer the use of two-stage analysis methods over combined analysis for structures with multiple SFRSs. It is felt that 1) the use of the two-stage method has significantly outgrown the intent when it was originally established, 2) the use of two-stage analysis has grown beyond the design space in which performance information from past earthquakes is available, 3) recent major studies of the seismic performance of currently-used SFRSs have not generally included study of designs using two-stage analysis methods, and 4) use of two-stage analysis has, in some cases, been identified to result in unconservative design where multiple towers are supported on a single podium. It is also noted that there has been significant progress in analytical tools available to judge performance and aid in the development of appropriate scoping provisions (as included in ASCE 7-22 Section 12.2.3.2) for two-stage analysis. With the need identified and appropriate tools available, further study and development of design provisions for two-stage analysis is recommended.

The intent of this resource paper is to document information collected by IT6, issues identified during the course of IT6 study, and analytical studies performed by IT6 members. Included in the IT6 work were focused studies on the dynamic interaction of towers and podiums, and the response of structures with

multiple towers on a single podium. This resource paper concludes with recommendations for further study of two-stage analysis.

RP4-2 Background and History

The first code adoption of two-stage analysis provisions was in the 1988 *Uniform Building Code* (UBC, ICBO, 1988) as part of a complete rewrite of the seismic provisions. At that time, the UBC was the predominant code used in western and central states. Section 2312(d)8B(iv), falling under “selection of lateral force procedure, static,” permitted use of the static lateral force procedure for structures having a flexible tower and rigid podium, provided that the average story stiffness of the podium was at least 10 times the approximate stiffness of the tower and the period of the entire structure was not more than 1.1 times the period of the tower. In addition, Section 2312(e)3B(ii), addressing combinations of structural systems, permitted a two-stage analysis procedure for structures meeting Section 2312(d)8B(iv).

Discussion of these provisions can be found in the 1990 SEAOC *Recommended Lateral Force Requirements and Commentary* (SEAOC Bluebook, SEAOC, 1990), including limited commentary. The commentary indicates that the provisions for vertical combinations were intended to prevent use of mixed systems that would result in concentration of inelastic behavior in the lower stories. It also indicates that the two-stage procedure is applicable for systems having the same R_w factor (precursor to an R-factor) for both the tower and the podium. No further discussion of intent is provided.

In the late 1980s and early 1990s, these provisions are understood to have been primarily used to permit separate seismic analysis for wood light-frame multi-family residential “tower” buildings constructed over concrete podiums; at that time, up to three stories of wood framing above a concrete podium was typical. Over subsequent decades, these building types expanded to include wood light-frame “towers” of up to five stories and cold-formed steel (CFS) light-frame construction. In addition, the two-stage analysis method has broadened well beyond its original application and is now used across low-rise, mid-rise, and high-rise construction, for virtually all materials and seismic force-resisting systems (SFRSs), and across a wide range of occupancies. In addition, when the two-stage analysis approach first appeared in the code, available computing power limited the use of nonlinear response history analysis, but computing power is now less of a hindrance.

The two-stage analysis provisions remained essentially unchanged in subsequent UBC updates through the 1997 UBC. The provisions were included in 2000 International Building Code (IBC) Section 1617.6.3.1, Exception 3, with similar criteria. These continued with moderate changes in the 2003 IBC. In the 2006 IBC, the two-stage analysis provisions, along with the bulk of the seismic design provisions, were deleted from the IBC and replaced with adoption by reference to ASCE 7-05 seismic design provisions. No commentary providing the technical basis for the UBC and IBC two-stage analysis provisions has been found.

Beginning with ASCE/SEI 7-05, the ASCE 7 standard incorporated the two-stage seismic analysis procedure. The eligibility criteria were consistent with those of the 2000 IBC, retaining the stiffness ratio and period ratio limits. This framework continued in subsequent editions of ASCE 7, up to and including ASCE/SEI 7-22. Full seismic commentary was first provided in the 2016 Edition of ASCE 7. The two-stage

analysis provisions were included in Section 12.2.3.2. In both the 2016 and 2022 ASCE 7 commentaries, the discussion addresses how to apply the provisions but does not address the technical background used to justify permitting the methodology.

ATC-3-06 Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC, 1978) does not appear to have addressed two-stage analysis. Subsequent versions of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings (NEHRP Provisions, FEMA, various), up through the 2003 Edition, did not include provisions for two-stage analysis. Starting with the 2009 Edition, the NEHRP Provisions adopted and amended ASCE 7. As a result of this adoption, ASCE 7 two-stage analysis provisions were incorporated into the NEHRP Provisions. No additional technical information providing the technical basis for two-stage analysis provisions has been found in the NEHRP Provisions.

Although limited explicit background on the theoretical basis of two-stage analysis is available, the procedure can be rationalized using structural-dynamics concepts. If the podium is so rigid that its deformation is negligible and it moves essentially as a rigid body with the ground, then the absolute acceleration at the top of the podium is approximately the ground acceleration at its base. Under this condition, the podium can be idealized as an effectively fixed base for the tower, making it reasonable to decouple the system: analyze the tower separately with a fixed base, then transfer the resulting interface forces to the podium for design. In practice, the provisions implement this rigid-podium approximation by specifying a minimum stiffness ratio (commonly expressed as an average story-stiffness ratio of at least 10 between the podium and the tower) as sufficient for the fixed-base idealization to be acceptable.

The only document found that implicitly reflects this concept is the SEAOC *Blue Book* (2019), which, when referring to SEAOC (1990), notes that the 1990 *Blue Book* introduced a two-stage analysis procedure permitting use of the linear static equivalent lateral force method when the lower system is stiff enough to act essentially as a fixed base for the upper system. While not stated explicitly in dynamic terms, this language aligns with the underlying assumption that podium flexibility is sufficiently small to justify decoupling the analyses.

Based on this review of code and standard history, limited information is available on the original intent and research that might have been used to justify the two-stage provisions.

RP4-3 The IT6 Survey Results, Issues, and Guidance

IT6 conducted a survey regarding the use of two-stage analysis, to which 12 entities responded. The responders were primarily engineering firms but also included building departments and other authorities having jurisdiction (AHJs). Information was provided on 24 projects designed using two-stage analysis. The following provides a summary of survey results. The complete survey results are tabulated in Appendix A.

RP4-3.1 Structure Systems, Configurations, and Occupancies

The survey responses included a broader range of structure size, range of SFRSs, and range of occupancies currently using two-stage analysis than had been anticipated by IT6. Overall, the information resulted in the understanding that the two-stage analysis is widely used across many structure and occupancy types. Table RP4-1 provides an overall summary of the 24 structures included in the survey. This collected data indicates that the broad design space needs to be considered when developing proposals to modify the two-stage analysis provisions.

It is noted that while the survey includes only one structure with a concrete shear wall tower and podium, the use of two-stage analysis for this combination is believed to be quite common and is an area of practice for several of the IT6 members.

Table RP4-1 Overall summary of IT-6 survey information

Systems	Number of Survey Responses (out of 24 total)	Occupancy	Number of Stories (Tower / Podium)
Wood light-frame over concrete shear wall	14	Many are residential plus commercial, but many are also mixed-use including storage, assembly, and business. Several are educational.	The majority are 5/2 or 5/3, with some at 2/1 and some at 6/4. Some are noted to have several podium stories and several below grade stories in addition.
CFS light-frame over concrete shear wall	1	Hotel	6/4
Mass timber over concrete shear wall	1	Residential	5/3 with one story of below grade basement.
Steel moment frame over concrete shear wall	5	Primarily institutional occupancy - Risk Category IV, some mixed-use	5/2, 6/1, 7/1, 1/4 + 33/4
Steel braced frame and moment frame over concrete shear wall	1	Institutional occupancy - Risk Category IV	8/1
Buckling restrained braced frame over concrete shear wall	1	Educational	3/2
Concrete shear wall over concrete shear wall	1	Residential	35/1-6 (This was a generic description of several structures designed by one of the responding firms.)

RP4-3.2 Reasons for Using Two-Stage Analysis

For each of the structures in the survey, the designer was asked why they used the two-stage analysis method. The responses fell into three main categories, outlined below.

The first reason for selecting two-stage analysis was to address a seismic design issue. When using ELF methods and a combined analysis of the tower and the podium structure, vertical distribution of the base shear per ASCE 7 Section 12.8.3 generates significant seismic demands in the tower. Seismic demand due to the mass of a heavier podium is imposed on the lighter and weaker stories above in the tower. This is particularly problematic for wood or CFS light-frame towers. Large podium plan footprints

(and mass) make this issue worse and untenable for the light-frame towers. Use of two-stage analysis results in separate vertical distributions for the tower and the podium, notably reducing design forces in the tower.

An analysis tool issue was the second reason for selecting two-stage analysis. Designers often use different analysis tools for the podium and the tower, even where towers are of concrete or steel construction. It is currently difficult or inefficient to model and design both tower and podium in the same analysis software. This is particularly true for wood and CFS light-frame construction, where most firms use spreadsheets or other in-house tools for seismic analysis of the tower (although commercial software is occasionally used).

A third reason for the use of two-stage analysis was identified as a design process issue. This issue occurs when structural design of the tower and the podium are conducted by two different design firms. This is fairly common in the design of wood and CFS light-frame towers over concrete podiums and allows each design firm to work in their area of specialty. This has also been seen on occasion where the SFRS of either tower or podium uses a proprietary system that has special analysis and design requirements.

Provided that the requirements for use of the two-stage analysis method are met, the above reasons for using two-stage analysis are reasonable and not of concern. Further discussion on the topic of vertical distribution of seismic demand will follow.

RP4-3.3 Calculation of Period and Stiffness

In response to the IT6 survey, significant variation was noted in the methods used to calculate period and stiffness to qualify for use of the two-stage analysis procedure under ASCE 7 Section 12.2.3.2, Items (a) and (b).

Regarding stiffness, survey respondents identified a wide range of methods used to determine stiffness and expressed concern regarding the range of methods used and the varying answers. Here it is noted that ASCE 7-22 included in Section 12.2.3.2 Item (a) additional clarification on the stiffness calculation method in order to provide increased consistency in implementation. This should reduce concerns going forward.

Regarding period, it is recognized that there are a wide range of methods available to calculate the period of the tower and the entire structure. Methods discussed included the approximate period formulas of ASCE 7 Section 12.8.2, eigenvalue analysis, Rayleigh method calculations based on FEMA 450 (2003 NEHRP Provisions) or the UBC through its 1997 edition. It was also noted that some firms were not checking the period criterion.

As part of issues related to period, it is generally recognized that using the ASCE 7 approximate period equation for both 1) the tower modeled separately and 2) the combined tower-plus-podium configuration makes it nearly impossible to satisfy the two-stage period-ratio limit. Because the same equation is applied to both the tower and the combined structure, the ratio $T_{(\text{tower}+\text{podium})}/T_{\text{tower}}$ is almost always greater than 1.1, even when the podium is designed to be comparatively stiff. This issue is illustrated in Figure RP4-1, which evaluates a structure with a one-story, 10-ft-tall podium across a range of tower heights. A 10-ft podium is a conservative benchmark in this context because it will typically be quite stiff relative to the light-frame tower. Yet the results show that meeting the 1.1 period-ratio requirement using the ASCE 7 approximate period equation would require a tower height on the order of 75 ft—a structural height (which is outside the height currently permitted for a wood light-frame tower under current building code provisions). Consistent with what was observed in example designs and reported by practitioners in the survey, designers commonly turn to alternative period-estimation approaches such as the Rayleigh method.

The use of a period ratio as a criterion for use of two-stage analysis is identified as needing further study. Topics include whether regulation of period ratio is required in addition to regulation of stiffness ratios and, if so, what methods are appropriate for determining periods for purposes of this criterion.

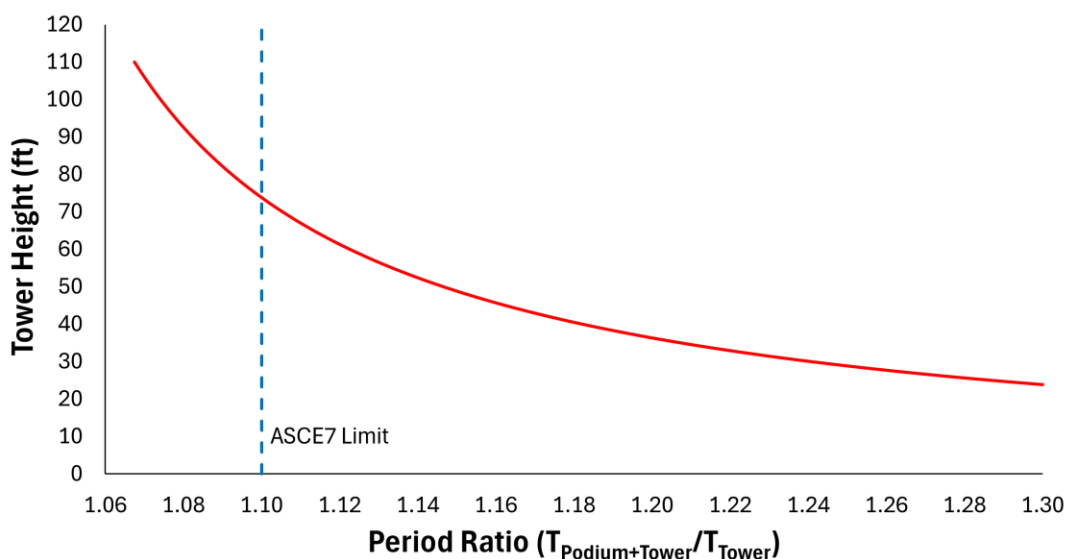


Figure RP4-1. Ratio of combined structure to tower approximate periods (T_a) as a function of tower (upper portion) height.

RP4-3.4 Base of Structure and Stories Below Grade

The survey asked for both the number of podium stories and the number of stories below grade, if applicable. In a number of survey responses, there was an indication of podium stories occurring both above and below grade. In some instances, the stories reported to be in the podium were the total of above and below grade. In other instances, the number of stories reported in the podium appeared to be only those above grade, with additional stories noted as being below grade basements. This difference in identification of the number of podium stories is likely related to the determination of the base of the

structure for purposes of analytical modeling (ASCE 7-22 Section 12.7.1 and Section 11.2 definition of base). This topic is currently being addressed in the 2026 NEHRP and ASCE 7-28 update work. In the IT6 studies, it was noted that while single-story podiums were generally quite stiff, the stiffness dropped significantly with additional podium stories, making it important that the correct number of podium stories be identified when checking eligibility to use the two-stage analysis procedure. Once the NEHRP and ASCE 7 update work is complete, it should be determined whether there is further guidance needed regarding identification of podium stories for purposes of qualifying for the two-stage analysis method.

RP4-3.5 Tower Extending Off of the Podium

The survey asked for approximate plan dimensions of the tower and podiums. One survey response indicated a tower having a footprint area larger than the supporting podium, implying that the tower is sitting partially on a podium and partially on foundation at grade level (Figure RP4-2) or a similar building configuration. Members of IT6 confirmed that this building configuration is currently being designed using two-stage analysis. This topic is currently being addressed in the ASCE 7-28 update process. It is being discussed that each tower designed using the two-stage procedure should be laterally braced exclusively by the podium on which it is supported. For the structure to have the dynamic response for which this procedure is intended, the tower(s) should have seismic lateral forces transmitted to the tower exclusively through the podium. Where the towers extending off the podium include only gravity system elements and not vertical elements of the SFRS, the dynamic response of the tower and podium is believed to remain reasonably aligned with two-stage analysis assumptions. Where vertical elements of the SFRS are included in the portion of the tower that extends off the podium, duplicative seismic force load paths (one through the podium and a second through the on-grade foundation and soils supporting the tower extension) make the dynamic response of this structure significantly more complex, and beyond what is properly considered in the two-stage analysis procedures.

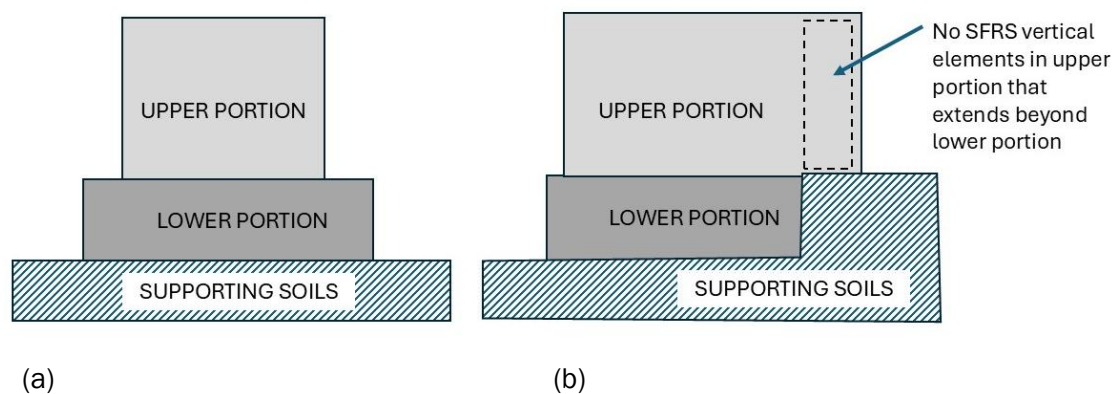


Figure RP4-2. Tower supported on and extending off of the podium (a) tower supported completely on the podium and (b) tower extending off of the podium.

RP4-3.6 Irregularities

Discussion initiated by the survey indicated that some ASCE 7 defined seismic irregularities are common in buildings that qualify for two-stage analysis. One irregularity of particular concern is a mass irregularity (Vertical Irregularity Type 2 under ASCE 7-16). A second irregularity of particular concern is a torsional irregularity (Horizontal Irregularity Type 1). Also, of note, was the triggering of the Out-of-Plane Offset (Horizontal Irregularity Type 4) and In-Plane Discontinuity (Vertical Irregularity Type 3).

The mass irregularity was of particular concern under ASCE 7-16 because triggering of this irregularity meant that per Table 12.6-1 “all other structures” was applicable under the “structural characteristics” column, meaning that ELF analysis was prohibited, while Section 12.2.3.2 Item (e) required ELF for the podium when using two-stage analysis. This concern was eliminated as a result of two significant changes between ASCE 7-16 and 7-22. First, the mass irregularity was deleted. Second, Table 12.6-1 and the prohibition of ELF analysis was dropped. Analytical studies identified that buildings with mass irregularities designed to ASCE 7 did not experience probability of collapse higher than those without mass irregularities. Further, collapse performance of buildings designed using ELF was identified to meet or exceed performance of paired buildings designed using MRSA. Under ASCE 7-22 and later editions, mass irregularities should no longer be a concern.

Regarding torsional irregularities, the ASCE 7-22 deletion of Table 12.6-1 and resulting allowance for use of ELF for structures with torsional irregularities has also addressed this concern.

Regarding irregularities due to out-of-plane and in-plane offsets of vertical elements of the SFRS, it is recognized that these are commonly occurring in buildings with vertical combinations of systems. The offsets generally occur at the top of the podium. Out-of-plane offsets in the vertical elements of the SFRS are recognized to result in significant transfer forces through the diaphragm. It is recognized that in transferring the horizontal seismic base shear from the tower through the podium structure, the horizontal seismic force needs to be multiplied by the ratio of R/ρ for the systems above and below. In addition, diaphragm transfer forces are required by Sections 12.10.1 and 12.10.3 to be amplified. Further, Section 12.3.3.4 requires that the gravity system supporting discontinued SFRS vertical elements be designed for Ω_0 forces. Each of these effects are triggered by different concerns, and all are intended to be applied simultaneously.

RP4-3.7 Stepped Podiums

It has been reported that two-stage analysis has been used for structures with top of podium configurations that vary up to a full story height in elevation (i.e., have vertical steps in the podium) within the footprint of a tower, as seen in Figure RP4-3. This topic is currently being addressed in the ASCE 7-28 update process. It is necessary that there be a reasonably level horizontal plane at which the transition between the SFRS of the tower and the podium occur, in order for the dynamic behavior of each portion to be relatively independent, such that two-stage analysis can be applicable. To achieve this, it is proposed that over the footprint of each tower, a general rule exists that requires a constant top of podium elevation. To address common configurations encountered in podium structures, a first exception is suggested that allows the elevation at which the tower vertical elements of the SFRS are

supported on the podium to vary by not more than the lesser of 3 ft. or one-quarter the story height of the lowest story in the tower. A second exception recognizes that this clear transition zone is necessary at the vertical elements of the SFRS, but not for the gravity system, which is only required to accommodate seismic drift. The second exception permits large steps in the podium provided the base of all SFRS vertical elements is set at the highest podium elevation (upper portion, right side in Figure RP4-3). The allowance of one-quarter the story height or 3 ft. variation in elevation recognizes that design requirements other than structural often force some articulation within the footprint of the tower. Variations in elevation at the tower to podium interface, including but not limited to full-story steps at the base of vertical element elevation, complicate dynamic interaction of the tower and the podium and therefore fall outside of the intended use of the two-stage analysis procedure. For the building shown in Figure RP4-3, an alternative (not shown in the figure) would be to provide a vertical seismic joint in the tower, aligned with the step in the podium.

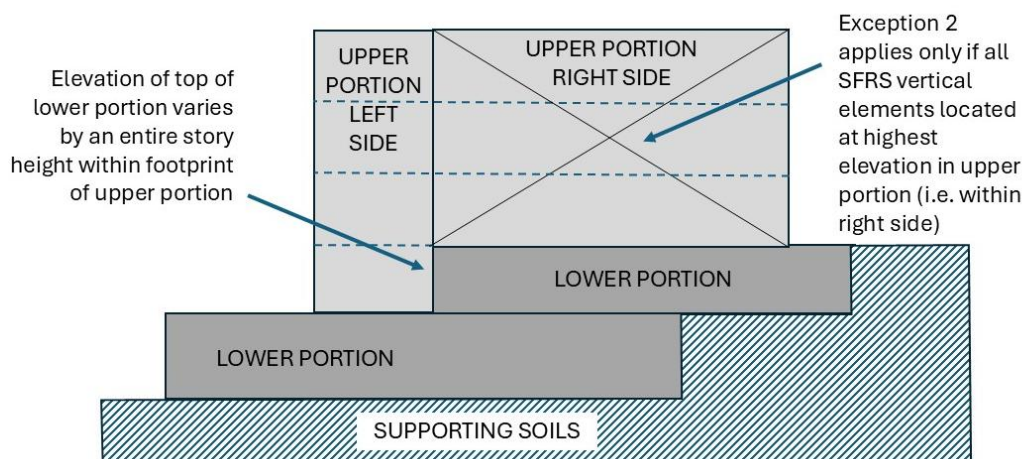


Figure RP4-3. Vertical step within the footprint of an upper portion. Two-stage analysis is permitted if vertical elements of the SFRS are only on one side of the podium step (either left side only or right side only) but not if they are on both sides.

RP4-4 Dynamic Interaction of Tower and Podium

As part of the IT6 work, analytical studies were performed to look in detail at the dynamic interaction of towers and podiums. This was identified by IT6 as one of the primary outstanding issues regarding the use of two-stage analysis. The study was done with the intent of 1) revisiting the stiffness and period criteria for use to qualify for use of two-stage analysis, and 2) determining if there are building configurations for which two-stage analysis should be specifically prohibited or specifically required. Several discussions resulting from the studies follow.

The stiffness ratio in Section 12.2.3.2 Item (a), currently specified as 10, was investigated in analytical studies of buildings designed using the two-stage analysis procedure. The driving concept of the study

was that the use of the two-stage analysis should be permitted where it results in seismic performance of the tower as part of the combined structure reasonably similar to the performance of the tower when analyzed as a stand-alone structure. Starting from an initial limited study by McBain (2023), Flores et al. (2025) conducted a parametric study of tower and podium interaction that included a five-story wood light-frame shear wall tower (bare structure model) over a two-story concrete podium modeled with varying stiffness. Nonlinear response history analysis was used to compare demands and the seismic collapse performance of a stand-alone tower on a fixed base to the demands and seismic performance with the tower modeled in combination with the concrete podium. With a podium-to-tower stiffness ratio of 10, the analysis indicated significant dynamic interaction between the tower and the podium. With a stiffness ratio of 30, the dynamic interaction was reduced to the point that the overall collapse performance and forces in the tower of the combined structure were similar to those of the stand-alone tower with a fixed base. Figure RP4-4 from Flores et al. (2025) shows the probability of collapse at maximum considered earthquake (MCE_R) dropping below 10% (used as a performance target for NEHRP Provisions and ASCE 7-22) for both elastic and inelastic concrete podium models with a stiffness ratio of 30. The stiffnesses that were used to characterize both the tower and podium were initial stiffnesses from the nonlinear response-history analysis. To investigate the validity of using initial stiffness to characterize stiffness ratios for purposes of the two-stage analysis, a separate code-based design consistent with standard practice was performed with the same building weights, and seismic design parameters adopted in the nonlinear model. SDPWS 2021 procedures were then applied to compute lateral stiffness for two shear-wall lengths (10 ft and 20 ft). The resulting SDPWS-based stiffness estimates were in the same range as those obtained from the nonlinear model, confirming validity. A similar recommendation of stiffness ratio closer to 30 was also made in a report on the Prescient system (Harris, 2023-2025). This was the result of in-depth analytical study of this structural system and lends support to the Flores et al. studies.

Another parameter studied by Flores et al. (2025) was the difference between the ground acceleration and the absolute (total) acceleration at the top of the podium. Comparing these two inputs led to several notable observations. First, the absolute acceleration at the top of the podium decreases when the podium yields, indicating that inelastic podium response can reduce transmitted accelerations. This was anticipated based on prior research on floor accelerations affecting nonstructural components.

Second, the absolute acceleration at the top of the podium exhibits amplified peaks relative to the ground acceleration. In addition, the podium alters the frequency content of the transmitted motion: the acceleration at the top of the podium develops a predominant frequency that matches the podium's natural frequency, reflecting the podium's filtering effect on the incoming ground motion. This shift in frequency content is potentially important because it may increase demand associated with tower modes resonant with the modified input spectrum, particularly higher modes. That possibility is especially relevant due to the application of two-stage analysis to high-rise buildings, where higher-mode effects can become significant. It merits further targeted investigation.

While the IT6 analysis provided an informative first study, the PUC identified that more work was needed to definitively substantiate increasing the ASCE 7 stiffness ratio requirement for two-stage analysis, since such a change may significantly alter current design practice. It was also suggested that soil-structure interaction could significantly influence dynamic response and needed to be addressed in future two-stage analysis studies.

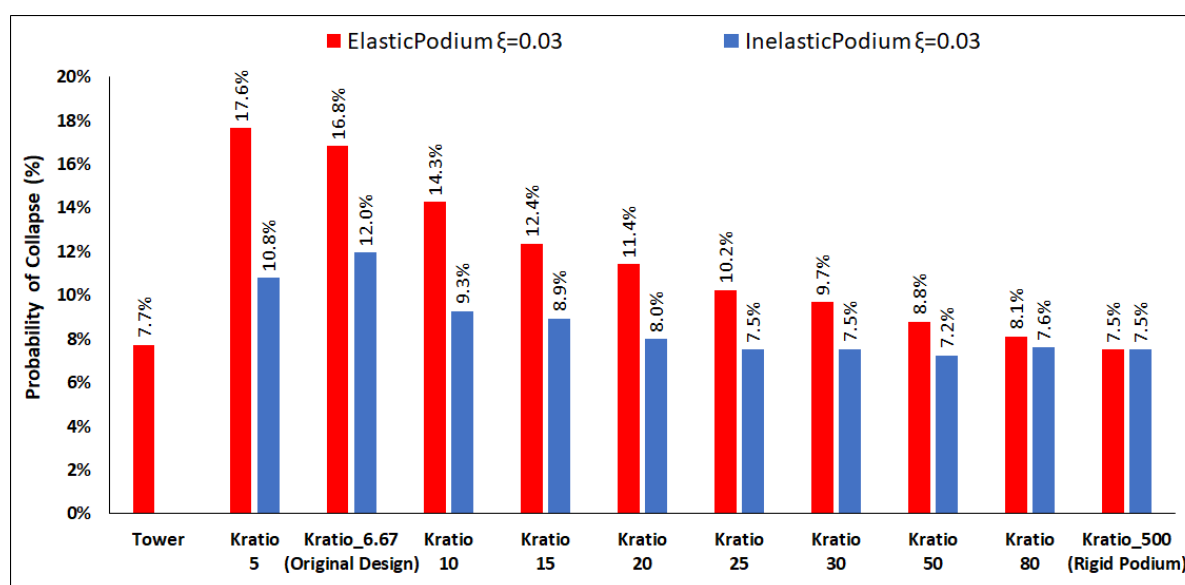


Figure RP4-4. Probability of collapse at MCE_R for buildings designed using the two-stage analysis procedure based on FEMA P-695 studies.

From studies of two-stage analysis conducted by Harris (2023-2025), it was identified that there are common design situations where two-stage analysis captures seismic demand better than combined analysis of the tower and the podium. When including a very stiff podium and flexible tower in a combined analysis, the period assigned to the combined building (using either approximate period formulas or other methods) will be significantly influenced by the flexible tower, resulting in periods and base shears that are more representative of the tower structure. Conversely, if designed using two-stage analysis, the short period and associated base shear for the podium structure will be recognized, resulting in an appropriately larger base shear. Comparison to nonlinear response-history analysis indicates that for this type of structure, the two-stage analysis better captures seismic demands in the podium, while combined analysis using ELF or MRSA methods tends to underestimate these demands. As an example, assume a ten-story tower with an estimated period of 2 seconds, a podium with a period of 0.1 seconds, and an estimated combined period of 2.06 seconds. When using two-stage analysis, a base shear assigned using the podium period of 0.1 seconds will much better match the podium shear demands than using a combined analysis based on the 2.06 second period.

RP4-5 Multiple Towers on a Single Podium

Of the 24 structures in the IT6 survey, five were noted to have multiple towers on a single podium. These included wood light-frame towers, steel moment frames, and several concrete shear wall towers. Occurrence of multiple towers on a single podium was identified by IT6 as commonly occurring across all materials and systems. It was noted, however, that ASCE 7 Section 12.2.3.2 is written envisioning a single tower. It does not adequately address criteria or implementation with multiple towers, resulting in varied design practice. Concerns generated by multiple towers were wide-ranging and included the possibility of out-of-phase response of the towers generating very significant forces in the podium transfer slabs, as well

as the possibility of some towers being occupied before all of the towers were constructed, generating a need to provide analysis considering only a portion of the towers complete.

Members of IT6 most familiar with design for multiple towers on a podium, based on their collective experience, developed the following analysis recommendations:

- An equivalent lateral force (ELF) procedure or a Modal Response Spectrum Analysis (MRSA) should be permitted in lieu of Linear Response History Analysis (LRHA) or Non-linear Response History Analysis (NLRHA) for the design of multiple towers (upper structures or upper portions) on a single podium.
- A separate analysis should be performed for each tower, without the podium,
- An analysis should be performed including all towers and podiums, and
- Separate analyses should be performed for each tower combined with the podium, with the base shear at grade scaled to ELF base shear in accordance with Section 12.8.

Unless specifically justified through analysis, the entire podium should be included in the combined structure model. When modeling of less than the full podium structure is justified, no less than two spans of the podium structure outside the footprint of the tower should be included in the analytical model. The podium structure should be designed for maximum force levels considering each analysis. For the elements at the interface of the towers and the podium, a combination of forces considering the directionality of the towers should be accounted for (+/+, +/-, -/-, -/+) in each direction. Members should be designed for 100 percent of the force for one direction plus 30 percent of the force in the perpendicular direction. The mass and stiffness irregularities should be evaluated considering the torsional sensitivity of the podium. If mass and stiffness irregularities exist, NLRHA is required.

In a separate but related effort, IT6 members undertook a limited study regarding use of the two-stage analysis and scaling of MRSA forces for buildings with more than one tower on a podium. It is pointed out that guidance is not provided for base shear scaling with multiple towers. Further investigation across the two-stage analysis design space is needed to generate broad guidance.

RP4-6 Recommendations for Future Study of Two-Stage Analysis

During IT6 work, it was identified that a notable and growing portion of the current building stock uses vertical combinations of SFRSs. Several questions exist, however, regarding the criteria used to qualify for use of two-stage analysis and appropriate implementation of the two-stage analysis provisions. These questions raise issues regarding whether structures designed using two-stage analysis will meet seismic performance targets intended by the NEHRP Provisions and ASCE 7. It is suggested that either further study occurs to better define appropriate scoping criteria and implementation of two-stage analysis, or that limitations of the materials and systems permitted to use two-stage analysis be considered. The following topics are recommended for further study:

- Both the stiffness and period criteria for the use of two-stage analysis across materials and SFRSs. Studies recommending changes to the stiffness ratio should demonstrate not only the change in dynamic interaction with varying ratios, but also that changing the ratio would result in meaningful changes in design and resulting performance. Soil-structure interaction (SSI) is identified as an important consideration in future studies. It is recommended that future stiffness and period criteria studies include study of SSI effects and provide specific direction to the designer regarding inclusion of SSI in modeling when evaluating two-stage analysis criteria as well as inclusion in the resulting analysis and design. Also important in future studies is appropriately capturing the range of and the anticipated dynamic behavior of SFRSs used for both the tower and the podium.
- The need for and the possible elimination of a period criterion. It should be evaluated whether a period criterion is needed in addition to a stiffness criterion to achieve intended performance targets. Based on the widely varying methods available to calculate the periods, if a period criterion is to be maintained, guidance is needed on how periods are to be calculated for purposes of this criterion.
- Whether combined analysis models underestimate seismic demand because periods used in calculating the base shear do not properly capture the high stiffness of the podium, thereby underestimating the podium story shear. If podium story shears are being underestimated, should analysis requirements be added to compensate for this?
- Establishment of criteria for use of two-stage analysis with multiple towers on a single podium.
- Establishment of MRSA analysis and base shear scaling procedures where multiple towers are supported on a single podium.
- Identification of the base of structure for analysis purposes. This relates to the occurrence of below grade basement stories, and whether these stories should be included in modeling of the podium for purposes of determining eligibility to use two-stage analysis provisions.
- According to the study “Two-Stage Analysis Loophole” (Shepherd and McDonald, 2024), a podium with extreme torsional irregularity can induce a torsional response in the tower, even when the latter does not have extreme torsional irregularity. In current two-stage analysis practice, designers may effectively neglect torsional effects transferred from an irregular podium. Because of this, the original two-stage analysis requirement that “each portion of the structure, when considered separately, is classified as regular”—removed in the 2000 IBC—should be reconsidered.
- The seismic performance implications of podium filtering that can shift the ground motion’s frequency content toward the podium’s natural frequency, potentially amplifying higher-mode response in high-rise towers.

RP4-8 Acknowledgements

Thank you to the members of IT6 for undertaking this study of two-stage analysis and related issues. The diverse building design experience of this group contributed to substantive discussion of the range of design using this method and the resulting issues.

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RP4-10 References

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RP4-Appendix A: Two-Stage Analysis Survey

General	IT6 Building Identifier	1a	1b	1c
	Building Description:	5 (Wood) over 2 (Concrete) Mixed Residential + Retail	5 (Wood) over 2 (Concrete) Mixed Residential + Retail	<i>5 (Wood) over 2 (Concrete) Mixed Residential + Retail</i>
	Occupancy(s)	Residential + Retail	Residential + Retail	Residential + Retail
	Podium structural system/seismic system/ R-factor	6	6	5
	Podium number of stories	<i>3 (2 above ground, 1 below)</i>	<i>2 (2 above ground, 0 below)</i>	2
	Podium number of stories below ground (if applicable)	1	NA	3
	Podium approximate plan dimensions	35,420 sf	48,600 sf	29,200 sf
	Tower structural system/ seismic system/ R-factor	6.5	6.5	6.5
	Number of towers on one podium	1	1	1
	Tower(s) number of stories	5	5	5
	Tower(s) approximate plan dimensions	33,910 sf	32,000 sf	24,600 sf
	Structural irregularities or other unique configurations	NA	NA	NA
	Governing building code/ ASCE 7 edition	ASCE7-10	ASCE7-10	ASCE7-10
	Did podium and tower have same EOR or different?	Same	Same	Same
Analysis Basics	Was two-stage analysis considered for the analysis?	Yes	Yes	Yes
	If used, why?	This is the methodology used when wood buildings are designed on top of a podium	This is the methodology used when wood buildings are designed on top of a podium	This is the methodology used when wood buildings are designed on top of a podium
	If decided against, why?	NA	NA	NA
	What ASCE 7 criterion/criteria for the two stage method were found difficult to satisfy? If so, what were the difficulties?	Stiffness criterion (ASCE7-10 12.2.3.2a) was difficult to achieve	Stiffness criterion (ASCE7-10 12.2.3.2a) was difficult to achieve	Stiffness criterion (ASCE7-10 12.2.3.2a) was difficult to achieve
	Was the podium torsionally irregular and if so, how was the torsional irregularity of the podium addressed in the podium and in the tower?	NA	NA	NA

	IT6 Building Identifier	1a	1b	1c
	Building Description:	5 (Wood) over 2 (Concrete) Mixed Residential + Retail	5 (Wood) over 2 (Concrete) Mixed Residential + Retail	5 (Wood) over 2 (Concrete) Mixed Residential + Retail
	Did the placement of the tower on the podium create torsion? If so, how was that response analyzed in the tower and the podium?	NA	NA	NA
	The following two questions about analysis pertain if the two stage method was used.			
	What analysis method was used for the tower? ELF or Response Spectrum	ELF	ELF	ELF
	If MRS was used, was the primary mode of vibration torsion or translation? If torsion, how did you demonstrate the	NA	NA	NA
	Have you ever used a method that rendered results that didn't make sense, such as the period of the combined structure being less than the upper structure alone? If so, which method was it?	NA	NA	NA
PERIOD OF VIBRATION (12.2.3.2b ASCE7-22)	How was the period of vibration of the PODIUM calculated?	Via ETABS analysis	Via ETABS analysis	Via ETABS analysis
	How was the period of vibration of the TOWER calculated?	ASCE7-10 Equation 12.8-7	ASCE7-10 Equation 12.8-7	ASCE7-10 Equation 12.8-7
	How was the period of vibration of the PODIUM + TOWER calculated?	Not computed	Not computed	Not computed
	PODIUM Period of Vibration from the model (Seconds)	0.33	0.28	0.22
	TOWER Period of Vibration from the model (Seconds)	NA	NA	NA
	PODIUM approximate period (Seconds)	NA	NA	NA
	TOWER approximate period (Seconds)	0.46	0.38	0.32
STIFFNESS (12.2.3.2a ASCE7-22)	How was the stiffness of the PODIUM calculated?	Via ETABS analysis	Via ETABS analysis	Via ETABS analysis
	How as the stiffness of the TOWER calculated?	Displacements per AWC SDPWS-2008 using ELF forces	Displacements per AWC SDPWS-2008 using ELF forces	Displacements per AWC SDPWS-2008 using ELF forces
	How as the stiffness of the TOWER calculated?	Displacements per AWC SDPWS-2008 using ELF forces	Displacements per AWC SDPWS-2008 using ELF forces	Displacements per AWC SDPWS-2008 using ELF forces
Special Considerations	What analysis/detailing methods were used to qualify a building that might not otherwise qualify?	NA	NA	NA
	Was the method used to analyze a structure where the interface is stepped? When the interface is at more than one level?	NA	NA	NA
SOFTWARE	What Software was mainly used to design the PODIUM, the TOWER and the combined building? Why was that software selected?	ETABS, S-Concrete and in-house Excel design workbooks for the Podium LFRS, and in-house Excel design workbooks for the Tower/Wood Frame LFRS	ETABS, S-Concrete and in-house Excel design workbooks for the Podium LFRS, and in-house Excel design workbooks for the Tower/Wood Frame LFRS	ETABS, S-Concrete and in-house Excel design workbooks for the Podium LFRS, and in-house Excel design workbooks for the Tower/Wood Frame LFRS

	IT6 Building Identifier	1a	1b	1c
	Building Description:	5 (Wood) over 2 (Concrete) Mixed Residential + Retail	5 (Wood) over 2 (Concrete) Mixed Residential + Retail	<i>5 (Wood) over 2 (Concrete) Mixed Residential + Retail</i>
Did you COMPARE between two-stage analysis and nonlinear time-history analysis?	What are the differences?	NA	NA	NA
	Are there any cases where two-stage analysis should not be used or could lead to unconservative designs?	NA	NA	NA
	Would you be able to provide a summary of your finding or present it to our task group?	NA	NA	NA
Multiple Towers	Does your project have multiple towers on a single base?	No	No	No
	If so, what is the number of towers and the number of stories in each tower?	NA	NA	NA
	What two-stage analysis difficulties were encountered due to the multiple towers and how were they resolved?	NA	NA	NA
	Did the analysis consider possible in-phase and out-of-phase behavior of the towers?	NA	NA	NA
	Was there any special consideration of podium diaphragm forces that might be generated between the towers?	NA	NA	NA
Analysis Modeling and Boundary Conditions	Was tower model treated as fixed base?	Yes	Yes	Yes
	Was podium model treated as fixed base?	Yes	Yes	Yes
	If podium has basement, what was treated as the base?	The lower most level (daylight basement)	The lower most level (daylight basement)	Street level
	Were soil springs used? What location and type?	Soil springs were considered strictly for the mat foundation design beneath the concrete shear walls (compression only springs corresponding with the design subgrade modulus provided by the geotechnical engineer)	Soil springs were considered strictly for the mat foundation design beneath the concrete shear walls (compression only springs corresponding with the design subgrade modulus provided by the geotechnical engineer)	Soil springs were considered strictly for the mat foundation design beneath the concrete shear walls (compression only springs corresponding with the design subgrade modulus provided by the geotechnical engineer)
	Were any aspects of modeling or boundary conditions particularly of concern or particularly influential on the loads or displacements?	Building drift for the tower (wood frame) portion was right at the code level limit of 2%.	Building drift for the tower (wood frame) portion was right at the code level limit of 2%.	Building drift for the tower (wood frame) portion was right at the code level limit of 2%.

General	IT6 Building Identifier	1d	1e	2a
	Building Description:	Five-Story over Podium Slab	Five-Story over Podium Slab	Typical Wood Residential over Concrete Podium
	Occupancy(s)	Podium: S-1,S-2, B, A-2,A-3 Tower: R-2	Podium: S-1,S-2, B, A-2,A-3 Tower: R-2	Residential, Ground Floor Retail
	Podium structural system/seismic system/ R-factor	6	6	Concrete Shear Walls, R=5
	Podium number of stories	5 (2 above ground, 3below)	3	1 to 3
	Podium number of stories below ground (if applicable)	2	2	0 to 2
	Podium approximate plan dimensions	66300 SF	64000 SF	100x100 to 350x250
	Tower structural system/ seismic system/ R-factor	6.5	6.5	Light Frame Wood Shear Walls, R=6.5
	Number of towers on one podium	2	1	1 to 3
	Tower(s) number of stories	5	5	4 to 6
	Tower(s) approximate plan dimensions	22200	52500 SF	100x100 to 300x100
	Structural irregularities or other unique configurations	NA	NA	Frequently Torsionally Irregular, often Extreme
	Governing building code/ ASCE 7 edition	ASCE7-16	ASCE7-10	ASCE 7-16
	Did podium and tower have same EOR or different?	Same	Same	Same
Analysis Basics	Was two-stage analysis considered for the analysis?	yes	yes	Yes
	If used, why?	Wood structure over concrete podium	Wood structure over concrete podium	Too difficult to model both structures in one analytical model
	If decided against, why?	NA	NA	-
	What ASCE 7 criterion/criteria for the two stage method were found difficult to satisfy? If so, what were the difficulties?	Period determination using approximate fundamental period equations per ASCE. Period of upper wood structure = 0.98. Period of entire structure = 1.0. Therefore, ratio is 1.0 <1.10 which is acceptable.	Period determination using approximate fundamental period equations per ASCE. Period of upper wood structure = 0.44. Period of entire structure = 0.27. Therefore, ratio is 0.62<1.10	It is not typically challenging to resolve two-stage analysis issues for most buildings of this type. Occasionally the stiffness ratio and period ratio are close, mostly when the lower structure is relatively flexible.
	Was the podium torsionally irregular and if so, how was the torsional irregularity of the podium addressed in the podium and in the tower?	No	No	If the podium is irregular, which it almost always is, this is addressed by applying Ax*5% accidental eccentricity in the model of the lower structure. This would not be applied to the upper structure. Similarly, finding that the upper or lower structure needs rho=1.3 would be an independent study - just because one model needs rho would not automatically be applied to both buildings

	IT6 Building Identifier	1d	1e	2a
	Building Description:	Five-Story over Podium Slab	Five-Story over Podium Slab	Typical Wood Residential over Concrete Podium
	Did the placement of the tower on the podium create torsion? If so, how was that response analyzed in the tower and the podium?	NA	NA	No, I would more attribute the torsional issues due to complex lateral system layout of the lower structures
	The following two questions about analysis pertain if the two stage method was used.			
	What analysis method was used for the tower? ELF or Response Spectrum	ELF	ELF	Exclusively ELF
	If MRS was used, was the primary mode of vibration torsion or translation? If torsion, how did you demonstrate the	NA	NA	We do not use MRSA for wood structures
	Have you ever used a method that rendered results that didn't make sense, such as the period of the combined structure being less than the upper structure alone? If so, which method was it?	NA	Yes	This has not been an issue
PERIOD OF VIBRATION (12.2.3.2b ASCE7-22)	How was the period of vibration of the PODIUM calculated?	Computer FEA model (ETABS)	Computer FEA model (ETABS)	Using ETABS model analysis of the rigid lower structure
	How was the period of vibration of the TOWER calculated?	First check using Eq. 12.8-7. ASCE7-16 then checked using FEMA 450 Eq. C5.2-1, Eq 30-10 1997 UBC to satisfy requirement. Followed steps using Woodworks example.	First check using Eq. 12.8-7. ASCE7-10 then checked using FEMA 450 Eq. C5.2-1, Eq 30-10 1997 UBC to satisfy requirement. Followed steps using Woodworks example.	Using an eigenvalue analysis in a spreadsheet based on story stiffness and mass
	How was the period of vibration of the PODIUM + TOWER calculated?	FEMA 450 Eq. C5.2-1, Eq 30-10 1997 UBC to satisfy requirement to use two stage analysis	FEMA 450 Eq. C5.2-1, Eq 30-10 1997 UBC to satisfy requirement to use two stage analysis	Using an eigenvalue analysis in a spreadsheet based on story stiffness and mass
	PODIUM Period of Vibration from the model (Seconds)	Computer FEA model (ETABS)	Computer FEA model (ETABS)	Generally 0.3-0.6 seconds
	TOWER Period of Vibration from the model (Seconds)	Not modeled	Not modeled	Generally 0.8-1.1 seconds
	PODIUM approximate period (Seconds)	0.4s	0.29s	Generally 0.25 seconds
	TOWER approximate period (Seconds)	0.98s	0.44s	Generally 0.45 seconds
STIFFNESS (12.2.3.2a ASCE7-22)	How was the stiffness of the PODIUM calculated?	From FEA model output	From FEA model output	Using ETABS model analysis of the rigid lower structure
	How as the stiffness of the TOWER calculated?	Hand calculation/spreadsheet. Calculated forces (ELF)and elastic deflection on wood shearwall line. The displacements per story are calculated using SDPWS. Hand calculation/spreadsheet. Calculated forces (ELF)and elastic deflection on wood shearwall line. The displacements per story are	Hand calculation/spreadsheet. Calculated forces (ELF)and elastic deflection on wood shearwall line. The displacements per story are calculated using SDPWS. Hand calculation/spreadsheet. Calculated forces (ELF)and elastic deflection on wood shearwall line. The displacements per story are	Using a RISA model of the upper structure
	How as the stiffness of the TOWER calculated?			Assuming you mean combined structure, in a spreadsheet using an eigenvalue analysis based on story stiffness and mass
Special Considerations	What analysis/detailing methods were used to qualify a building that might not otherwise qualify?	NA	NA	N/A
	Was the method used to analyze a structure where the interface is stepped? When the interface is at more than one level?	NA	NA	An envelope approach lumping mass conservatively and hardening diaphragms
SOFTWARE	What Software was mainly used to design the PODIUM, the TOWER and the combined building? Why was that software selected?	ETABS to model the podium only. The tower is not modeled.	ETABS to model the podium only. The tower is not modeled.	PODIUM: ETABS - easy for concrete TOWER: RISA - easy for wood COMBINED: Excel - all we need

	IT6 Building Identifier	1d	1e	2a
	Building Description:	Five-Story over Podium Slab	Five-Story over Podium Slab	Typical Wood Residential over Concrete Podium
Did you COMPARE between two-stage analysis and nonlinear time- history analysis?		NA	NA	No
	What are the differences?			
	Are there any cases where two-stage analysis should not be used or could lead to unconservative designs?	NA	NA	Unknown!
	Would you be able to provide a summary of your finding or present it to our task group?	NA	NA	
Multiple Towers	Does your project have multiple towers on a single base?	No	No	Frequently
	If so, what is the number of towers and the number of stories in each tower?	NA	NA	2-3 towers on one base
	What two-stage analysis difficulties were encountered due to the multiple towers and how were they resolved?	NA	NA	The entire lower structure stiffness is compared to each upper structure. No real way around this.
	Did the analysis consider possible in- phase and out-of-phase behavior of the towers?	NA	NA	Occasionally, but rarely.
	Was there any special consideration of podium diaphragm forces that might be generated between the towers?	NA	NA	Occasionally, but rarely.
Analysis Modeling and Boundary Conditions	Was tower model treated as fixed base?	NA	NA	Yes
	Was podium model treated as fixed base?	No. Pinned	No. Pinned	Yes
	If podium has basement, what was treated as the base?	Ground level as seismic base.	Ground level as seismic base.	The bottom of the basement
	Were soil springs used? What location and type?	No	No	No
	Were any aspects of modeling or boundary conditions particularly of concern or particularly influential on the loads or displacements?	NA	NA	No

General	IT6 Building Identifier	2b	3	
	Building Description:	Mass Timber over Concrete Structure	Groups	
	Occupancy(s)	Residential		
	Podium structural system/seismic system/ R-factor	Concrete Shear Walls, R=5	Concrete shear wall R = 5 for 5/2, R=6 building frame conc shear wall	
	Podium number of stories	3	1 to 2	Run into issues with more stories
	Podium number of stories below ground (if applicable)	1		
	Podium approximate plan dimensions	119'x119'		
	Tower structural system/ seismic system/ R-factor	Light Frame Wood Shear Walls, R=6.5		
	Number of towers on one podium	1		
	Tower(s) number of stories	5	up to 5	Run into issues with more stories
	Tower(s) approximate plan dimensions	119'x119'		
	Structural irregularities or other unique configurations	Extreme torsional irregularity		
	Governing building code/ ASCE 7 edition	ASCE 7-16		
	Did podium and tower have same EOR or different?	Same		
Analysis Basics	Was two-stage analysis considered for the analysis?	Yes		
	If used, why?	Too difficult to model both structures in one analytical model		
	If decided against, why?	-		
	What ASCE 7 criterion/criteria for the two stage method were found difficult to satisfy? If so, what were the difficulties?	Didn't find too difficult to meet criteria for this project		Used woodworks document - helped with problem buildings
	Was the podium torsionally irregular and if so, how was the torsional irregularity of the podium addressed in the podium and in the tower?	rho of 1.3 was used. Accidental torsional moment was amplified		

	IT6 Building Identifier	2b	3	
	Building Description:	Mass Timber over Concrete Structure	Groups	
	Did the placement of the tower on the podium create torsion? If so, how was that response analyzed in the tower and the podium?	No		
	The following two questions about analysis pertain if the two stage method was used.			
	What analysis method was used for the tower? ELF or Response Spectrum	ELF		
	If MRS was used, was the primary mode of vibration torsion or translation? If torsion, how did you demonstrate the	We do not use MRSA for wood structures		
	Have you ever used a method that rendered results that didn't make sense, such as the period of the combined structure being less than the upper structure alone? If so, which method was it?	This was not an issue		
PERIOD OF VIBRATION (12.2.3.2b ASCE7-22)	<u>How was the period of vibration of the PODIUM calculated?</u>	Using ETABS model analysis of the rigid lower structure		
	<u>How was the period of vibration of the TOWER calculated?</u>	Using an eigenvalue analysis in a spreadsheet based on story stiffness and mass		
	<u>How was the period of vibration of the PODIUM + TOWER calculated?</u>	Eigenvalue analysis		
	PODIUM Period of Vibration from the model (Seconds)	0.51 in X & 0.63 in Y		
	TOWER Period of Vibration from the model (Seconds)	0.9		
	PODIUM approximate period (Seconds)	0.26		
	TOWER approximate period (Seconds)	0.41		
STIFFNESS (12.2.3.2a ASCE7-22)	<u>How was the stiffness of the PODIUM calculated?</u>	Using ETABS model analysis of the rigid lower structure		
	<u>How as the stiffness of the TOWER calculated?</u>	Using a RISA model of the upper structure		
	<u>How as the stiffness of the TOWER calculated?</u>	Assuming you mean combined structure, in a spreadsheet using an eigenvalue analysis based on story stiffness and mass		
Special Considerations	What analysis/detailing methods were used to qualify a building that might not otherwise qualify?	N/A		
	Was the method used to analyze a structure where the interface is stepped? When the interface is at more than one level?	N/A		
SOFTWARE	What Software was mainly used to design the PODIUM , the TOWER and the combined building ? Why was that software selected?	PODIUM: ETABS - easy for concrete TOWER: RISA - easy for wood COMBINED: Excel - all we need		

	IT6 Building Identifier	2b	3	
	Building Description:	Mass Timber over Concrete Structure	Groups	
Did you COMPARE between two-stage analysis and nonlinear time-history analysis?		NA		
	What are the differences?			
	Are there any cases where two-stage analysis should not be used or could lead to unconservative designs?	NA		
	Would you be able to provide a summary of your finding or present it to our task group?	NA		
Multiple Towers	Does your project have multiple towers on a single base?	No		
	If so, what is the number of towers and the number of stories in each tower?	NA		
	What two-stage analysis difficulties were encountered due to the multiple towers and how were they resolved?	NA		
	Did the analysis consider possible in-phase and out-of-phase behavior of the towers?	NA		
	Was there any special consideration of podium diaphragm forces that might be generated between the towers?	NA		
Analysis Modeling and Boundary Conditions	Was tower model treated as fixed base?	Yes		
	Was podium model treated as fixed base?	Yes		
	If podium has basement, what was treated as the base?	The bottom of the basement		
	Were soil springs used? What location and type?	No		
	Were any aspects of modeling or boundary conditions particularly of concern or particularly influential on the loads or displacements?	No		

General	IT6 Building Identifier	4	5a	5b
	Building Description:	Wood shear walls over concrete shear walls		
	Occupancy(s)	E	Occupancy Category IV	Occupancy Category IV
	Podium structural system/seismic system/ R-factor	5	Concrete SOMD and Concrete Shear - Retaining Wall / Special Reinforced Concrete Shear Wall / R=6	Concrete SOMD (normal weight) and Concrete Shear - Retaining Wall / Special Reinforced Concrete Shear Wall / R=6
	Podium number of stories	1	1	2
	Podium number of stories below ground (if applicable)	none	1	2 partial levels (unbalanced soils for two levels)
	Podium approximate plan dimensions	100ft x 100ft	160'x270'	250'x210'
	Tower structural system/ seismic system/ R-factor	6.5	Concrete SOMD and steel gravity columns and beams / Special Steel Moment Frame / R=8	Concrete SOMD (light weight) and steel gravity columns and beams / Special Steel Moment Frame / R=8
	Number of towers on one podium	1	1	1
	Tower(s) number of stories	1 or 2	7 + Roof	5 + penthouses
	Tower(s) approximate plan dimensions	100ft x 100ft	410'x100'	330'x150'
	Structural irregularities or other unique configurations	Re-entrant corners	Torsional Irregularity (Type 1A), Reentrant corner irregularity (Type 2), Out of plane offset irregularity (Type 4)	Extreme torsional irregularity (level 2 and 3) meeting exception in 2019 CBC (Type 1B), Reentrant corner irregularity (Type 2), Out of plane offset (type 4), non parallel system (type 5), designed for rho = 1.3 in both directions and considered orthogonal effects.
	Governing building code/ ASCE 7 edition	ASCE 7-10	2016 CBC including OSHPD sections	2016 CBC including OSHPD sections
	Did podium and tower have same EOR or different?	Yes	same EOR	same EOR
Analysis Basics	Was two-stage analysis considered for the analysis?	Yes	Yes	No
	If used, why?	Ability to use the higher R value on the wood design.	It is the method prescribed by code that was considered	n/a
	If decided against, why?	-	-	due to extreme torsional irregularity at podium levels, need to maintain lower drifts at those levels (less than 10% of code limit). Therefore, lowest overall base shear was evaluated (using a single base shear for all levels, rather than two stage). This was used in order to meet the exception for extreme torsional irregularity allowed in 2016 CBC. Later used exception from 2019 CBC because of an appendage/wing's diaphragm aspect ration (which did not govern the overall behavior of the building) was more than 3.
	What ASCE 7 criterion/criteria for the two stage method were found difficult to satisfy? If so, what were the difficulties?	The period requirement. Additional guidance was provided by woodworks for alternate ways to calculate building periods for podium structures	Criteria per ASCE 7-16 (stiffness & period) was not difficult to satisfy for this structure	n/a
	Was the podium torsionally irregular and if so, how was the torsional irregularity of the podium addressed in the podium and in the tower?	Did not apply	No, the podium fell within the 10% of the allowable drift limits since it was a concrete shear wall) which relieved the torsional irregularity check at the podium level.	yes, addressed throughout because it was single stage rather than two stage. designed for rho = 1.3 in both directions and for orthogonal effects.

	IT6 Building Identifier	4	5a	5b
	Building Description:	Wood shear walls over concrete shear walls		
	Did the placement of the tower on the podium create torsion? If so, how was that response analyzed in the tower and the podium?	Did not apply	Not really, the placement of the tower was well distributed on the podium and the surrounding pad footings	no, tower was very regular and balanced (very little coupling when looking at the mode shapes at upper structure)
	The following two questions about analysis pertain if the two stage method was used.			
	What analysis method was used for the tower? ELF or Response Spectrum	ELF	Response Spectrum	Response Spectrum
	If MRS was used, was the primary mode of vibration torsion or translation? If torsion, how did you demonstrate the	-	Translation.	Translation.
	Have you ever used a method that rendered results that didn't make sense, such as the period of the combined structure being less than the upper structure alone? If so, which method was it?	-	No	No
PERIOD OF VIBRATION (12.2.3.2b ASCE7-22)	How was the period of vibration of the PODIUM calculated?	EQ 12.8-7 ASCE 7-10	F.E. software (ETABS) determined the period of the Podium	not evaluated
	How was the period of vibration of the TOWER calculated?	EQ 12.8-7 ASCE 7-10	F.E. software (ETABS) determined the period of the tower	not evaluated
	How was the period of vibration of the PODIUM + TOWER calculated?	FEMA 450 EQ C5.2-1	F.E. software (ETABS) determined the period of the tower	F.E. software (ETABS) determined the period of the tower+podium
	PODIUM Period of Vibration from the model (Seconds)	Not Computed	2 Stage design criteria per ASCE was met at time of design, but the detailed information is no longer available.	n/a
	TOWER Period of Vibration from the model (Seconds)	Not Computed	2.413 sec	n/a
	PODIUM approximate period (Seconds)	0.129sec	2 Stage design criteria per ASCE was met at time of design, but the detailed information is no longer available.	n/a
	TOWER approximate period (Seconds)	0.342sec	1.426 sec	n/a
STIFFNESS (12.2.3.2a ASCE7-22)	How was the stiffness of the PODIUM calculated?	Computer Analysis	Static load case in F.E. software	n/a
	How as the stiffness of the TOWER calculated?		Static load case in F.E. software	n/a
	How as the stiffness of the TOWER calculated?	-	See above	n/a
Special Considerations	What analysis/detailing methods were used to qualify a building that might not otherwise qualify?	-	-	n/a
	Was the method used to analyze a structure where the interface is stepped? When the interface is at more than one level?	-	Drag members were detailed to take of adjacent lateral foundation forces into the top of the building walls / level 1 diaphragm	we had stepped foundations (partial level at lowest level, expanded in two levels above). This can be an analytical "hard spot" if there is a wall continuous in-plane across this step (this issue was not observed in walls that had a perpendicular wall transition at the step). lateral spring restraints were used in lieu of horizontal pins (vertical pins were used for vertical restraint however).
SOFTWARE	What Software was mainly used to design the PODIUM , the TOWER and the combined building ? Why was that software selected?	Podium - Etabs. Tower - by hand.	ETABS since it is the industry standard for a building of this magnitude / complexity	ETABS since it is the industry standard for a building of this magnitude / complexity

	IT6 Building Identifier	4	5a	5b
	Building Description:	Wood shear walls over concrete shear walls		
Did you COMPARE between two-stage analysis and nonlinear time-history analysis?		NA	na	n/a
	What are the differences?			
	Are there any cases where two-stage analysis should not be used or could lead to unconservative designs?	NA	na	na
	Would you be able to provide a summary of your finding or present it to our task group?	NA	na	na
Multiple Towers	Does your project have multiple towers on a single base?	NA	no	no
	If so, what is the number of towers and the number of stories in each tower?	NA	na	na
	What two-stage analysis difficulties were encountered due to the multiple towers and how were they resolved?	NA	na	na
	Did the analysis consider possible in-phase and out-of-phase behavior of the towers?	NA	na	na
	Was there any special consideration of podium diaphragm forces that might be generated between the towers?	NA	na	na
Analysis Modeling and Boundary Conditions	Was tower model treated as fixed base?	No	Yes	yes and no:
	Was podium model treated as fixed base?	Yes	Yes	no - restraints were pinned base we had stepped foundations (partial level at lowest level, expanded in two levels above). This can be an analytical "hard spot" if there is a wall continuous in-plane across this step (this issue was not observed in walls that had a perpendicular wall transition at the step). lateral spring restraints were used in lieu of horizontal pins (vertical pins were used for vertical restraint however).
	If podium has basement, what was treated as the base?	NA	base was the bottom of the basement since it was daylight on certain sides	base was the bottom of the basement since it was daylight on certain sides
	Were soil springs used? What location and type?	NA	no	pins were used for vertical restraint in all locations. lateral springs were used at shear walls and at adjacent columns that were on the same footing. It should be noted that this was done to satisfy plan check comment and the results (shear distribution in walls) were not significantly different from when columns were analyzed with a lateral pin restraint if it was separate from the wall in the ETABS model.
	Were any aspects of modeling or boundary conditions particularly of concern or particularly influential on the loads or displacements?	NA	no	yes, modeling rigid restraints at two levels creates an unrealistic point of rigidity in the elevated diaphragm. In reality, the footing transition is smoothed with stepping. Our preferred method is to analyze all walls with a common base (an average height of wall or conservatively consider base at the lowest level--conservative for overturning in the walls with a higher base). Simplification of the analysis is preferred. Plan check comments required that we model wall heights matching the drawings--with a pinned base this would not have yielded a more appropriate analysis. IT creates a "diving board" effect.

General	IT6 Building Identifier	5c	5d	6
	Building Description:			
	Occupancy(s)	I-2 typical, varies by floor	I-2 typical, other occupancies (S-1, B, F-1, A-2, A-3, etc.) vary by floor. Risk Category IV	Educational
	Podium structural system/seismic system/ R-factor	Concrete Two-Way Slab (normal weight) and Concrete Shear - Retaining Wall / Special Reinforced Concrete Shear Wall / R = 6	Concrete SOMD (normal weight 6" to 9" topping over 3" metal deck) / Concrete Shear Walls / R = 6	5
	Podium number of stories	1	1	2
	Podium number of stories below ground (if applicable)	1	1	1
	Podium approximate plan dimensions	200'x165'	Approx 250' x 300' with skewed grids along NE edges	Can't remember
	Tower structural system/ seismic system/ R-factor	Concrete SOMD (light weight) and steel gravity columns and beams / Special Steel Moment Frame / R=8	Concrete SOMD (typically LWT 3.25" conc over 3" metal deck) / Special Steel Moment Frame / R = 8 SCBF / R = 6 at mechanical penthouse (qualifies as floor level)	BRBF R=8
	Number of towers on one podium	1	1	1
	Tower(s) number of stories	6 + mechanical penthouse	(8) stories plus (1) story mechanical penthouse	3
	Tower(s) approximate plan dimensions	300'x90'	Approx 250' x 300' with skewed grids along NE edges for levels 1, 2, and 3. Approx 225' x 300' for the upper levels.	Can't remember
	Structural irregularities or other unique configurations	Torsion Irregularity (Type 1A) Reentrant Corner Irregularity (Type 2) Nonparallel Systems Irregularity (Type 5) Weight (Mass) Irregularity (Type 2) - At Roof	Horizontal 1A Torsional Irregularity 2 Reentrant Corner Irregularity 4 Out-Of-Plane Offset 5 Nonparallel Systems Irregularity Vertical 2 Weight/Mass Irregularity (between low roof mechanical level and typical patient level above)	Plan Irregularity
	Governing building code/ ASCE 7 edition	2019 CBC / ASCE 7-16	2016 CBC / ASCE 7-10	Asce7-16
	Did podium and tower have same EOR or different?	Same EOR	Same EOR	yes
Analysis Basics	Was two-stage analysis considered for the analysis?	Yes	Yes	yes
	If used, why?	To separate the analysis of the tower from the stiffer podium.	To separate the analysis of the tower from the stiffer podium.	To scale the seismic shear for transition from BRBF to Bearing wall system below
	If decided against, why?	n/a	n/a	
	What ASCE 7 criterion/criteria for the two stage method were found difficult to satisfy? If so, what were the difficulties?	Criteria per ASCE 7-16 (stiffness & period) was not difficult to satisfy for this structure	Criteria per ASCE 7-16 (stiffness & period) was not difficult to satisfy for this structure	The limit on the period shift of 1.1 because of modeling the podium slab flexibility was just on the border
	Was the podium torsionally irregular and if so, how was the torsional irregularity of the podium addressed in the podium and in the tower?	Yes, for seismic loads in one direction, amplification per ASCE 7-16 was applied for analysis. The torisional irregularity for the tower only occurred in one direction and was minor. IE did not result in amplification, Ax=1.0.	Yes, amplification per ASCE 7-16 was applied for the analysis. The podium was modeled as massless in the ETABS model so that the stiffness and torsional response of the podium was explicitly considered when evaluating the torsional response of the tower above.	The podium had adequate walls on all sides. Both the podium and the superstructure were modeled together in the same 3-D model so overall behavior of the structure was properly captured via one overall model

	IT6 Building Identifier	5c	5d	6
	Building Description:			
	Did the placement of the tower on the podium create torsion? If so, how was that response analyzed in the tower and the podium?	No see response above.	Placement of the tower was centralized on the podium, but some torison still did exist within the allowable limits (Not a Type 1B irregularity). Refer to comment above.	please see answer above for #18
	The following two questions about analysis pertain if the two stage method was used.			
	What analysis method was used for the tower? ELF or Response Spectrum	Response Spectrum	Response Spectrum	Response spectrum
	If MRS was used, was the primary mode of vibration torsion or translation? If torsion, how did you demonstrate the	Translation	Translation was the first two modes of vibration (x and y)	Translational mode
	Have you ever used a method that rendered results that didn't make sense, such as the period of the combined structure being less than the upper structure alone? If so, which method was it?	No	No	No
PERIOD OF VIBRATION (12.2.3.2b ASCE7-22)	<u>How was the period of vibration of the PODIUM calculated?</u>	F.E. software (ETABS) determined the period of the podium.	F.E. software (ETABS) determined the period of the podium (tower structure was removed)	As an independent structure usually resulting into a period with the peak acceleration from the response Spectra
	<u>How was the period of vibration of the TOWER calculated?</u>	F.E. software (ETABS) determined the period of the tower.	F.E. software (ETABS) determined the period of the tower. The podium was modeled in the same ETABS model when calculating the period of the Tower. Podium mass was set to zero in order to isolate the Tower response.	As part of the overall model with the top of the podium retained from lateral movement
	<u>How was the period of vibration of the PODIUM + TOWER calculated?</u>	F.E. software (ETABS) determined the period of the tower+podium	F.E. software (ETABS) determined the combined period of the tower+podium	As part of one overall model
	PODIUM Period of Vibration from the model (Seconds)	0.12	0.133 (Y), 0.106 (X)	Can't remember
	TOWER Period of Vibration from the model (Seconds)	1.26	1.948 (X), 1.742 (Y)	Can't remember
	PODIUM approximate period (Seconds)	0.21	0.16	Can't remember
	TOWER approximate period (Seconds)	0.98	1.541	Can't remember
STIFFNESS (12.2.3.2a ASCE7-22)	<u>How was the stiffness of the PODIUM calculated?</u>	Applied an arbitrary 100 kip lateral load at roof level and extracted the resulting deflections.	Applied an arbitrary 100 kip lateral load at roof level and extracted the resulting deflections.	The entire building was modeled(3-D) with appropriate stiffness modifiers (walls etc). The slab were modeled with finite element mesh with appropriate modifiers for concrete stiffness.
	<u>How as the stiffness of the TOWER calculated?</u>	Applied an arbitrary 100 kip lateral load at podium level and extracted the resulting deflections.	Applied an arbitrary 100 kip lateral load at podium level and extracted the resulting deflections.	Please see response to line 32
	<u>How as the stiffness of the TOWER calculated?</u>	Duplicate question.	Duplicate question.	Please see response to line 32
Special Considerations	What analysis/detailing methods were used to qualify a building that might not otherwise qualify?	n/a	n/a	None
	Was the method used to analyze a structure where the interface is stepped? When the interface is at more than one level?	Podium slab had some changes in top of concrete elevation & some footings were stepped down. A weighted average of the wall heights was used for modeling of the basement level.	Podium diaphragm steps/depressions were relatively minor (approximately 1.5" to 3")	No interface was essentially at the same level
SOFTWARE	What Software was mainly used to design the PODIUM , the TOWER and the combined building ? Why was that software selected?	ETABS	ETABS. This software is the standard tool in our office for the linear analysis and design of new structures.	RAM and ETABS typical software

	IT6 Building Identifier	5c	5d	6
	Building Description:			
Did you COMPARE between two-stage analysis and nonlinear time-history analysis?		n/a	n/a	No
	What are the differences?			
	Are there any cases where two-stage analysis should not be used or could lead to unconservative designs?	n/a	n/a	Yes, if the overall behavior of the building is not captured properly with 2-stage analysis. Example: the overturning forces from the superstructure are not properly captured
	Would you be able to provide a summary of your finding or present it to our task group?	n/a	n/a	Sorry No!
Multiple Towers	Does your project have multiple towers on a single base?	No	No	No
	If so, what is the number of towers and the number of stories in each tower?	n/a	n/a	
	What two-stage analysis difficulties were encountered due to the multiple towers and how were they resolved?	n/a	n/a	
	Did the analysis consider possible in-phase and out-of-phase behavior of the towers?	n/a	n/a	This is the main challenge with 2 stage analysis with multiple superstructures. I
	Was there any special consideration of podium diaphragm forces that might be generated between the towers?	n/a	n/a	
Analysis Modeling and Boundary Conditions	Was tower model treated as fixed base?	No, tower & podium were modeled together as a single model. For the two-stage analysis mass assigned according to the analysis being	No, tower & podium were modeled together as a single model. For the two-stage analysis mass assigned according to the analysis being	No overall building was modeled in 3-D with superstructure above and columns
	Was podium model treated as fixed base?	No, restraints were pinned base.	No, restraints were pinned base.	No the podium including the supporting walls were modeled as overall 3-D model for the building
	If podium has basement, what was treated as the base?	Tower Analysis: Podium considered base Podium Analysis: Bottom of basement	Tower Analysis: Podium considered base Podium Analysis: Bottom of basement	No overall building through the basement was modeled
	Were soil springs used? What location and type?	No	No	No
	Were any aspects of modeling or boundary conditions particularly of concern or particularly influential on the loads or displacements?	No	No	No

General	IT6 Building Identifier	7	8	9a
	Building Description:			
	Occupancy(s)	Residential, commercial at ground floor	B, M, R-2, S-2	Podium: S-2, M, R-2, B Superstructure: R-2, B, S-2, A-3
	Podium structural system/seismic system/ R-factor	Special reinforced concrete shear walls, R=6	6	Special Reinf. Conc. Shear Wall, 5
	Podium number of stories	3	1 Story+Basement	2
	Podium number of stories below ground (if applicable)	0	1	1
	Podium approximate plan dimensions	90ft x 125ft	300' x 425'	13000 sq ft
	Tower structural system/ seismic system/ R-factor	Light frame wood walls sheathed with wood structural panels, R=6.5	6.5 Wood Shear Wall	Wood Panel Bearing Shear Wall, 6.5
	Number of towers on one podium	1	4	1
	Tower(s) number of stories	5	5	5
	Tower(s) approximate plan dimensions	90ft x 125ft	150' x 100'	13000 sq ft
	Structural irregularities or other unique configurations	Re-entrant corners at diaphragm notch at Level 3 and above	Wood superstructure has shear wall party wall at 45 deg planview at reentrant courtyard corner: Non Parallel System	NA
	Governing building code/ ASCE 7 edition	ASCE 7-16	ASCE7-16	ASCE 7-10
	Did podium and tower have same EOR or different?	Same	Same	Same
Analysis Basics	Was two-stage analysis considered for the analysis?	Yes	Yes	Yes
	If used, why?	To avoid the complications of modeling both the concrete and wood structures in a single analytical model. Also, we expect we would have larger forces in the wood structure if they were modeled together.	This is the methodology used when wood buildings are designed on top of a podium	This is the methodology used when wood buildings are designed on top of a podium
	If decided against, why?	N/A	Wood frame superstructure wont work with concrete mass put through vertical redistribution	NA
	What ASCE 7 criterion/criteria for the two stage method were found difficult to satisfy? If so, what were the difficulties?	The stiffness check was fairly close in one direction (ratio of ~15). Period checks were easy to satisfy. There are a few difficulties related to engineering judgement that makes us less confident in the two stage analysis method: 1) Code gives direction on calculating stiffness for a single story wood structure, but is silent on multistory. In order to treat the stiffness of the wood structure correctly, you need to do onerous calculations. We are concerned that many engineers would not take all contributions to flexibility into account for the wood structure. (This relates more to multistory wood structures in general than two stage analysis specifically.)	Biggest challenge is making sure ELF can be used on substructure.	NONE
	Was the podium torsionally irregular and if so, how was the torsional irregularity of the podium addressed in the podium and in the tower?	Not torsionally irregular.	Adjusting shearwalls and more "complex" stiffness calculation.	Drifts are very, very small. No torsional irregularity

	IT6 Building Identifier	7	8	9a
	Building Description:			
	Did the placement of the tower on the podium create torsion? If so, how was that response analyzed in the tower and the podium?	No, tower is full floor plate of podium.	No	No
	The following two questions about analysis pertain if the two stage method was used.			
	What analysis method was used for the tower? ELF or Response Spectrum	ELF	ELF	ELF
	If MRS was used, was the primary mode of vibration torsion or translation? If torsion, how did you demonstrate the	N/A	NA	NA
	Have you ever used a method that rendered results that didn't make sense, such as the period of the combined structure being less than the upper structure alone? If so, which method was it?	No.	Yes, the effects of non-structural elements changed results in modeling.	NO
PERIOD OF VIBRATION (12.2.3.2b ASCE7-22)	How was the period of vibration of the PODIUM calculated?	Directly from ETABS model with cracked section properties and mass of wood structure included at the podium level.	Per code	ASCE EQ 12.8-7 and Alternatively FEMA 450 EQ C5.2-1
	How was the period of vibration of the TOWER calculated?	We use proprietary software to calculate lateral deflections at each floor. We took the deflection at 2/3 the height of the wood structure and divided by the base shear to get a stiffness. Then we calculated the period from the stiffness based on a single DOF structure with the full mass of the wood structure (2*pi*sqrt(M/k)).	Per code	ASCE EQ 12.8-7 and Alternatively FEMA 450 EQ C5.2-1
	How was the period of vibration of the PODIUM + TOWER calculated?	We formed stiffness and mass matrices for the structure as a 2 DOF system with the stiffnesses noted above and the masses of the concrete structure and wood structure, respectively. We performed an eigen analysis to find the mode shapes and period of the combined structure.	Code -Raleigh's Formula	ASCE EQ 12.8-7 and Alternatively FEMA 450 EQ C5.2-1
	PODIUM Period of Vibration from the model (Seconds)	0.39s (east-west) and 0.32s (north-south)	Modeled on a 3D FEA software but period not mentioned	0.096
	TOWER Period of Vibration from the model (Seconds)	0.84s (east-west) and 0.91s (north-south)	Not Modeled	0.37
	PODIUM approximate period (Seconds)	Ta=0.14s (approximate fundamental period from code)	Per code max	
	TOWER approximate period (Seconds)	Ta=0.38s (approximate fundamental period from code)	Per code max	0.37
STIFFNESS (12.2.3.2a ASCE7-22)	How was the stiffness of the PODIUM calculated?	Base shear of full structure divided by displacement at the podium (with mass of wood structure lumped at podium level).	RAM Software	Base shear of Podium divided by deflection
	How as the stiffness of the TOWER calculated?	Deflection at 2/3 the height of the wood structure and divided by the base shear of the wood structure.	Using ELF forces and computing the displacements, individual shear wall drifts are computed. Overall super-structure not calculated.	Base shear of Tower divided by deflection
	How as the stiffness of the TOWER calculated?	(repeat question)		
Special Considerations	What analysis/detailing methods were used to qualify a building that might not otherwise qualify?	None	Sometimes common sense needs to be applied because the modeling has limitations	NA
	Was the method used to analyze a structure where the interface is stepped? When the interface is at more than one level?	N/A	Some steps can be ignored, more significant steps are often ignored but shouldn't be.	NA
SOFTWARE	What Software was mainly used to design the PODIUM , the TOWER and the combined building ? Why was that software selected?	Podium: ETABS Tower: Proprietary software (LightShearWalls, Tipping Applications) - we make a 3d model with elements that account for stiffness inputs specific to light shear walls, such as nonlinear nail slip, hold down stiffness, and the effects of gravity load and stacked wall flexibility. Combined: Hand calculations	Podium Software = FEM (Ram). Superstructure software is proprietary. Combined structure is not directly analyzed in one software, the combined effects are generally analyzed in a spreadsheet or hand calc.	Podium: RAM Frame Tower: RISA Floor

	IT6 Building Identifier	7	8	9a
	Building Description:			
Did you COMPARE between two-stage analysis and nonlinear time-history analysis?		We didn't for this project. We did do a comparison study with NLRHA for a previous project.	NA	NA
	What are the differences?	In that case, we overlaid spectra at the ground with spectra at the top of the podium and found large amplifications in spectral acceleration at some period ranges. A podium on flexible foundations will		
	Are there any cases where two-stage analysis should not be used or could lead to unconservative designs?	I think the basic premise for doing two stage analysis is right. The problem is that it relies on period calculations that aren't very rigorous. We have concerns unless the	NA	NA
	Would you be able to provide a summary of your finding or present it to our task group?	Sure.	NA	NA
Multiple Towers	Does your project have multiple towers on a single base?	N/A	Yes	NA
	If so, what is the number of towers and the number of stories in each tower?	N/A	4 towers with 5 stories	NA
	What two-stage analysis difficulties were encountered due to the multiple towers and how were they resolved?	N/A	Generally, the wood frame superstructures have many shear walls distributed fairly evenly over the	NA
	Did the analysis consider possible in-phase and out-of-phase behavior of the towers?	N/A	No	NA
	Was there any special consideration of podium diaphragm forces that might be generated between the towers?	N/A	No	NA
Analysis Modeling and Boundary Conditions	Was tower model treated as fixed base?	Yes	Yes	Tower base at podium considered fixed
	Was podium model treated as fixed base?	Yes	Yes	Podium at ground level considered fixed
	If podium has basement, what was treated as the base?	N/A	Lowest floor fully subterranean	Ground Level
	Were soil springs used? What location and type?	Not used	No	No
	Were any aspects of modeling or boundary conditions particularly of concern or particularly influential on the loads or displacements?	See notes above in NLRHA comparison section.	No	No

General	IT6 Building Identifier	9b	9c	10a
	Building Description:			Woodframe on Concrete Podium
	Occupancy(s)	Podium: S-2, M, R-2, A-2 Superstructure: R-2	Podium: S-2, R-2 Superstructure: R-2	Residential Above Grade, Parking below grade
	Podium structural system/seismic system/ R-factor	Special Reinf. Conc. Shear Wall, 5	Special Reinf. Conc. Shear Wall, 5	Special structural walls, R=5
	Podium number of stories	3	3	2 above seismic base
	Podium number of stories below ground (if applicable)	2	NA	2
	Podium approximate plan dimensions	19500 sq ft	26000 sq ft	200ft x 175ft
	Tower structural system/ seismic system/ R-factor	Wood Panel Bearing Shear Wall, 6.5	Wood Panel Bearing Shear Wall, 6.5	Wood frame Shear Wall R=6.5
	Number of towers on one podium	1	1	1
	Tower(s) number of stories	5	5	5
	Tower(s) approximate plan dimensions	16500 sq ft	18000 sq ft	200ft x 175ft
	Structural irregularities or other unique configurations	NA	NA	Torsional Irregularity, Horiz Offset, Mass irregularity, Reentrant Corners
	Governing building code/ ASCE 7 edition	ASCE 7-16	ASCE 7-16	ASCE 7-10
	Did podium and tower have same EOR or different?	Same	Same	Same
Analysis Basics	Was two-stage analysis considered for the analysis?	Yes	Yes	Yes
	If used, why?	This is the methodology used when wood buildings are designed on top of a podium	This is the methodology used when wood buildings are designed on top of a podium	Standard practice for WF, simplify design process, does not penalize lighter tower structure as if designed as 1 building
	If decided against, why?	NA	NA	
	What ASCE 7 criterion/criteria for the two stage method were found difficult to satisfy? If so, what were the difficulties?	NONE	NONE	Does not satisfy criteria with approx period calculations, need to do more comprehensive calc.
	Was the podium torsionally irregular and if so, how was the torsional irregularity of the podium addressed in the podium and in the tower?	Drifts are very very small. No torsional irregularity	Drifts are very very small. No torsional irregularity	Yes, Torsional amplification applied to podium structure

	IT6 Building Identifier	9b	9c	10a
	Building Description:			Woodframe on Concrete Podium
	Did the placement of the tower on the podium create torsion? If so, how was that response analyzed in the tower and the podium?	No	No	Tower and Podium were aligned / same size
	The following two questions about analysis pertain if the two stage method was used.			
	What analysis method was used for the tower? ELF or Response Spectrum	ELF	ELF	ELF
	If MRS was used, was the primary mode of vibration torsion or translation? If torsion, how did you demonstrate the	NA	NA	NA
	Have you ever used a method that rendered results that didn't make sense, such as the period of the combined structure being less than the upper structure alone? If so, which method was it?	NO	NO	No
PERIOD OF VIBRATION (12.2.3.2b ASCE7-22)	<u>How was the period of vibration of the PODIUM calculated?</u>	ASCE EQ 12.8-7 and Alternatively FEMA 450 EQ C5.2-1	ASCE EQ 12.8-7 and Alternatively FEMA 450 EQ C5.2-1	ETABS model
	<u>How was the period of vibration of the TOWER calculated?</u>	ASCE EQ 12.8-7 and Alternatively FEMA 450 EQ C5.2-1	ASCE EQ 12.8-7 and Alternatively FEMA 450 EQ C5.2-1	Rayleigh method hand calculation
	<u>How was the period of vibration of the PODIUM + TOWER calculated?</u>	ASCE EQ 12.8-7 and Alternatively FEMA 450 EQ C5.2-1	ASCE EQ 12.8-7 and Alternatively FEMA 450 EQ C5.2-1	Rayleigh method hand calculation
	PODIUM Period of Vibration from the model (Seconds)	0.074	0.145	
	TOWER Period of Vibration from the model (Seconds)	0.38	0.39	
	PODIUM approximate period (Seconds)			
	TOWER approximate period (Seconds)	0.38	0.39	
STIFFNESS (12.2.3.2a ASCE7-22)	<u>How was the stiffness of the PODIUM calculated?</u>	Base shear of Podium divided by deflection	Base shear of Podium divided by deflection	Etabs
	<u>How as the stiffness of the TOWER calculated?</u>	Base shear of Tower divided by deflection	Base shear of Tower divided by deflection	Hand calc
	<u>How as the stiffness of the TOWER calculated?</u>			
Special Considerations	What analysis/detailing methods were used to qualify a building that might not otherwise qualify?	NA	NA	
	Was the method used to analyze a structure where the interface is stepped? When the interface is at more than one level?	NA	NA	
SOFTWARE	What Software was mainly used to design the PODIUM , the TOWER and the combined building ? Why was that software selected?	Podium: RAM Frame Tower: RISA Floor	Podium: RAM Frame Tower: RISA Floor	Spreadsheets/hand calcs for wood, ETABS+Spreadsheets+S-Concrete for concrete

	IT6 Building Identifier	9b	9c	10a
	Building Description:			Woodframe on Concrete Podium
Did you COMPARE between two-stage analysis and nonlinear time-history analysis?		NA	NA	Not typically done
	What are the differences?			
	Are there any cases where two-stage analysis should not be used or could lead to unconservative designs?	NA	NA	
	Would you be able to provide a summary of your finding or present it to our task group?	NA	NA	
Multiple Towers	Does your project have multiple towers on a single base?	NA	NA	No
	If so, what is the number of towers and the number of stories in each tower?	NA	NA	
	What two-stage analysis difficulties were encountered due to the multiple towers and how were they resolved?	NA	NA	
	Did the analysis consider possible in-phase and out-of-phase behavior of the towers?	NA	NA	
	Was there any special consideration of podium diaphragm forces that might be generated between the towers?	NA	NA	
Analysis Modeling and Boundary Conditions	Was tower model treated as fixed base?	Tower base at podium considered fixed	Tower base at podium considered fixed	Yes
	Was podium model treated as fixed base?	Podium at ground level considered fixed	Podium at foundation level considered fixed	Yes
	If podium has basement, what was treated as the base?	Ground Level	NA	Typically modelled to bottom of basement, but seismic base for base shear calc is ground level.
	Were soil springs used? What location and type?	No	No	Not typically done
	Were any aspects of modeling or boundary conditions particularly of concern or particularly influential on the loads or displacements?	Modeling of unbalanced soil pressures on the basement walls	Modeling of unbalanced soil pressures on the basement walls	

General	IT6 Building Identifier	10b	10c	11
	Building Description:	Single Story Steel Moment Frame on Shared Concrete Podium w/ Highrise	Multiple Towers on Shared Podium (generic example)	High-rise Hotel
	Occupancy(s)	Parking and Res (podium), Residential Amenity (MF)		Hotel (R1)
	Podium structural system/seismic system/ R-factor	Special structural walls, R=6	Special structural walls, R=6	SRCSW + 1 Conc Frame with BRB braces
	Podium number of stories	4 above seismic base	1-6 typically	4
	Podium number of stories below ground (if applicable)	4	2-6 typically	0
	Podium approximate plan dimensions	140x180	150x200	100' x 124'
	Tower structural system/ seismic system/ R-factor	Special Moment Frame R=8	Special structural walls, R=6 + Woodframe	Light-frame (CFS) walls sheathed with WSP
	Number of towers on one podium	2	typically 2, occasionally 3, rarely 4	1
	Tower(s) number of stories	1 story for 2-stage building, 33 for main tower.	4-50 (eg, 5 story woodframe and 35 story highrise on podium, or 2 highrises on podium)	6
	Tower(s) approximate plan dimensions	65ftx45ft	100x100ft to 100x150ft Concrete towers, 70x180ft WF	100' x 124'
	Structural irregularities or other unique configurations	Torsional Irregularity, Horiz Offset, Mass irregularity	Torsional Irregularity, Horiz Offset, Mass irregularity	non parallel systems, torsional irregular (extreme)
	Governing building code/ ASCE 7 edition	ASCE 7-16	ASCE 7-16, NBCC2015	2015 Seattle Building Code / ASCE 7-10
	Did podium and tower have same EOR or different?	Same		same EOR
Analysis Basics	Was two-stage analysis considered for the analysis?	Yes	Yes	Yes
	If used, why?	Main building was PBD w/ NLTHA. Design Team wanted to remove small amenity building from scope of PBD.	To provide rationale for base shear scaling of overall structure and each tower	Wanted to reset structural height at top of podium since max structural height for CFS shear walls is limited to 65' and this was exceeded if measured from the seismic base
	If decided against, why?		Upper/lower both easily modeled in ETABS. Desire to keep all analysis in ETABS. Large discrepancy between RSA shear and Two stage base shear (Scale factor > 2)	n/a
	What ASCE 7 criterion/criteria for the two stage method were found difficult to satisfy? If so, what were the difficulties?	None	None	Stiffness and period ratio criteria were difficult to meet. SEOR tried to include non participating elements to lower the period of the upper structure (was not consistent with the lower portion that did not include non participating elements) SEOR had to complete a Code Mod for AHJ to accept a period ratio higher than 1.1 in one direction (1.15) Note that in addition, the stiffness ratio in one direction was 17 (>10, so okay but close)
	Was the podium torsionally irregular and if so, how was the torsional irregularity of the podium addressed in the podium and in the tower?	No	About 50% of the time, torsional amplification applied if so	Yes, was not specifically addressed as a dynamic amplification on the tower

	IT6 Building Identifier	10b	10c	11
	Building Description:	Single Story Steel Moment Frame on Shared Concrete Podium w/ Highrise	Multiple Towers on Shared Podium (generic example)	High-rise Hotel
	Did the placement of the tower on the podium create torsion? If so, how was that response analyzed in the tower and the podium?	Not significant due to mass disparity (podium >>> tower). NLTHA was used for podium including the amenity mass	Towers are usually not centered on podium, so likely. Effects are accounted for in ETABS model	No
	The following two questions about analysis pertain if the two stage method was used.			Ok
	What analysis method was used for the tower? ELF or Response Spectrum	ELF	RSA	ELF for tower MRSA for podium
	If MRS was used, was the primary mode of vibration torsion or translation? If torsion, how did you demonstrate the	Translational. In the hypothetical situation where it was torsional, we might try locking the analysis to the	Translation typically	don't know, was not provided in output
	Have you ever used a method that rendered results that didn't make sense, such as the period of the combined structure being less than the upper structure alone? If so, which method was it?	No	Occasionally. For instance, assumption of boundary condition may change tower period. In standalone model, could be assumed fixed or pinned, but in combined structure it will be somewhere in between.	
PERIOD OF VIBRATION (12.2.3.2b ASCE7-22)	<u>How was the period of vibration of the PODIUM calculated?</u>	ETABS model	ETABS model	Rayleigh formula taken from FEMA 450 eq. C5.2-1
	<u>How was the period of vibration of the TOWER calculated?</u>	ETABS model	ETABS model	Rayleigh formula taken from FEMA 450 eq. C5.2-1
	<u>How was the period of vibration of the PODIUM + TOWER calculated?</u>	ETABS model	ETABS model	Rayleigh formula taken from FEMA 450 eq. C5.2-1
	PODIUM Period of Vibration from the model (Seconds)			Tx all = 1.044 Ty all = 1.108
	TOWER Period of Vibration from the model (Seconds)			Tx upper = 0.952 Ty upper = 0.967
	PODIUM approximate period (Seconds)			Ta = 0.465
	TOWER approximate period (Seconds)			N/A
STIFFNESS (12.2.3.2a ASCE7-22)	<u>How was the stiffness of the PODIUM calculated?</u>	Etabs	ETABS model	Force model: applied a force to top story of podium. Used cracked properties reduction factor
	<u>How as the stiffness of the TOWER calculated?</u>	Etabs	ETABS model	Force model: applied a force to top story of tower.
	<u>How as the stiffness of the TOWER calculated?</u>		ETABS model	
Special Considerations	What analysis/detailing methods were used to qualify a building that might not otherwise qualify?			
	Was the method used to analyze a structure where the interface is stepped? When the interface is at more than one level?			N/A
SOFTWARE	What Software was mainly used to design the PODIUM , the TOWER and the combined building ? Why was that software selected?	PERFORM3D+ETABS+Spreadsheets +S-Concrete for concrete, ETABS+spreadsheets for tower	ETABS+Spreadsheets+S-Concrete for concrete buildings	podium: Etabs tower: handcalcs (flex diaph)

	IT6 Building Identifier	10b	10c	11
	Building Description:	Single Story Steel Moment Frame on Shared Concrete Podium w/ Highrise	Multiple Towers on Shared Podium (generic example)	High-rise Hotel
Did you COMPARE between two-stage analysis and nonlinear time-history analysis?		No, two stage was used to remove the tower from scope of NLTHA	Not typically done	N/A
	What are the differences?			
	Are there any cases where two-stage analysis should not be used or could lead to unconservative designs?			When podium is too tall/flexible. What are the limits?
	Would you be able to provide a summary of your finding or present it to our task group?			
Multiple Towers	Does your project have multiple towers on a single base?	Yes, but the two-stage tower was insignificant to overall response	Yes	No
	If so, what is the number of towers and the number of stories in each tower?	two towers, 1 story and 35 story	typically 2, occasionally 3, rarely 4	
	What two-stage analysis difficulties were encountered due to the multiple towers and how were they resolved?		Scaling forces, determining design base shear in podium, combining forces for podium from each tower	
	Did the analysis consider possible in-phase and out-of-phase behavior of the towers?		Yes, when full structure is modelled in ETABS	
	Was there any special consideration of podium diaphragm forces that might be generated between the towers?	Forces were small, but were added to main tower demands	Yes, when full structure is modelled in ETABS	
Analysis Modeling and Boundary Conditions	Was tower model treated as fixed base?	Pinned base moment frame	Yes	Yes
	Was podium model treated as fixed base?	Fixed at foundation (2 levels underground)	Yes	Yes, but not modeled
	If podium has basement, what was treated as the base?	Typically modeled to bottom of basement, but seismic base for base shear calc is ground level.	Typically modelled to bottom of basement, but seismic base for base shear calc is ground level.	N/A
	Were soil springs used? What location and type?	Not typically done	Not typically done	no
	Were any aspects of modeling or boundary conditions particularly of concern or particularly influential on the loads or displacements?			

General	IT6 Building Identifier	12
	Building Description:	Five-Story Wood over Three-Story Concrete Podium + Basement
	Occupancy(s)	Podium: S, S-2, B, A-3, R-2 Tower: R-2
	Podium structural system/seismic system/ R-factor	
		6
	Podium number of stories	3 Stories (not including basement)
	Podium number of stories below ground (if applicable)	1
	Podium approximate plan dimensions	48,200 square feet
	Tower structural system/ seismic system/ R-factor	6.5
	Number of towers on one podium	1
	Tower(s) number of stories	5
	Tower(s) approximate plan dimensions	36,600 square feet
	Structural irregularities or other unique configurations	Potential torsional irregularities
	Governing building code/ ASCE 7 edition	IBC 2018/ASCE 7-16
	Did podium and tower have same EOR or different?	Same
Analysis Basics	Was two-stage analysis considered for the analysis?	Yes
	If used, why?	Simplify analysis, wood buildings not typically conducive to analysis in same packages as concrete.
	If decided against, why?	NA
	What ASCE 7 criterion/criteria for the two stage method were found difficult to satisfy? If so, what were the difficulties?	Period given height of podium, accurately calculating out period of wood structure, combatting torsional irregularities in stiff podium that require use of modal response spectrum while two-stage analysis requires bottom levels design with ELF.
	Was the podium torsionally irregular and if so, how was the torsional irregularity of the podium addressed in the podium and in the tower?	Yes. TBD for podium, is an ongoing design challenge on how to account for torsional irregularities when two-stage analysis requires ELF for podium.

	IT6 Building Identifier	12
	Building Description:	Five-Story Wood over Three-Story Concrete Podium + Basement
	Did the placement of the tower on the podium create torsion? If so, how was that response analyzed in the tower and the podium?	No
	The following two questions about analysis pertain if the two stage method was used.	
	What analysis method was used for the tower? ELF or Response Spectrum	ELF
	If MRS was used, was the primary mode of vibration torsion or translation? If torsion, how did you demonstrate the	
	Have you ever used a method that rendered results that didn't make sense, such as the period of the combined structure being less than the upper structure alone? If so, which method was it?	
PERIOD OF VIBRATION (12.2.3.2b ASCE7-22)	How was the period of vibration of the PODIUM calculated?	Calculated using a modal analysis.
	How was the period of vibration of the TOWER calculated?	Currently looking at using Rayleigh's method, equation C5.2-1 in old 2003 version of FEMA 450.
	How was the period of vibration of the PODIUM + TOWER calculated?	Currently looking at using Rayleigh's method, equation C5.2-1 in old 2003 version of FEMA 450.
	PODIUM Period of Vibration from the model (Seconds)	0.5 seconds in X, 0.2 seconds in Y
	TOWER Period of Vibration from the model (Seconds)	Not Modeled
	PODIUM approximate period (Seconds)	0.3 seconds
STIFFNESS (12.2.3.2a ASCE7-22)	How was the stiffness of the PODIUM calculated?	Using the 3D model, the average displacement was obtained. The base shear is divided by the displacement.
	How as the stiffness of the TOWER calculated?	Using ELF forces and computing the displacements. The displacements per story are calculated using SDPWS.
	How as the stiffness of the TOWER calculated?	
Special Considerations	What analysis/detailing methods were used to qualify a building that might not otherwise qualify?	Still in progress. Likely going to be needing to justify higher PT slab stiffnesses than traditional 0.25Ig to lower period for concrete portion and manually calculate wall stiffnesses.
	Was the method used to analyze a structure where the interface is stepped? When the interface is at more than one level?	No stepped interface, have a clean transition for concrete to wood at top of level 4 slab.
SOFTWARE	What Software was mainly used to design the PODIUM , the TOWER and the combined building ? Why was that software selected?	ETABS 2021 for podium. Tower is by hand. ETABS selected due to high seismic environment, good history of accurate/reasonable results, ease of modeling, and familiarity.

	IT6 Building Identifier	12
	Building Description:	Five-Story Wood over Three-Story Concrete Podium + Basement
Did you COMPARE between two-stage analysis and nonlinear time-history analysis?		
	What are the differences?	
		NA
	Are there any cases where two-stage analysis should not be used or could lead to unconservative designs?	NA
	Would you be able to provide a summary of your findings or present it to our task group?	NA
Multiple Towers	Does your project have multiple towers on a single base?	NA
	If so, what is the number of towers and the number of stories in each tower?	NA
	What two-stage analysis difficulties were encountered due to the multiple towers and how were they resolved?	NA
	Did the analysis consider possible in-phase and out-of-phase behavior of the towers?	NA
	Was there any special consideration of podium diaphragm forces that might be generated between the towers?	NA
Analysis Modeling and Boundary Conditions	Was tower model treated as fixed base?	NA
		NOT DESCRIBED
	Was podium model treated as fixed base?	
	If podium has basement, what was treated as the base?	NA
	Were soil springs used? What location and type?	NA
	Were any aspects of modeling or boundary conditions particularly of concern or particularly influential on the loads or displacements?	NA

RESOURCE PAPER 5 A DESIGN PROCESS TO NURTURE CREATIVITY AND INNOVATION IN SEISMIC SYSTEM DESIGN

RP5-1 Abstract

This paper is the work of Issue Team 13: Nurture Engineer Creativity/Innovation. It outlines a conceptual framework for a Behavior-Based Design (BBD) process that enables engineers to be more creative and provides the freedom to innovate when designing seismic lateral systems. The process is in development, and this framework describes the major steps. The process has steps and validation checks intended to ensure that designs meet the safety objectives of the building code. The design process will apply to new and existing buildings and be simpler to use and less costly to implement than alternative means of compliance⁶. The process requires a limited² peer review that is intended to be suitable for the complexities of the method and reduces cost impediments that can arise from a comprehensive⁷ peer review. The goal is to provide designers with increased flexibility to respond to their project needs. The six-step process requires designers to work in the nonlinear realm, with single-degree-of-freedom idealizations, as well as plastic mechanisms and pushover curves for their systems. The explicit reliance on nonlinear behavior allows designers to mix materials in novel configurations. The paper presents the design steps, a flow-chart for use (see Figure 1), and an appendix with three proof-of-concept studies and two other sub-topics. An analytically complete example is a topic of a forthcoming SEAONC-sponsored University Research Program. The design example will be the first step to validate and calibrate the BBD process. The design example will be assessed with an Incremental Dynamic Analysis (IDA), following the procedure in FEMA P-695.

RP5-2 Introduction

Issue Team 13's objective is to address the following question:

⁶ Performance Based Procedure in ASCE 7-22 and Alternative Rational Analysis in ASCE 41-23.

⁷ Limited peer review utilizes a single Structural Engineer reviewer. Comprehensive peer review refers to the common practice by AHJs of utilizing a team of peer reviewers. This is often necessitated by the complexities of nonlinear response-history analysis, including the selection of ground motions.

How can we encourage creativity and maintain safety but not trigger a full alternative means of compliance and a comprehensive peer review when there is the need or desire for a non-standard seismic design?

The proposed Behavior-Based Design (BBD) process offers significant benefits to structural engineers and stakeholders, when the latitude to use unconventional solutions is desired or needed. When considering the use of novel lateral systems, engineers lack design flexibility due to the prescriptive requirements of current code (ASCE 7-22). The alternative means of compliance in ASCE 7-22 (Performance Based Procedure) and ASCE 41-23, (Alternative Rational Analysis) offer design flexibility, but they present impediments to use. They are both costly due to the peer review requirements of the authorities having jurisdiction (AHJs) and the technical complexity. As such, alternative means of compliance is often cost prohibitive for smaller projects. For seismic retrofits, ASCE 41, Tier 3 Linear Dynamic Procedures provide limited flexibility to account for complexities and irregularities commonly found in existing buildings. However, the sole reliance on linear models constrains designs, and it may obscure potentially undesirable mechanisms and poor behavior. Appendix D briefly explores this concern.

The proposed design process addresses these shortcomings and enables creativity and innovation in seismic design by allowing increased design flexibility, while targeting compliance with established performance objectives. The process is a series of steps and validation checks, which employ the established analysis methods of capacity design, Direct Displacement-Based Design⁸ (DDBD), and Modified Modal Superposition⁹ (MMS).

In addition to using a linear model for their system, engineers work directly in the nonlinear realm. To this end, the engineer defines the system's plastic mechanism and pushover curve. The performance of the lateral system is explicitly confirmed with ASCE 41 limits on plastic hinge demands, the direct accounting for irregularities, and the design for higher modes. The gravity system is checked for deflection compatibility under expected deformations. The IT 13 team hopes that the Behavior-Based Design (BBD) process would enable wider adoption of performance-based design by encouraging creativity and innovation while delivering safe structures. The team believes that this process is especially needed for projects with constrained resources.

RP5-3 Summary of the Design Process

A summary of the design process is presented in the flow chart in Figure 1. The six steps are as follows: Step 1 of the process requires the engineer to propose a lateral system, define its plastic

⁸ Priestley, Calvi, and Kowalsky, reference 6 on page 16.

⁹ Martin and Deierlein, reference 9 on page 16.

mechanism, and select the type of hysteretic behavior of the nonlinear elements. Step 2 has the engineer establish the minimum strength and stiffness from the perspective of a new code compliant building assuming a generic $R=8$ system. The engineer next uses Direct Displacement-Based Design (DDBD), or other equivalent linearization methods to determine a trial set of strength-displacement demands at the MCE_r hazard, for a single-degree-of-freedom (SDOF) idealization of the structure. Additionally, the engineer develops a pushover curve utilizing capacity design. Step 3 has the engineer account for additional displacement demands due to irregularities (plan and vertical), ensuring these geometric effects are incorporated into the overall performance assessment. In Step 4, the engineer determines the rotation demands at all plastic hinge locations, and they are validated against the established ASCE 41 acceptance criteria. If any demands are exceeded, the engineer iterates by either reducing demands (Step 2), or by improving hinge capacities with better detailing, or a combination of both. In Step 5, the engineer ensures that the single degree-of-freedom assumption is valid by designing the system to respond elastically to higher mode demands, using the Modified Modal Superposition method (MMS). In Step 6, all gravity elements are checked for ultimate displacement demands to ensure displacement compatibility. If any elements of the gravity system are inadequate for the displacement demands, the engineer iterates again, by either adding displacement capacity where needed to the gravity system through detailing, or reducing demands with additional strength, or a combination of both.

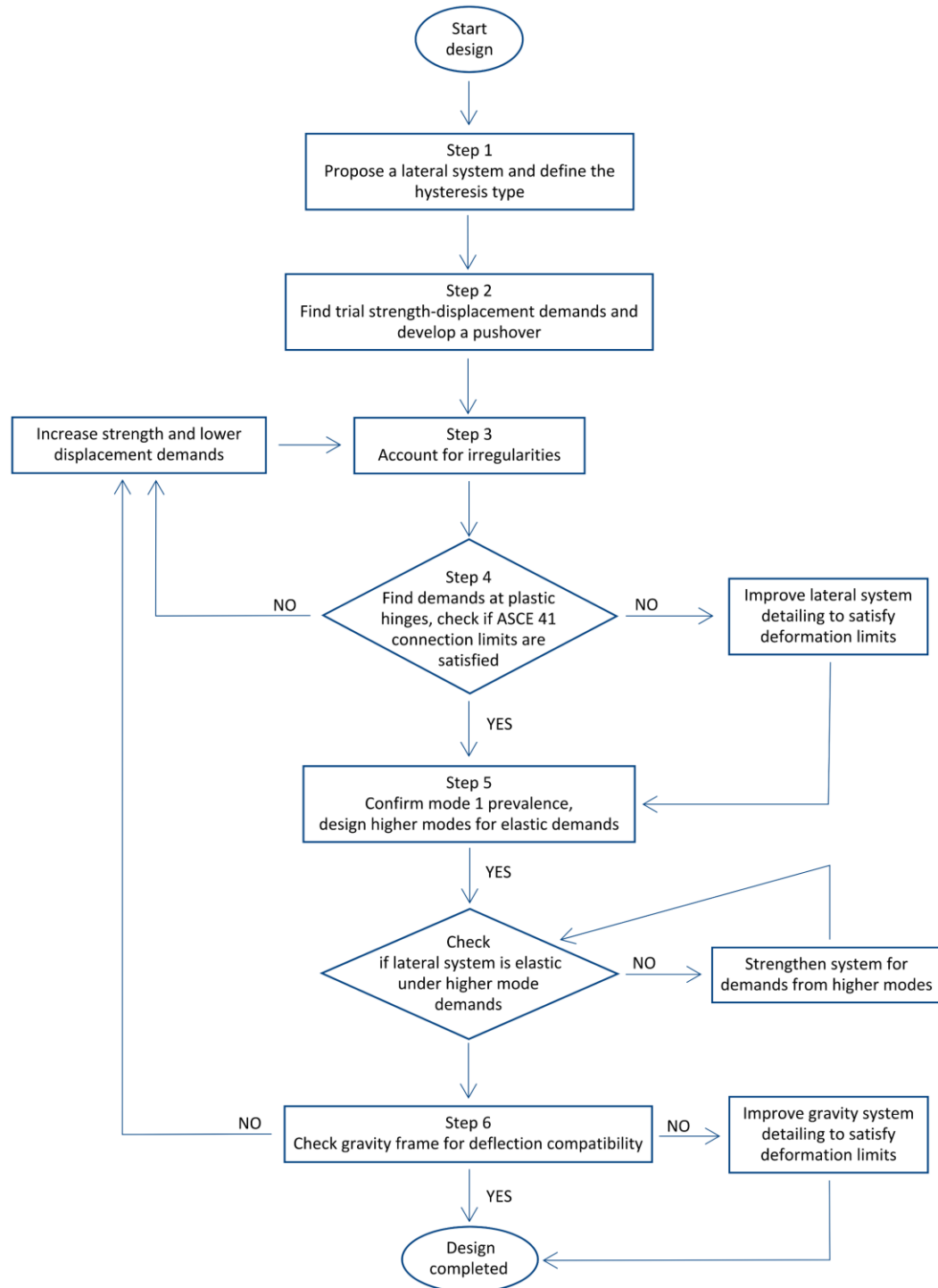


Figure 1: The flowchart of the design process.

RP5-4 Recommended Initial Limitations

Single-Degree-of-Freedom (SDOF) – The process only applies to seismic force-resisting system designs that can be readily idealized as single-degree-of-freedom (SDOF) systems. The lateral systems require a well-understood plastic mechanism, and this plastic mechanism is to be the only source of nonlinear action that would be included in the analytical model. The system should have similar strength and stiffness in the positive and negative excursions for each direction, to limit the potential for unbalance and ratcheting. The current recommendation is that the backbone curve for the positive and negative loading directions be similar. The differences in peak strength, peak displacement, and elastic stiffness are to be 15% or less.

Hysteretic Damping Assignments for the System – The method idealizes energy dissipation based on a concept of effective damping derived from the shape of hysteretic cyclic response. The nonlinear actions are displacement-based, and the plastic hinges are limited to those defined in ASCE 41-23, Section 7.4.3.2. This ensures the criteria for plastic hinges are suitable. The process in its current form cannot be used with velocity-dependent dampers. Designers can mix the types of hysteretic damping for a given system, but with stipulations for mixed systems, described below. The hysteretic damping options are steel (S), reinforced concrete (R/C), and post-tension systems with flag-shaped loops (PT). “Post-tensioned” refers to rocking systems, and the nonlinear elastic re-centering action can come from applied post-tensioning, or from weight tributary to the element, or a combination of both.

A system with an overall assignment of steel yielding for hysteretic damping (S) can have any combination of steel moment joints, buckling restrained braces, and eccentric yielding links. A system with a reinforced concrete hysteretic damping assignment (R/C) can have flexural joints in walls, beams, columns, and coupling beams, as well as diagonally reinforced coupling beams. Post-tensioned (PT) systems with rocking can be used with any steel yielding device for energy dissipation, such as buckling restrained braces (BRBs), machined steel bars, flexural beams, and so forth.

Hysteretic Damping Assignments with Mixed Nonlinear Elements – Systems can be created with a mix of nonlinear hysteretic elements. The current set of combinations are (S) + (R/C), (R/C) + (PT), (S) + (PT), and (S) + (R/C) + (PT). Figure 3 shows several example building systems with mixed hysteretic elements. When the nonlinear elements of a system are mixed, the hysteretic damping assignment of the system is to be based on the type of nonlinear element with the least hysteretic damping per cycle at a given peak force and peak displacement. Steel nonlinear elements have the most energy absorption per cycle, because of the full hysteretic loops. Rocking systems (PT) have flag-shaped hysteretic loops. These have the least energy absorption per cycle. Reinforced concrete hinges (R/C), are in between, because of the pinched hysteretic loops.

Foundation Modeling and Hysteretic Damping – Foundation flexibility should be considered when determining the elastic response. Systems that activate foundation rocking are allowed. If a

foundation rocks, prior to significant yielding of the supported wall, frame, or column, then the foundation would be idealized as undergoing a nonlinear elastic response, with no significant hysteretic damping. As such, it is to be treated as a nonlinear hinge (M_r) for the plastic mechanism, but treated as a pin, with respect to the energy absorption and ignored for the hysteretic damping assignment. Rocking foundations that have significant hysteretic damping, as well as foundations with significant radiation damping, are beyond the scope of this process.

Additional Limitations – The following additional limitations apply. Buildings shall be limited to those of Risk category II, and for those with structural heights above the grade plane, up to 8 stories and 120 feet tall. Systems that have a story mechanism shall be limited to 4 stories. The lateral system strengths shall have at least the strength of new code buildings with an $R = 8$, using nominal material properties and phi factors. This is intended to keep nonlinear actions from being excessive, and keep drift demands from exceeding those experienced by conventional gravity systems. When calculating the minimum strength, the fundamental period used for a system shall be limited to no more than the ASCE 7 limitation of C_u times the Approximate Fundamental Period ($C_u T_a$). Finally, the buildings should not be too flexible. The actual period of the system shall be limited to 2.0 times $C_u T_a$. The period calculation shall include foundation flexibility. The 2.0 factor on the Approximate Fundamental Period was selected based on a cursory review of projects. This factor will require validation and calibration.

Additionally, the first mode in each direction shall have at least 50% mass participation, and the first three modes in each direction shall have at least 80% mass participation. Finally, the diaphragms are to be designed using current code requirements.

RP5-5 Step 1 – Propose a Lateral System

In this step, the engineer proposes a lateral system and defines the hysteresis type. See Figure 2 for an example system and plastic mechanism. The plastic mechanism consists of plastic hinges at the base of the wall, at the outrigger beams, and at the base of the outrigger column. In this system, all nonlinear elements are reinforced concrete (RC) flexural hinges.

Designers benefit from flexibility to respond to architecturally complex buildings and challenging retrofit conditions. Within the limitations described above, designers can mix and match materials and develop non-conventional system configurations, to best suit their project conditions and performance needs. Figure 3 shows seven examples of systems for new buildings that currently do not neatly fit the Chapter 12 system tables of ASCE 7 (see 1, 2, 4, 5, 7, 8, and 9). Retrofits also benefit from a greater variety of system options (see 3, 6, and 10). While Tier 3, Linear Dynamic Procedures, of ASCE 41, provide unconstrained options, it can be problematic in that it relies solely on the elastic model and the initial load distribution. This obscures the potential nonlinear mechanisms, despite the use of m-factors representing the available nonlinear response of components. As a result, it is possible to attain undesirable mechanisms. See Appendix D for a more

detailed discussion. The behavior-based design process has the advantage of providing more direct control of the nonlinear response.

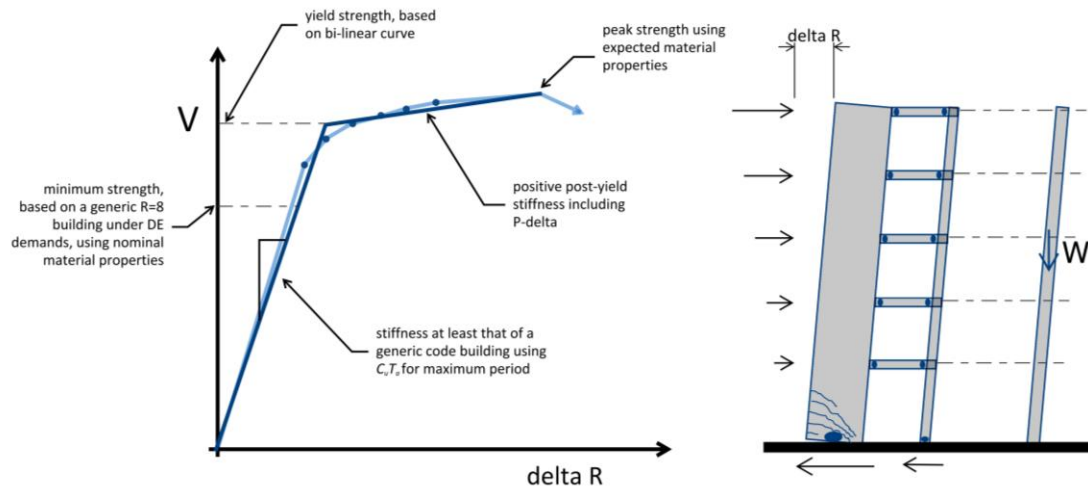


Figure 2: Mode 1 pushover curve for a representative “novel” system (System Example 1), including strength and stiffness constraints. The capacity is defined by the peak strength and the corresponding displacement.

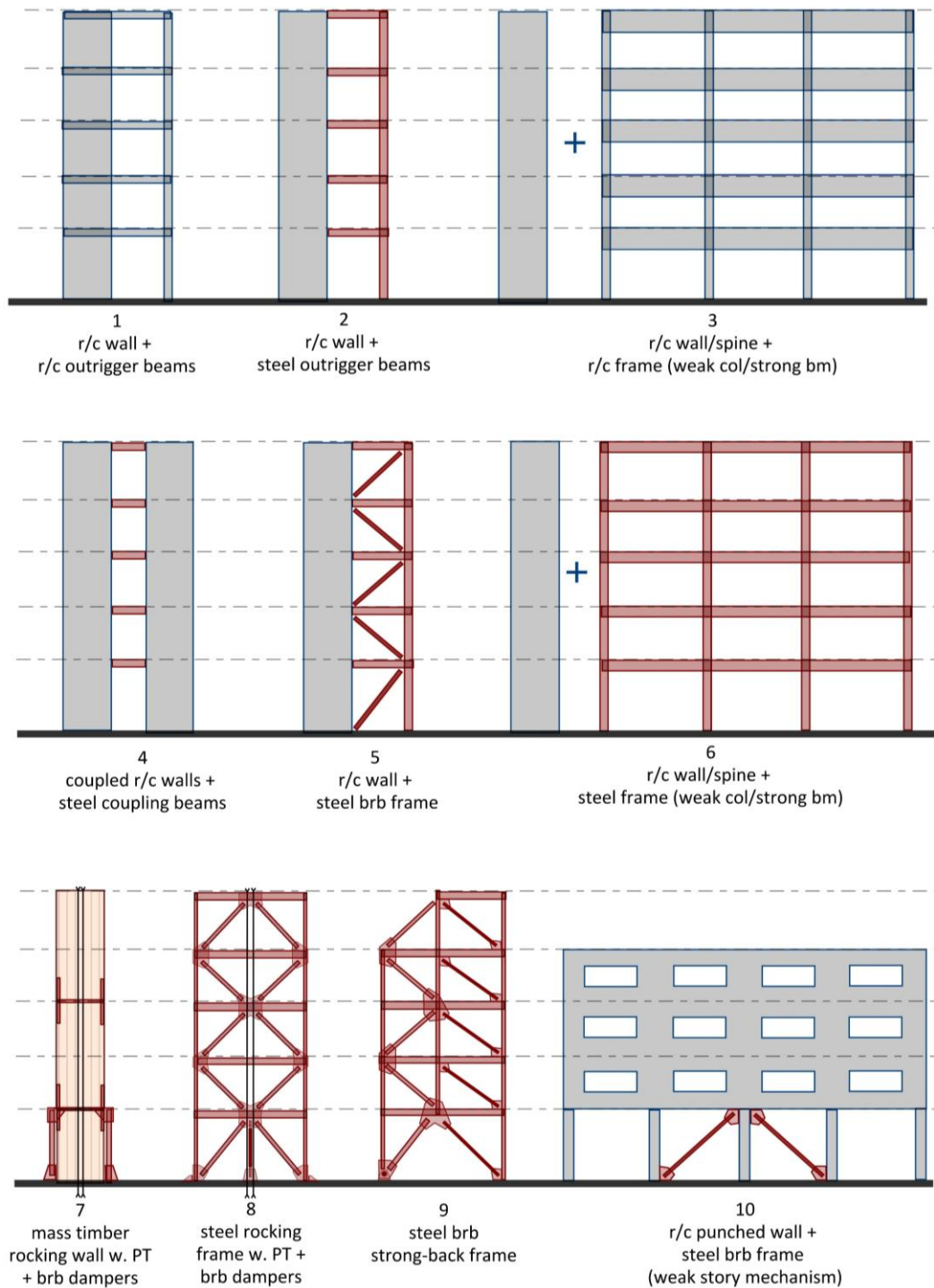


Figure 3: Examples of "novel" lateral systems.

RP5-6 Step 2 – Find Trial Strength-Displacement Demands and Develop a Pushover

In this step, the engineer determines the trial capacity of the system (see Figure 2, left). This includes the nominal strength and stiffness, as well as the peak strength, the displacement capacity at peak strength, and the displacement demands at the MCE_r.

Minimum Strength – The first sub-step is to determine the minimum nominal strength and stiffness, using ASCE 7, demands from the design earthquake (DE), and a generic Response Modification Factor (R) of 8. The nominal material values are to be used, and the stiffness is from the elastic model, and it need not include foundation flexibility. As noted in the *limitations* section above, the fundamental period of a system shall be no more than C_u times the Approximate Fundamental Period ($C_u T_a$). For T_a , use $C_t = 0.03$ and $x = 0.75$. While these are values for BRBFs and EBFs, they are suitable for constraining the acceptable period range. The minimum nominal strength and stiffness ensure that the system is not excessively weak or soft. This is important for two reasons. The first is to avoid problems with the deflection compatibility of conventional gravity systems. The second is to avoid excessive drift demands and nonstructural damage to conventional architectural systems.

Trial Pushover Curve – The engineer sizes the elements of the lateral system, based on the minimum nominal strength. In the case of the system Example 1, the R/C wall and outrigger beams are to be reinforced using nominal material properties, and phi factors. The expected yield strength and the pushover curve can then be determined by using expected material values (per ASCE 41-23) on the sections previously sized and reinforced. The loading pattern is a Mode 1 shape. The methods described in FEMA 440 and NIST (2017d) can be used for guidance in defining the plastic mechanism and creating pushover curves. A useful concept is to see the plastic mechanism as an assemblage of lateral load-resisting elements. These elements are either the plastic hinges, which are displacement controlled, or they are elastic elements, which are force controlled. The system (assemblage) needs to satisfy the strength and stiffness limits noted above and shown in Figure 2. The post yield stiffness is to be flat or positive, after the inclusion of P-delta effects. The system capacity is defined by the peak strength and the corresponding displacement. This point is reached as the nonlinear plastic hinges get exhausted and the system starts losing strength. The plastic hinge rotation limits are defined in ASCE 41-23.

The last elements to consider in the plastic mechanism are the foundations, which support walls or frames, as well as those supporting the columns. With this process, foundations are to be considered in one of two possible states. In the first state, the foundation is fixed, and it is strong enough to support a plastic hinge (at peak drift) at a wall, frame, or column. In the second state, the foundation rocks or the lateral element has a deliberate pin. The rocking foundation has a nonlinear elastic response, and little energy is dissipated during excursions. A rocking foundation is set as a

pin with the rocking moment (M_r), for the plastic mechanism, and ignored as an energy dissipating element. Deep foundations should be made strong enough to be fixed, using capacity design and expected strengths.

Ultimate Displacement Demands – The ultimate displacement demands are to be evaluated at the MCE hazard. The engineer uses a single-degree-of-freedom (SDOF) idealization of the building. A lateral system with known hysteretic characteristics can have any number of strength-displacement dyads that are sufficient for a particular hazard. A dyad is a particular combination of strength and the corresponding displacement demand. Note that the initial strength based on the pushover described above, is a minimum value for the process. The actual strength demand may need to be greater, and it has yet to be determined.

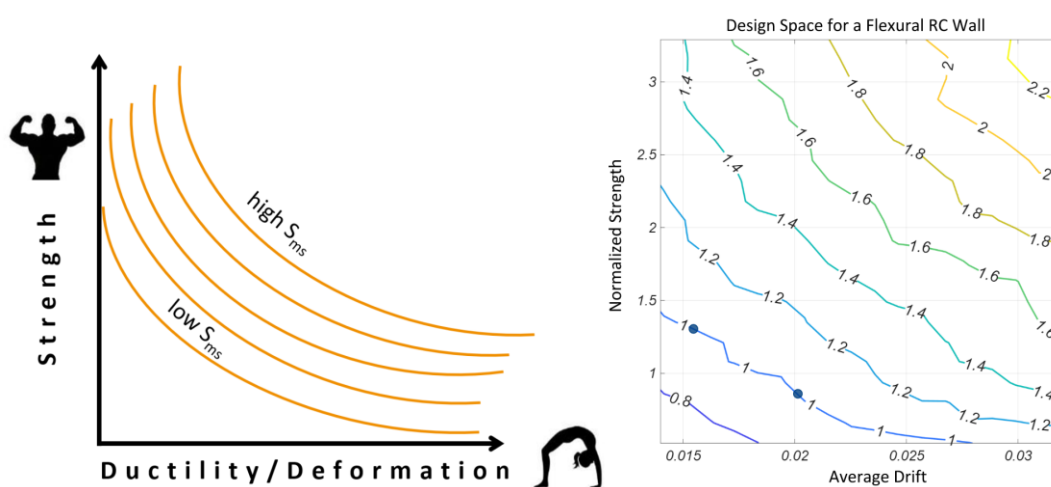


Figure 4: (Left) the conceptual diagram of a strength-deformation design space. (Right) a strength-displacement design space, for reinforced concrete (RC) wall system with a flexural plastic hinge, subjected the FEMA P-695, far-field records. It is called a Universal Design Space. The contours are created from different strength-displacement dyads, for a particular MCE hazard. See Appendix A for a detailed explanation.

For example, a strong building with lower displacement demand can have the same capacity as a weak building with a higher displacement demand. As mentioned above, a lower-bound strength is set to be that of a new building with a generic $R=8$. Selecting relatively low strength results in lower first costs for the system, but it would have higher drifts, and more damage and repair costs, as well as longer loss-of-use during repair. Higher lateral system drifts can also incur costs for special detailing of architectural systems. Selecting higher strength would have higher first costs. But the system would have relatively lower drift demands, and potentially less seismic damage. The choice of suitable strength-displacement dyads allows designers to iterate with strength and displacement limits, to find a desired performance choice. See the conceptual strength-displacement demand diagram in Figure 4 (left).

The strength-displacement dyads are to be selected using a nonlinear static analysis procedure. The two recommended methods are the Direct Displacement-Based Design (DDBD) and Capacity Spectrum. An engineer needs to apply one of the methods, and the choice is a matter of preference. The strength and displacement demands are at the MCE_r hazard. The DDBD offers the benefit of succinctly describing the interplay between strength, displacement demands, and the equivalent damping derived from the hysteretic damping type. Figure 5c shows the equivalent damping ratio for various nonlinear elements related to the displacement ductility. The formulas behind this figure are found in reference No. 6. They allow the engineer to determine equivalent damping. The Capacity Spectrum method uses a similar process to determine equivalent damping by calculating the area under hysteretic curve. The steel systems (S) conservatively correspond to the steel frame curve. Any type of reinforced-concrete hinge (R/C) is to conservatively correspond to the concrete bridge curve, which has less energy absorption than concrete frame curve. This curve is for the Takeda thin hysteresis model. The post-tensioned and rocking hinges (PT) correspond to the hybrid prestress curve. Figure 5d shows the displacement spectra with lower drift demands corresponding to higher hysteretic damping. In the case where a system has a nonlinear element of different types (a mixed hysteretic system), use the lowest hysteretic damping rate for the entire system.

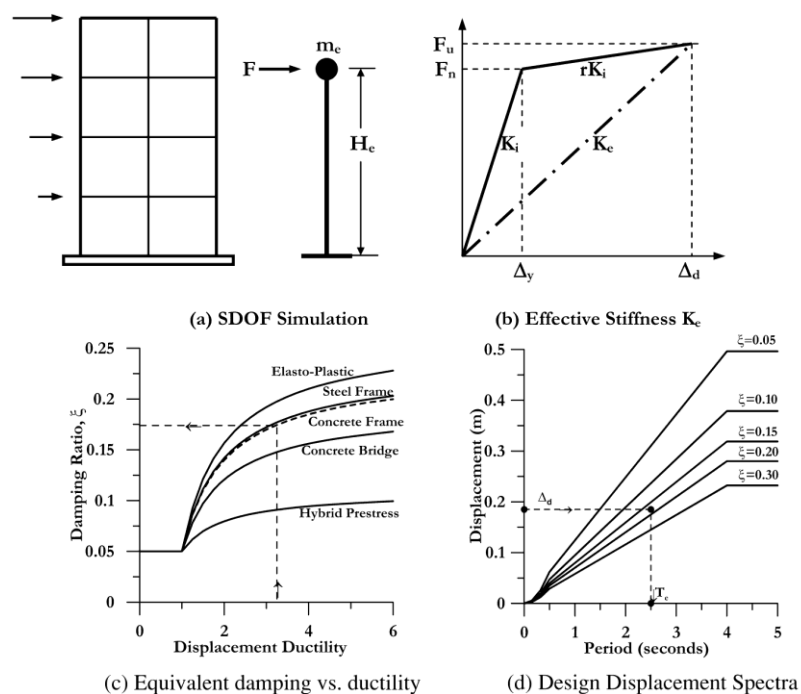


Figure 5: Overview of fundamentals of Direct-Displacement Based Design (Kowalsky et. al. 2024).

The DDBD process is iterative. It has a suggested workflow, starting the design by selecting the desired displacement, then determining the period T_e , from the displacement spectra. Using the

period and displacement, the engineer finds the required strength. See *Displacement-Based Seismic Design of Structures* (Priestley et. al.) reference No. 6 for details on how to apply the process.

The process could also be started by selecting a trial strength, then figuring out the drift demand. This is the workflow proposed for this method. This is because most engineers are used to working with minimum strength requirements, since this aligns with the current prescriptive code process. Once one knows their system's yield strength, the engineer uses the DDBD to determine the drift demand needed to satisfy the MCE_r . The engineer creates the initial pushover curve for the system, using expected properties. The pushover needs to be extended so that the displacement at peak strength is found (see Figure 2). The pushover is not used beyond the point of peak strength, because the system is losing strength and this is deemed to be the limit of usable capacity. The pushover is then simplified to a bilinear curve. The effective stiffness K_e is found with the secant stiffness, through the point of peak strength and the corresponding displacement. Using the effective stiffness K_e , and the effective mass M_e , one determines the effective period, T_e . The engineer then uses the bilinear pushover to determine the displacement ductility. Using Figure 5c, the equivalent damping ratio is found based on the type of hysteresis for the system. Knowing the effective damping ratio and the effective period T_e , the engineer can determine the displacement demand by using the Displacement Spectra for the MCE_r .

If the displacement demand at the MCE_r is lower than the displacement capacity at peak strength, then the design is adequate for this stage of the process. If the displacement demand is greater than the displacement capacity, then the system can either be made stronger (to lower displacement demands) or made to have higher drift capacity through improved detailing. If the system is to be made stronger, then the DDBD process needs to be performed again with a pushover for the revised system. Note that this displacement demand is for the center of mass of the building for an SDOF system. Step 3 is needed to account for potential increased demands due to irregularities.

See *Generalized Modified Modal Superposition Procedure for Seismic Design of Rocking and Pivoting Steel Spine Systems* (Martin and Deierlein) reference No. 9 for an example of using an equivalent linearization process, starting with strength.

A complementary process being developed is the creation of strength-displacement design spaces, referred to as Universal Design Spaces (see Figure 4, right and Appendix A for a more detailed explanation). The concept of these spaces combines the ideas of the equivalent linearization procedures such as Direct Displacement-Based Design (DDBD) with the incremental dynamic analysis process in FEMA P-695 (using the far-field record set). The products are design spaces of strength and displacement demand, mapping various hazards with contours at a consistent rate of collapse (10%). The design space shown is for a reinforced concrete (RC) wall system. In concept, it could be used for any lateral system with RC plastic hinges (System Examples 1 and 3) or used conservatively for a system with a mix of RC hinges and steel (S) plastic hinges (System Examples 2, 4, 5, and 6). For a given hazard, a designer could select any number of strength-displacement pairs

as a starting point for the design. In Figure 4 (right), two designs are shown with blue dots on the 1.0 MCE contour. The stronger design has a displacement demand of around 1.5%. The weaker design has a displacement demand of around 2.0%. This idea is in development, and an explanation of the study is detailed in Appendix A. Until the application of strength-displacement design spaces has been sufficiently calibrated, engineers should use established nonlinear static methods such as DDBD for any proposed design.

In all cases, the designer may need to iterate with strength, to keep plastic hinges within the ASCE 41-23 limits.

RP5-7 Step 3 – Account for Irregularities

In this step, the engineer makes any necessary adjustments to the system's deflection demands, from the single-degree-of-freedom (SDOF) deflection. Adjustments can directly account for both vertical and horizontal irregularities, in the displacement demands of a system. The process of increasing deflection demands differs from the ASCE7 approach for new buildings, where strength penalties and restrictions are applied to mitigate torsional imbalance. While the Tier 3, Linear Dynamic Procedures of ASCE 41, does not limit irregularity, the reliance of the elastic model may not adequately capture higher drift demands.

Vertical Irregularities – By examining the plastic mechanisms in Figure 6, the engineer can see the concentration of drift and rotation demands of the plastic hinges, for the weak-story condition on the right. Systems with a story mechanism shall be limited to 4 stories.

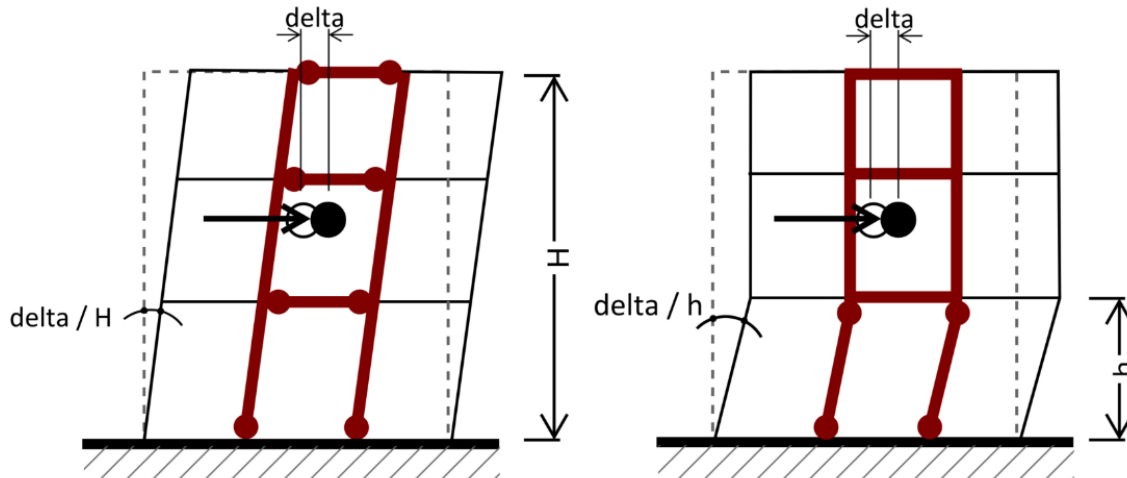


Figure 6: The center of mass deformation results in concentrations of rotation demands for the system on the right with the vertical irregularity.

Horizontal Irregularities – By examining the plastic mechanisms in plan as shown in Figure 7, the engineer can see the concentration of drift and rotation demands in the yielding frames, for the system with plan irregularity, shown on the right. The condition in Figure 7 represents the extreme plan irregularity often found with reinforced concrete property walls (blue) at existing corner buildings requiring retrofit. The strength and stiffness difference between the concrete walls, and new frames (maroon) is obviously large, thus the frame drift demands are near the upper limit of 2x the center of mass deflection.

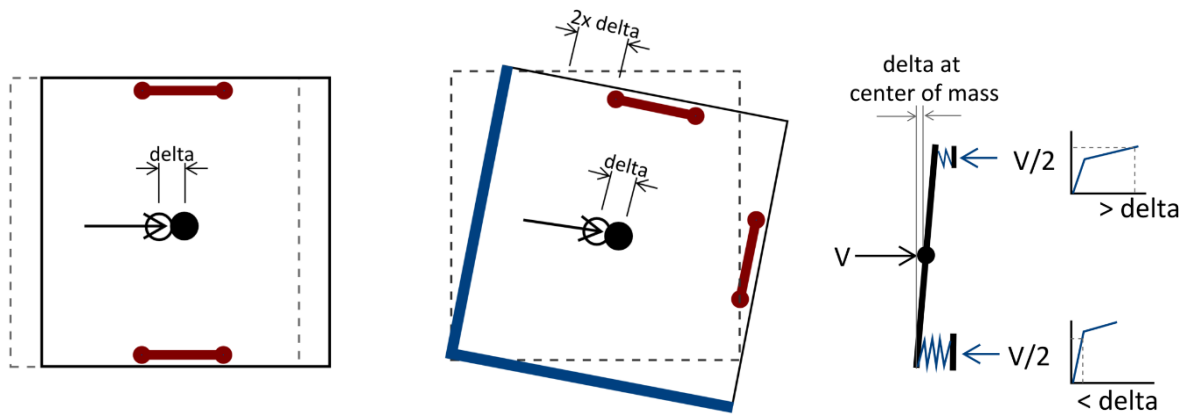


Figure 7: Examining the plastic mechanisms in plan illustrates the increased deformation demand of the system with the plan irregularity. To capture the increased displacement demand, use equilibrium and the nonlinear frame and wall responses. For any direction, neglect the contribution from perpendicular walls, because they may be subject to a simultaneous demand.

When the strength imbalances are lower, determine equilibrium using the strength (not stiffness) and the nonlinear response of the lateral elements. For both primary directions, neglect the contribution from perpendicular walls, because they may be subject to a simultaneous non-linear excursion, and have low stiffness. This constraint effectively precludes the use of three-sided systems. The simplified static solution, illustrated on the right of Figure 7, will provide a conservative deflection amplification.

Accidental torsion shall also be accounted for, using the code process. This test would ensure that a minimum level of torsional resistance is provided, such as in the case of a cruciform configuration of lateral elements.

RP5-8 Step 4 – Find Rotation Demands at Plastic Hinges, Check Against ASCE 41 Connection Limits

In this step, the engineer determines the rotation demands at all plastic hinges (or displacement demands at BRBs). These demands are then checked against the ASCE 41 connection limits for nonlinear actions. The nonlinear rotation check ensures that the system has sufficient deformation capacity. ASCE 41 connection limits are a single source of nonlinear components, with vetted deformation limits. If any plastic hinge demands are exceeded, the designer needs to iterate and improve the design. There are two options. The first is to improve the connection details, if possible,

to increase the nonlinear deformation capacity. The second option is to increase the strength of the system and reduce the overall deformation demands.

RP5-9 Step 5 – Ensure Prevalence of Mode 1 and Design Higher Modes for Elastic Demands

In this step, the engineer validates the SDOF idealization, by ensuring that the lateral system's primary plastic mechanism (associated with the first mode), is the only source of nonlinear action counted on to resist demands. This is required because of the SDOF idealization and the use of simplified static nonlinear analysis (DDBD or Capacity Spectrum). The primary plastic mechanism needs to have sufficient capacity to account for all the nonlinear actions. This condition is reasonable because the primary mechanism is proportioned adhering to capacity design, known plastic hinge demands, and known plastic hinge capacities from ASCE 41. Because the method is for novel systems that do not have extensive performance histories, the behavior of the higher modes needs to remain elastic. Designing nonlinear behavior in higher modes is too complex for the method.

Systems with spines, such as flexural and rocking R/C walls, rocking steel frames, and strong-back steel frames are susceptible to high demands from higher modes. Figure 8 shows higher-mode demands, relative to first-mode demands for rocking systems (see references 11 and 12). Similar results occur in strongback systems (see reference 10). The equivalent lateral force (ELF) procedure and response spectrum analysis (RSA) do not adequately address higher mode effects, and these procedures are unconservative for systems with spines.

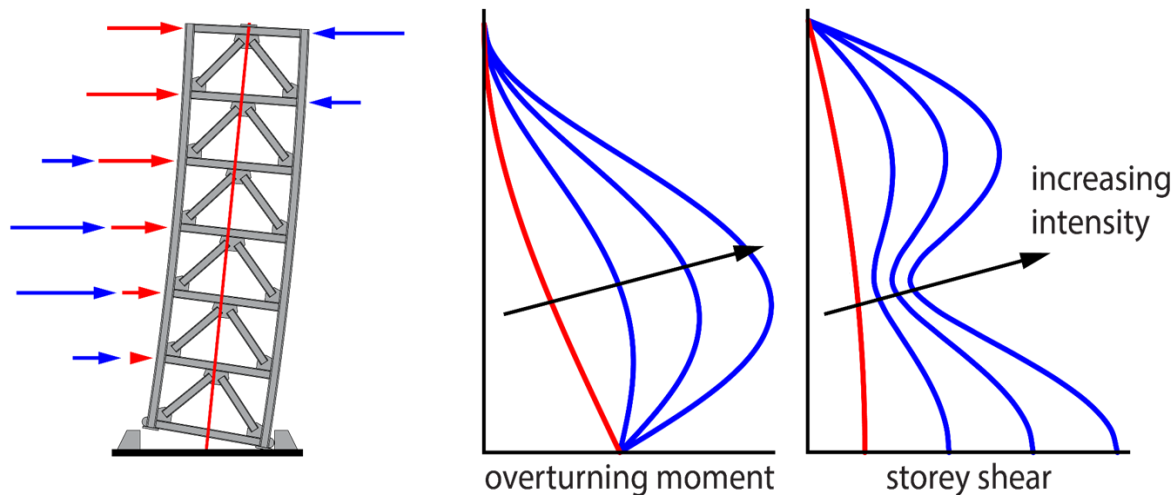


Figure 8: This figure is by Steele, T.C. and Wiebe, L.D. It illustrates the capacity protected mode 1 loads in red. The mode 2 loads in blue are not capacity protected.

The Modified Modal Superposition method (MMS) can be used to design systems for a single nonlinear mode, without the need for nonlinear dynamic analysis. This method of modal analysis keeps the higher modes elastic under the demands of the MCE spectrum. To implement the method, an equivalent linear model is created for the system, in its yielded state. For the rocking system shown, the inertial loads required to rock the frame are applied as forces F_1 - F_6 . The effect of the energy dissipator is the applied force F_{ED} . The effect of the post tensioning is the applied initial prestress load PT_0 , and a spring for the PT. The higher modes demands are determined, and the frame forces are computed from a modified SRSS (see Figure 9). A detailed discussion of this procedure, applied to rocking frames is by Steele and Wiebe (Reference 11).

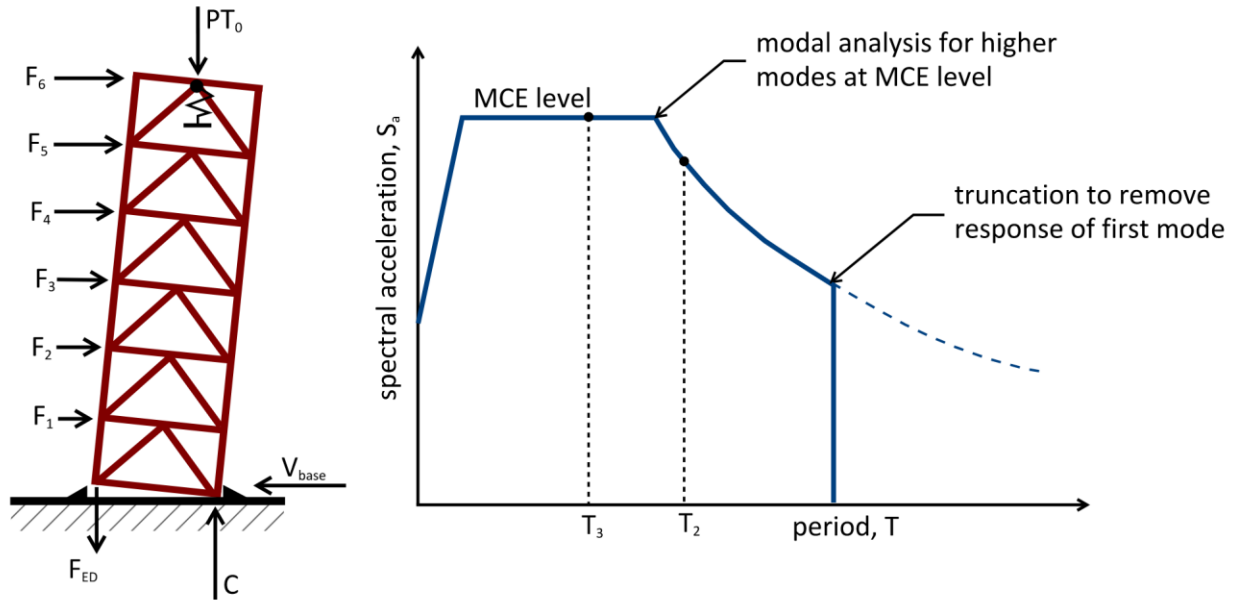


Figure 9: This re-drawn figure is from Steele, T.C. and Wiebe, L.D. The lateral system is placed in its yielded state. The truncated MCE response spectrum is used to determine the elastic higher mode demands.

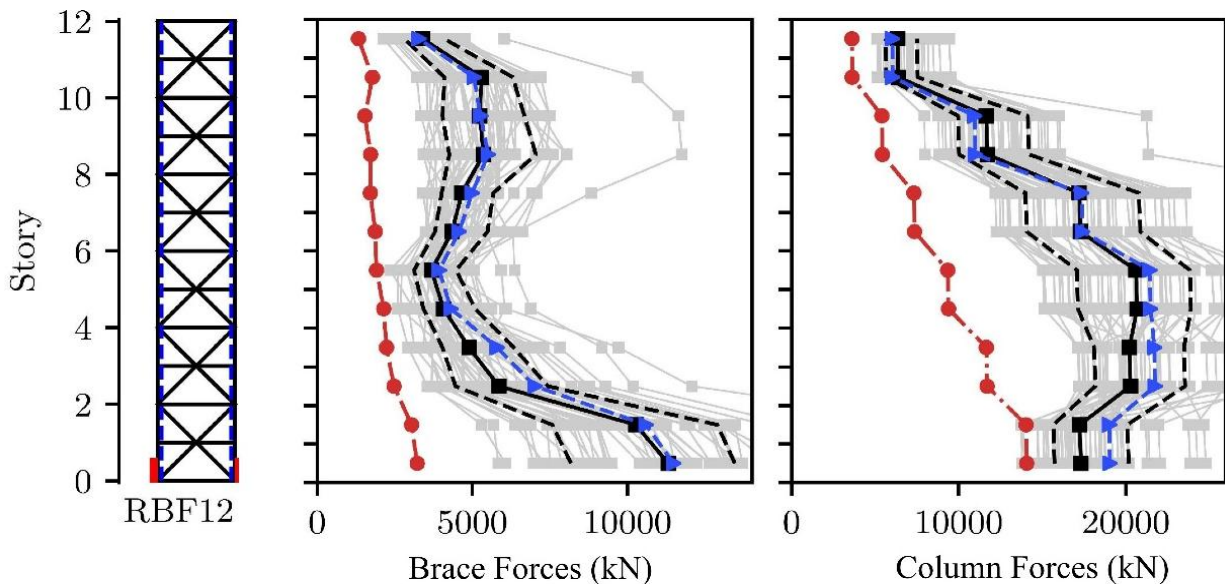


Figure 10: This figure is by Martin, A. and Deierlein, G. It shows the brace and column forces of a 12-story rocking frame. The MMS results in blue generally correspond to the median results of the NLRHA shown in black. The ASCE 7 results shown in red underpredict the brace and column demands.

A study of the MMS process applied to several steel rocking and strongback systems is by Martin and Deierlein (Reference 9). The paper validates the method with extensive nonlinear response-history analyses, as shown in Figure 10. The red traces are the ASCE 7 demands. The blue traces are results of the MMS, and they generally agree with the median NLRHA results shown in black.

RP5-10 Step 6 – Check Gravity Frame for Deflection Compatibility

In this step, the engineer determines the drift demands of the gravity frame at all the regions of the gravity frame. The system's deformation patterns need to be investigated for concentrated deformation demands, like the process in Step 3.

This is especially critical for concrete flat plate construction. For example, corner columns and the associated slab joints are susceptible to high deformation demands for buildings with plan irregularities. Ground floor columns and the associated slab joints are susceptible to high deformation demands for weak-story buildings and other systems with vertical irregularities. A linear model will be needed for thorough investigation. In some cases, the gravity system may undergo nonlinear actions. These should be checked to ensure that the ability to carry gravity loads is not compromised. Some limited nonlinear redistribution may be necessary. Appendix B is a proof-of-concept study, showing the importance of the deflection compatibility of the gravity frame. The study shows how the gravity frame controls the deformation limits of concrete building under many conditions.

Hogging – Other sources of deflection compatibility damage come from the interactions of walls or frames, with concrete slabs and steel floors. When walls form plastic hinges, the tension face of the wall uplifts in proportion to the base rotation plus contributions from the elastic deformation of the wall, creating localized uplift (aka hogging) in the slab. Hogging occurs with traditional reinforced concrete flexural walls, PT rocking walls, PT rocking steel frames, and PT rocking mass timber walls. The local uplift creates internal moments and corresponding shears within the floor system. See the potential plastic hinges in Figure 11, right. For concrete flat plates, column strips are used to represent internal beams. These are then checked for internal moments and shears. This effect is a source of additional strength and stiffness, although it is rarely utilized. It is also a source of damage.

If the deformation capacity of any gravity frame element is exceeded, the designer needs to iterate and improve the design. There are two options. The first is to improve the connection details, if possible, to increase the nonlinear deformation capacity. With concrete columns, this may be accomplished with increasing transverse reinforcement. With concrete beams, and concrete slabs, increasing transverse reinforcement (stirrups and stud-rails) is often the most efficient way to improve deformation capacity. Local reductions to longitudinal reinforcement can also reduce end moment demands, and the associated shear stress. For reinforced concrete columns in existing buildings, carbon fiber reinforcement is commonly used to mitigate shear critical members and

improve the rotation capacity at plastic hinges and lap splices. The second option is to increase the strength of the lateral system, and reduce the overall deformation demands.

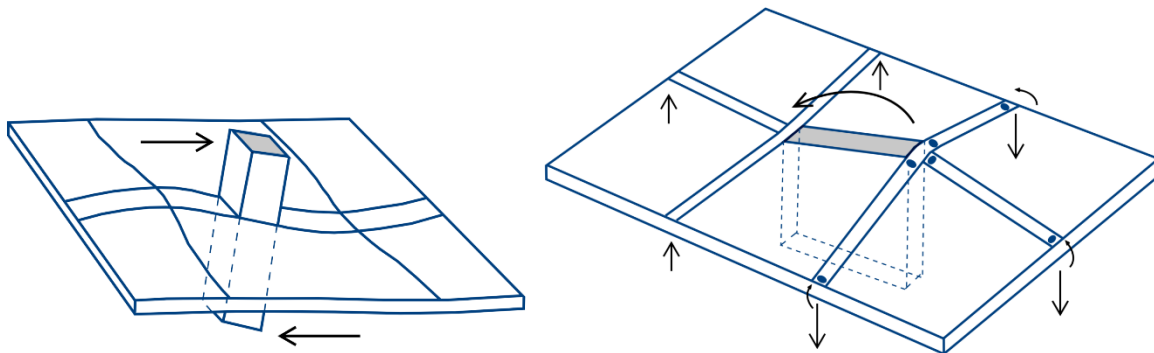


Figure 11: Concrete flat slabs (left) are highly susceptible to punching shear damage. Slabs adjacent to walls (right) are also vulnerable to local uplift loads (hogging) and the associated internal moments and shears.

RP5-11 Limited Peer Review

This process requires the structural engineer to directly work in the nonlinear realm, with the creation of a plastic mechanism for the system, the plotting of pushover curves capturing nonlinear events, and proportioning members using capacity design. Many building departments (AHJs) may not have the technical capacity for an effective design review. Where this is the case, a limited peer review is recommended. A brief outline of a peer reviewer's responsibilities would be the following:

1. Ensure that the strength and displacement demands are correct based on using one of the equivalent linearization methods (Step 2).
2. Ensure that the mechanism and corresponding pushover curve satisfy equilibrium at each event as well as other explicit requirements (Step 2).
3. Ensure that the capacity design of the lateral system has been satisfied (Step 2).
4. Ensure that irregularities are correctly accounted for (Step 3).
5. Ensure that the plastic hinge demands are within the connection limits of ASCE 41 (Step 4).
6. Ensure that the system can adequately resist higher modes (Step 5).
7. Ensure deflection compatibility of the gravity frame (Step 6).

The peer reviewer is required to be a practicing structural engineer with experience with nonlinear design. The framework requires that a limited and explicit set of items be validated. This would allow a single reviewer to cost effectively ensure that the technical needs are met.

RP5-12 Conclusion

This paper presents a conceptual framework for the Behavior-Based Design (BBD) process. The process will evolve as it is validated and calibrated. There are numerous items presented in this paper that need future validation. These include, but are not limited to the following:

1. *Weak-story system limits*. Determine the requirements needed for story mechanisms, to ensure acceptable performance. These include validating the maximum number of stories, the weak-story strength, and the weak-story drift limits.
2. *Torsion limits*. Determine a maximum torsion limit, if needed, to ensure acceptable performance.
3. *Minimum torsion*. Determine the minimum requirements for torsional resistance. This is to guard against systems with little torsional demand and little torsional resistance, such as a building cruciform shaped in plan. Determine the requirements for accidental torsion tests to get acceptable performance.

An analytically complete design example using the BBD process is a topic of a SEAONC sponsored University Research Program. The design example will include validation of multi-degree-of-freedom (MDOF) models with nonlinear response-history analyses including a FEMA P-695 collapse assessment. The results of the study will be available in the fall of 2026.

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RP5-Appendix A: Universal Design Spaces for Strength and Displacement

RP5-A.1 Abstract

The intent of this concept study is to create a system and material-independent design space for lateral elements, based on three parameters: strength, displacement ability, and hysteretic characteristics. This paper links equivalent linearization procedures such as Capacity Spectrum and Displacement-Based Design with the FEMA P-695 system. The products are design spaces of strength and displacement demand, mapping various hazards with contours targeting a consistent rate of collapse (10%). The proof-of-concept study examines two reinforced concrete archetype buildings, a conventional and a rocking re-centering type. Each has distinctly different hysteretic characteristics (pinched loops vs. flag-shaped loops). This design space demonstrates that a lateral system with given hysteretic characteristics can have any number of strength-displacement dyads for a particular hazard. If this concept were to be progressed, it would give engineers freedom to select suitable strength-displacement demands, with the confidence of having capacity consistent with the FEMA P-695 uniform collapse methodology.

RP5-A.2 Introduction

When engineers are exploring novel lateral systems, often with a mixture of materials, they lack guidance to proportion the strength and displacement demands, short of doing a time-consuming and complex analysis for alternative structural systems per ASCE 7-22 or ASCE 41-23. To solve this shortcoming, the design space maps strength and displacement requirements, with a particular hysteretic loop shape. The results are contours at various hazard levels.

This work builds on the equivalent linearization methods such as Capacity Spectrum and Direct Displacement Based Design (DDBD). It couples the clear relationships between strength, displacement capacity, and hysteretic shape with the FEMA P695 uniform collapse methodology.

The design space allows engineers to invent or explore novel systems, given the hysteretic loop shape and hazard. The designer can select different combinations of suitable strength-displacement demands, independent of material or system type, allowing design freedom.

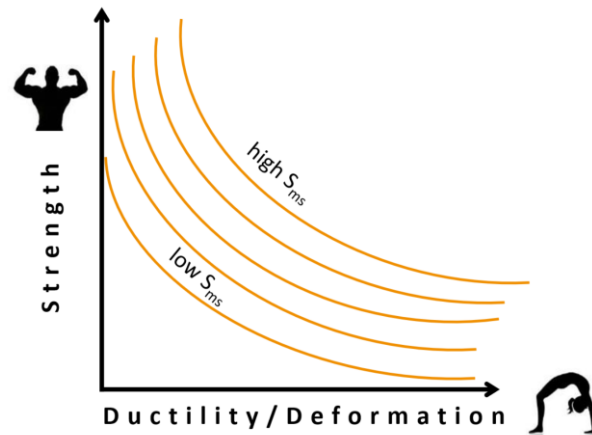


Figure A-1. Initial conceptual sketch of the strength-deformation design space.

RP5-A.3 Equivalent Linearization and FEMA P440A

The equivalent linearization method, Direct Displacement-Based Design (DDBD), clearly demonstrates the relationship between strength, displacement, and hysteretic characteristics. These simple relationships provide behavioral understanding, enabling design freedom and creativity. If a designer knows a system's hysteretic characteristics, various combinations of strength and displacement demands can be found for any hazard (see Figure A-2). The system needs to adhere to SDOF simplification.

FEMA P440A can be seen as an extension of DDBD where the shape of the backbone curve and the hysteretic characteristics can be used to quantify collapse rates using Incremental Dynamic Analysis (IDA). The elastic and plastic strain, hardening sections of the backbone curve (Figure A-3) correspond to the pushover curve in DDBD (see Figure A-2b). While FEMA P440A explored the collapse rates of numerous systems (SDOF spring types), this study focuses on well-behaved systems, like those for a modern building (as shown in Figure A-3).

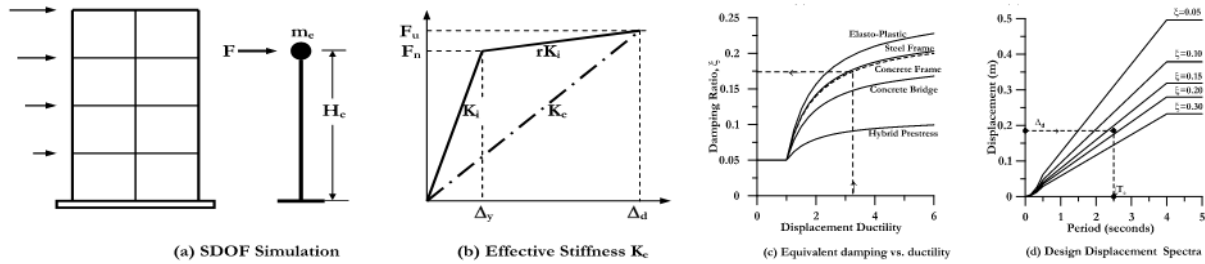


Figure A-2. Overview of fundamentals of Direct-Displacement Based Design (Kowalsky et. al. 2024).

The well-behaved system in FEMA P440A was expanded to create a design space. We used FEMA P-695 to implement Incremental Dynamic Analysis. FEMA P-695 was selected because it is used to unify collapse risks for the validation of new systems.

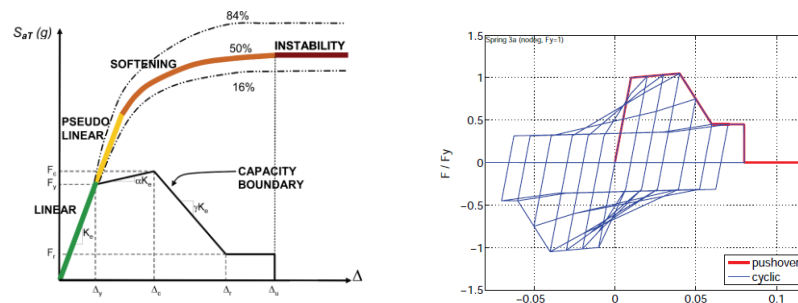


Figure A-3. (a) Relation between IDA curves and force-displacement capacity boundary (b) Hysteretic and backbone curve for a spring with pinching and cyclic strength degradation

RP5-A.4 Archetypes

There are two archetype families. One family has conventional flexural walls, and the other family has rocking walls. All buildings are four-story reinforced concrete buildings, designed for a site in downtown Los Angeles. They have a 15-foot ground floor height and a 12-foot typical story height. There are two walls in each direction, and the plan dimensions are 120 feet x 60 feet.

The conventional flexural walls were designed in accordance with ACI 318-19 and ASCE 7-22. The behavior of concrete walls was characterized by full hysteretic loops based on the parameters specified in Table 7.4.1.1.1a, ASCE 41-23. To capture a broad range of behavior, twenty-one displacement capacities were established by interpolating between the upper (most ductile) and lower (least ductile) limits defined in ASCE 41. Twenty-one strength levels were set. There are 441 versions of conventional concrete buildings in this archetype that were used to create a design space. Figure A-4c shows an example of a concrete wall section and the full hysteretic loop.

The rocking flexural walls were designed in accordance with ACI 550.7-19. The behavior of rocking walls is characterized by flag-shaped loops, as shown in Figure A-4d. Ten levels of displacement capacity were used. This was achieved by varying the length of energy dissipation bars. Eleven levels of strength were selected. For these walls, failure occurred by the fracturing of energy dissipator bars. The rocking walls were detailed with armored steel plates at the wall ends—about which the wall rocks to prevent compression failures. There are 110 versions of rocking concrete wall buildings in this archetype family.

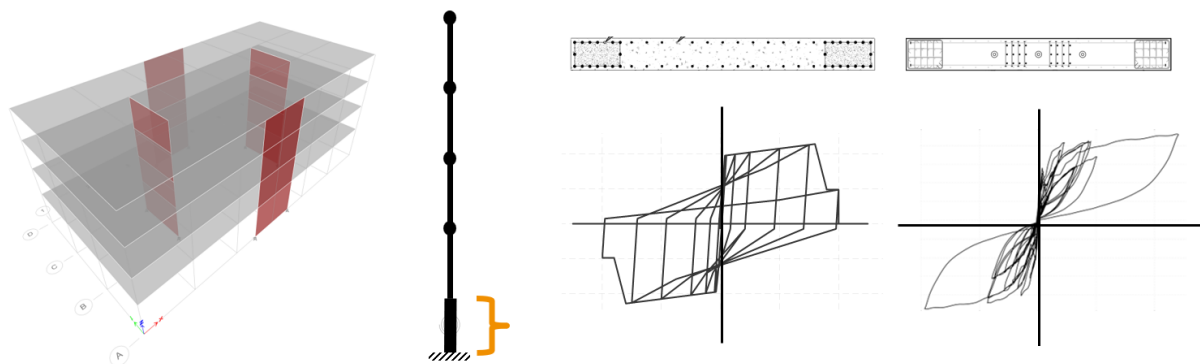


Figure A- 4. (a) 3D Analysis Model (b) Schematic of Concentrated Plasticity Model (c) Conventional wall section with a full hysteresis loop (d) Rocking wall section with a flag-shaped hysteresis loop.

RP5-A.5 Experiment

Nonlinear models were created with OpenSees, for the buildings in both archetype families. Buildings with conventional walls were modeled with elastic beam-column elements with concentrated plasticity at the base, using Pinching4 material. Buildings with rocking walls were modeled with the SelfCentering concentrated plasticity material. The archetypes were regular 2D buildings with no torsion effects. There was no detrimental gravity failure considered, and the strength and stiffness contributions from the gravity system were neglected.

The FEMA P-695¹⁰ analysis used the far-field motion record set of 22 paired ground motions for the Incremental Dynamic Analysis. Each direction was used giving 44 ground motions, and 50 scale factors. For both archetypes, “collapse/failure” is defined during the analysis, when a wall first reaches its point of peak strength, and the corresponding displacement. This is the point on the pushover when the wall starts to lose strength. The uncertainty modification factors were 0.4 for

¹⁰ There are plans for a major update to FEMA P-695. This study would need to be revised if the process were updated.

record to record, 0.35 for modeling, 0.10 for test data, and 0.10 for design requirements uncertainty.

RP5-A.6 Creation of Design Spaces

The conventional wall design space and the rocking wall design space are shown in Figures A-5a and A-5b, respectively. Conventional walls have relatively more energy dissipation per cycle. However, these walls also have relatively lower displacement capacities. In comparison, rocking walls with their flag-shaped hysteresis loops have much less energy dissipation per cycle. However, rocking walls have relatively higher displacement capacities. The differences in the design spaces are explained with the DDBD. Figure A-2c of the DDBD graphic shows the differences in the hysteretic damping between the conventional systems (Takeda Fat/Concrete Frame) and the rocking systems (Hybrid Prestress). This corresponds to differences in displacement demands, as shown in Figure A-2d.

RP5-A.7 Design Space Plots

The vertical axis is the normalized strength M_n/M_u . M_n is the flexural capacity of a wall with expected properties. It is the yield moment on the wall's bilinear pushover curve. M_u is the moment from the code equivalent lateral load based on $R=5$. The horizontal axis is the average drift (roof drift/building height). The drift capacity corresponds to the drift at the peak strength.

To create the space, the normalized strength and drift capacity of each building is first plotted in the design space. The design space for the conventional wall has a 21 x 21 array corresponding to the 21 strength levels and the 21 displacement levels. The design space for the rocking wall has an 11 x 10 array corresponding to the 11 strength levels and the 10 displacement levels. For each building, the IDA is performed, and its adjusted fragility is determined. The spectral acceleration, which corresponds to a 10% probability of "failure," is used to assign a preliminary hazard contour assignment for the particular building. "Failure" is defined by the building reaching the drift at peak strength, just before strength loss. Once all preliminary hazard contour assignments are applied to the entire array, the contours are fit to the data to create the design space. The contour of 1.0 corresponds to the MCE hazard for a site in downtown Los Angeles, California.

RP5-A.8 Conclusions

While the archetypes used for the proof-of-concept study are limited, the design spaces show the interplay of strength and displacement demands for conventional and rocking systems. Seeing a design space with abstracted system parameters encourages creativity and the use of novel configurations, once the designer sees that they need only to work with strength, displacement, and hysteretic characteristics. The requirement to ensure single degree-of-freedom behavior is essential to the application of this simple design tool. Additional building heights and hysteretic systems can also be created using the same process. The long-term vision is to have numerous versions of such design spaces available for structural engineers. These design spaces will need to be peer-reviewed.

The plots are presented as a potential alternative to the R, Cd, structural system table in Ch. 12 of ASCE 7. The current plots represent contours of buildings reaching peak strength, just before strength loss. This can be deemed to be a collapse prevention performance state. The design spaces could also be reformulated for higher performance states corresponding to lower nonlinear demands. The higher performance could be immediate occupancy, functional recovery, and life safety, using the same analysis data.

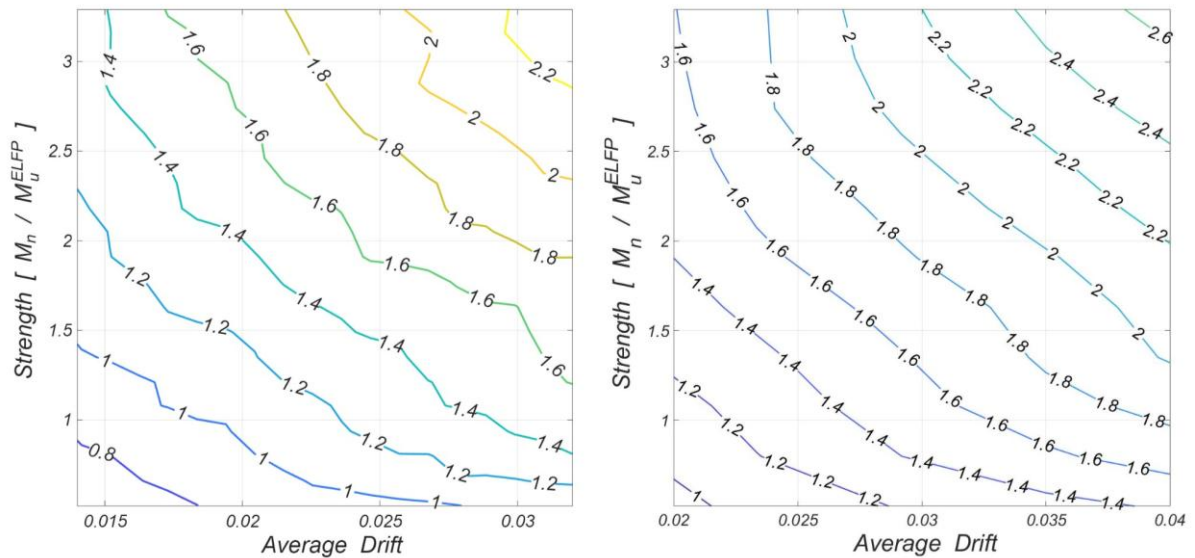


Figure A-5. Uniform Collapse Probability Curve (a) Conventional RC wall with full hysteresis loop (b) Rocking RC wall section with flag-shaped hysteresis loop

RP5-Appendix B: Gravity Frame Compatibility Experiment

RP5-B.1 Introduction

This proof-of-concept study supports the importance of Step 6 of the design process—the gravity system deflection compatibility checks. This study uses four configurations of a 4-story reinforced concrete wall building, with flat plate slab construction. The study compares the building with no torsion, against versions with three levels of torsional imbalance. The study finds the rates of failure and determines when the failures arise from limits of the walls (the lateral system) or limits of the slab column joints (the gravity system). The collapse rates were controlled by limits to the gravity system in cases with moderate to large torsional balance.

RP5-B.2 The Experiment

The archetypes are like those used in the Appendix A study, with respect to the building size, weights, and so forth. The walls are well confined, with low axial load, allowing the largest rotation capacity for the plastic hinges per ASCE 41-17, Table 10-19. Lumped plasticity hinges, modeled with Pinching4 in OpenSees, are provided at the base of each wall. The flat slabs are reinforced with stud rails, to improve the rotation capacity of the slab column interface joints. The gravity loads are assumed to be $0.5 V_w / \phi V_c$, and the joint is deemed to have an ultimate drift capacity of 6%. (See Figure B-1 on the right). The strength and stiffness of the slab-column system were not added to the model. The drifts at the corners of the building were used to monitor the slab-column joints. The wall drift limit was determined when the individual Incremental Dynamic Analyses (IDAs) for the walls became flat.

There are four configurations of the building. T0 has no torsional imbalance, and T1, T2, and T3 have progressively larger imbalances; see Figure B-2. The buildings were subjected to incremental dynamic analyses, per FEMA P-695, and the far-field record set was used. The adjusted fragilities of each building are plotted in Figure B-3. The solid lines are the fragilities of the controlling wall in each of the buildings, with no consideration of the gravity system. The dashed lines are the fragilities of the gravity system. The gravity system fragilities are established by monitoring the peak drifts of the four corners of the buildings on each floor. When any column reaches the drift limit of 6%, then the event is deemed to represent a collapse.

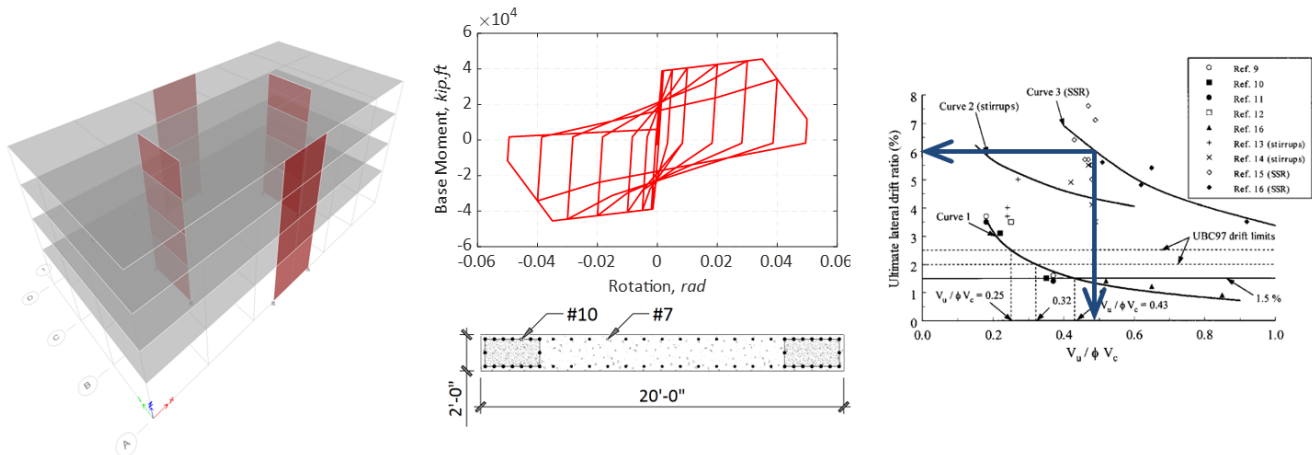


Figure B- 2: The 4-story R/C wall building (left). The walls are modeled with lumped plasticity Pinching4hinges in Opensees (middle). The column-slab joints limit drift of the gravity system to 6% ultimate drift (right).

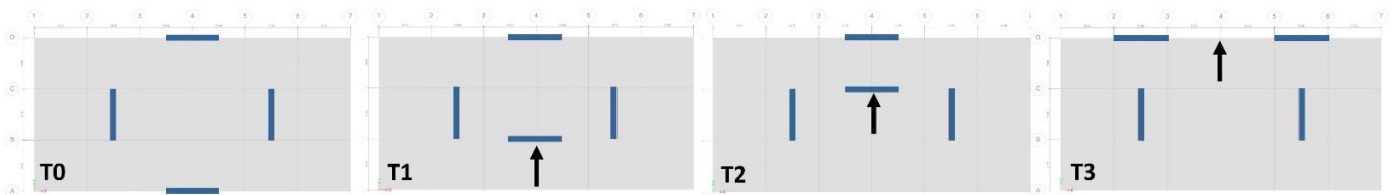


Figure B- 1: The wall layouts with varying torsional unbalance. T0 has no unbalance. T1-T3 have progressively increasing torsional unbalance.

RP5-B.3 The Conclusions

The proof-of-concept experiment utilized a well performing lateral system (R/C walls) and a well detailed flat slab capable of deflecting 6%. The drift limit of the slabs far exceeds the code drift limits for new buildings.

The capacity of the building can be controlled by either the lateral system or the gravity system. The gravity system controlled the building's capacity in cases of moderate to extreme torsional imbalance (T1-T3). The study shows that even small torsional imbalances can exacerbate deflection demands. The wall collapse rates are relatively insensitive to the imbalance, but not so with the gravity system. Unlike the code for new buildings, the BBD process does not limit the plan irregularity of the system. This gives designers more options for managing irregular buildings, as is often needed with retrofits. To avoid negative consequences due to irregularities, Step 3, of the design process requires the engineer to account for irregularities of the lateral system and determine the higher drift demands from the irregularities. The results of this study confirm the importance of Step 6 in the design process.

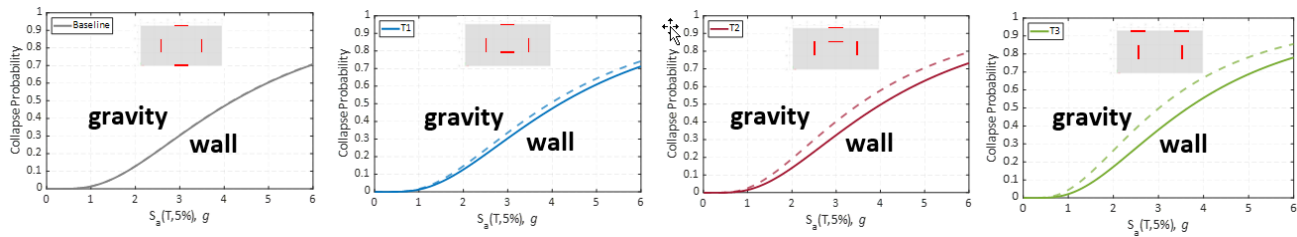


Figure B- 3: Fragility plots are shown for the four archetypes with increasing torsional imbalance (T0-T3). The solid fragilities are plots of the wall capacities. The dashed fragilities are plots of the gravity system capacities.

RP5-Appendix C: Torsion Effects on Displacement Demand

RP5-C.1 Introduction

This proof-of-concept study supports the importance of Step 3 of the design process—the accounting for plan irregularities. As with Appendix B, this study uses the same four configurations of a 4-story reinforced concrete wall building, with flat plate slab construction, and various levels of torsional imbalance. In this study, the rate of collapse is explored as torsion is increased. This is important because, unlike ASCE 7, the Behavior-Based Design (BBD) process does not preclude excessive torsion. Per Step 3 requirements, accidental torsion must be accounted for using the current code process. This ensures a minimum level of torsional resistance, particularly for layouts like a cruciform configuration of lateral elements. The goal is to provide design flexibility, whenever possible, as long as the safety impacts of the irregularity can be assessed and addressed. The process treats plan irregularity as a cause of increased deflection demands. As such, both the gravity and lateral system need to be evaluated at the higher deflections when plan irregularities are present.

RP5-C.2 The Experiment

The archetypes are like those used in the Appendix A study, with respect to the building size, weights, and so forth. The walls are well confined, with low axial load and low shear stress, allowing the largest rotation capacity for the plastic hinges per ASCE 41-17, Table 10-19. The wall drift limit was determined when the IDAs for the building became flat. There are four configurations of the building. T0 has no torsional imbalance, and T1, T2, and T3 have progressively more imbalances, see Figure B-2 (above). The buildings were subjected to incremental dynamic analyses, per FEMA P-695, and the far-field record set was used. The gravity system is not being evaluated with this study.

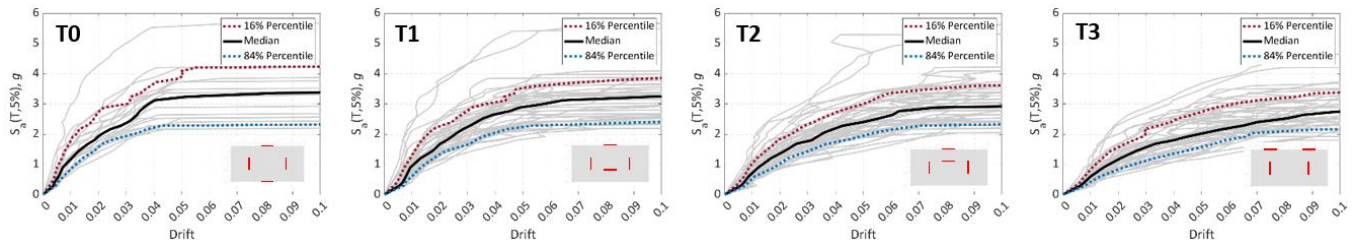


Figure C- 1: The incremental dynamic analyses are plotted for the four archetypes, with increasing amounts of torsional imbalance.

The incremental dynamic analyses are plotted in Figure C-1. The solid lines are the median of the IDAs. The red dashed lines and the blue dashed lines are the 16% and 84% percentiles, respectively.

In Figure C-2, far left, the median of the IDAs for T0-T3 are plotted together, along with the MCE demand at the considered location of 1.8g. One trend is that the increase in torsional imbalance lowers the building's capacity. Additionally, buildings with greater torsional imbalance need to undergo larger drifts to reach the reference capacity. Building T3 needs to undergo a peak drift of 4% to reach the reference capacity. In comparison, building T0 only needs to undergo a peak drift of around 2% to reach the reference capacity. Both buildings may be adequate, but in the case of T3, the rotation demands of the system, and the drift demands of the gravity frame are twice that of T0. This shows the effectiveness of increasing drift demand (via Step 3) as a means of ensuring an adequate design.

In Figure C-2, the second from the left to the far right, the benefits of adding additional strength to the walls are explored. The plot for T1 (second from the left), shows that when a building has moderate levels of imbalance, adding strength (T1 x 1.45) is sufficient to reduce the drift demand to be that of the T0 building with no torsional imbalance. This conclusion aligns with the code practice (ASCE 7-22) of strength penalties for torsion. However, in the cases of extreme torsional imbalances, T2 and T3, adding 45% more strength is insufficient to reach the baseline drift demand of the T0 building with no torsion. This suggests that accounting for the increased deflection demand may be the only effective means to design a safe building with large torsional imbalance.

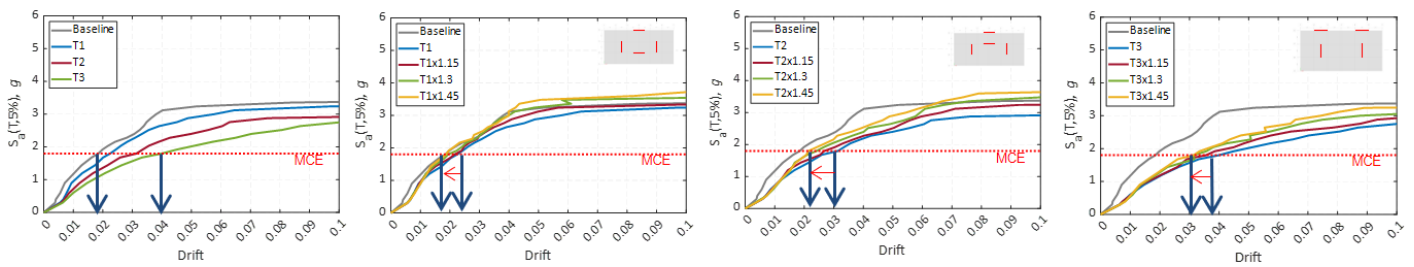


Figure C- 2: (Far left) Median IDAs for T0-T3 with MCE demand. (Second from the left) T1 median IDA, with median IDAs of stronger versions of T1 (x 1.15, x 1.3, and x 1.45). Similar plots are shown for T2 (second from the right) and T3 (far right).

RP5-Appendix D: A Critique of ASCE 41, Tier 3 Linear Dynamic Procedures

The Tier 3 Linear Dynamic Procedures in ASCE 41 are generally effective in providing engineers with flexibility to design retrofits and manage the complexities and irregularities that are common to retrofits. Moreover, the use of m-factors provides design insights with respect to a component's ability to sustain nonlinear actions.

A major shortcoming of the process is the sole use of the elastic model. As a result, the building's mechanisms are obscured, and engineers cannot be certain of a system's performance. A hypothetical condition is the retrofit example below in Figure D-1. The original condition is comprised of the concrete pier and spandrel system above the ground floor, and bare R/C columns below. Clearly there is the need to add strength to the weak-soft ground floor. Using the Behavior-Based Design process, an engineer could explore a retrofit using a ductile weak-story plastic mechanism, as shown on the right. It would require checks and/or improvements to the ground floor columns for the concentrated deformations. This would occur in Step 3 and Step 6 of the process. For this discussion, the columns are improved for displacement with FRP reinforcement. The nonlinear action could be confined to the ground floor, and the retrofit would preclude brittle shear failures on the upper floors. The design strategy is to achieve a sweet spot of being strong enough for the demands, but not too strong to force damage to the upper structure. This is the strategy adopted by FEMA P-807 for the retrofits of weak-story multi-unit apartment buildings. However, to achieve this, an engineer needs to understand the desired mechanism. Specifically, he/she needs to avoid over strengthening the ground floor braces.

Unfortunately, the Tier 3, Linear Dynamic Procedures do not address the mechanism or the need for capacity design. As a result, it is possible to get the retrofit on the left. This situation could occur by over-sizing the BRB frames and driving brittle nonlinear actions into the upper structure.

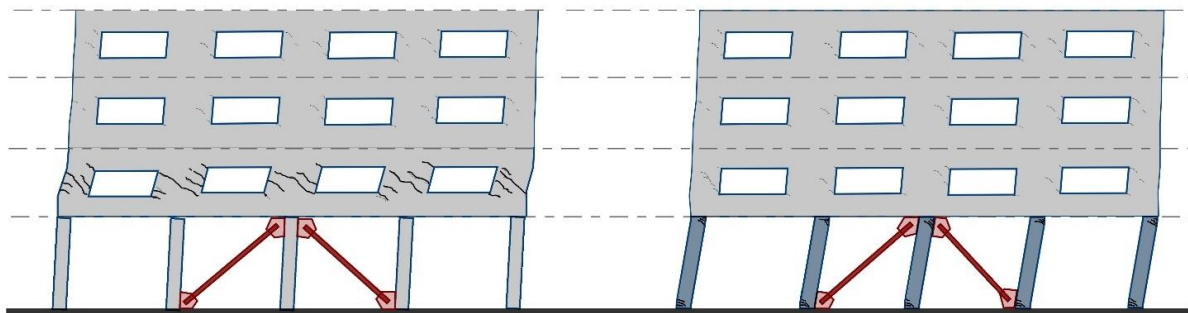


Figure D- 1: The cartoon shows two possible retrofit results, when using the Tier 3, Linear Dynamic Procedures of ASCE 41. The behavior-based design method gives the engineer the means to select the mechanism and better control the nonlinear behavior.

P5-Appendix E: Process to Design Rocking Foundation Hinges

As noted in Step 2, the shallow foundations designed in this process are to be either fixed or elastically rocking (see Figure E-1). For the fixed state, the foundation shall have sufficient strength to ensure a plastic hinge of the supported wall, frame, or column. Capacity design is to be applied, using expected material properties for the supported element and the soil.

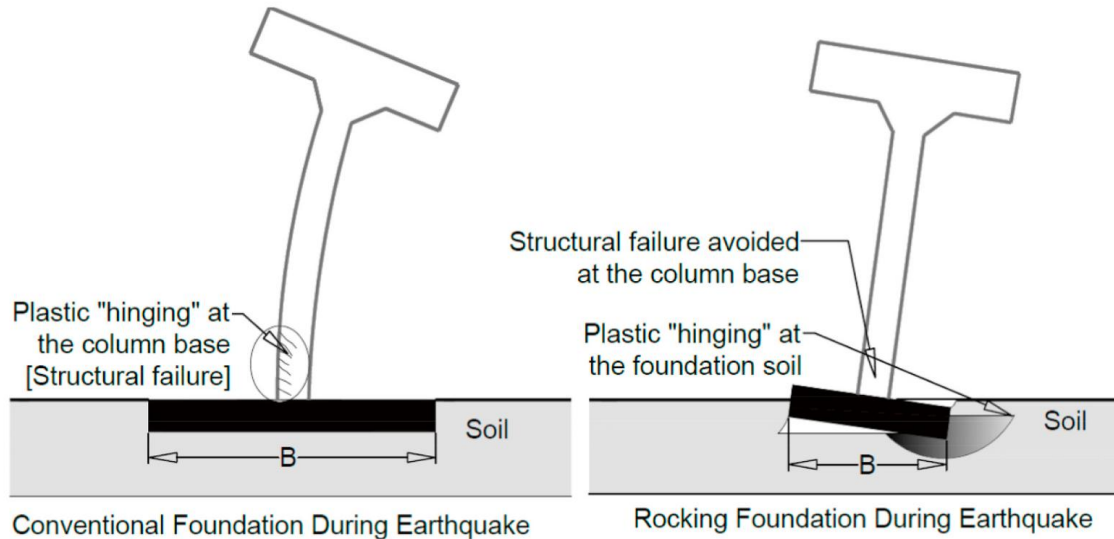


Figure E- 1: The image from Gajan, Soundararajan, Yang, and Akchurin, shows the two idealized states of shallow foundations, being considered for this process. On the left, the foundation is strong enough to support a plastic hinge in the superstructure. On the right, the foundation rocks. See reference 25.

The second possible state is that the foundation is allowed to rock, creating a nonlinear near-elastic hinge. This is the state shown in specimen (d) SSG03-03 of Figure E-2. The near-elastic rocking is achieved by keeping the bearing pressure low. Note that $A/A_c=12.3$, where A_c is the contact area during rocking and A is the foundation area. This occurs because specimen (d) has low axial load and high soil bearing capacity. The data from the shake table test shown in Figure E-3 also shows the desired rocking hinge. Note the re-centering of the specimen and the lack of significant settlement. If particular soils are unable to support high bearing pressures, it is possible to achieve rocking with localized soil improvement, as well as with providing additional concrete footing below the rocking footing. The supplemental bearing footing functions to reduce the high bearing pressures under the rocking footing.

It is possible to develop a plastic hinge with soil yielding, as shown in specimens (a-c) in Figure E-2 below. Although soil yielding dissipates energy, yielding soil hinges may be problematic for settlement, residual drift, and reparability. Note the correlation between the fullness of the hysteretic loops and the settlement. The use of plastic hinges with soil yielding is beyond the limits of this process.

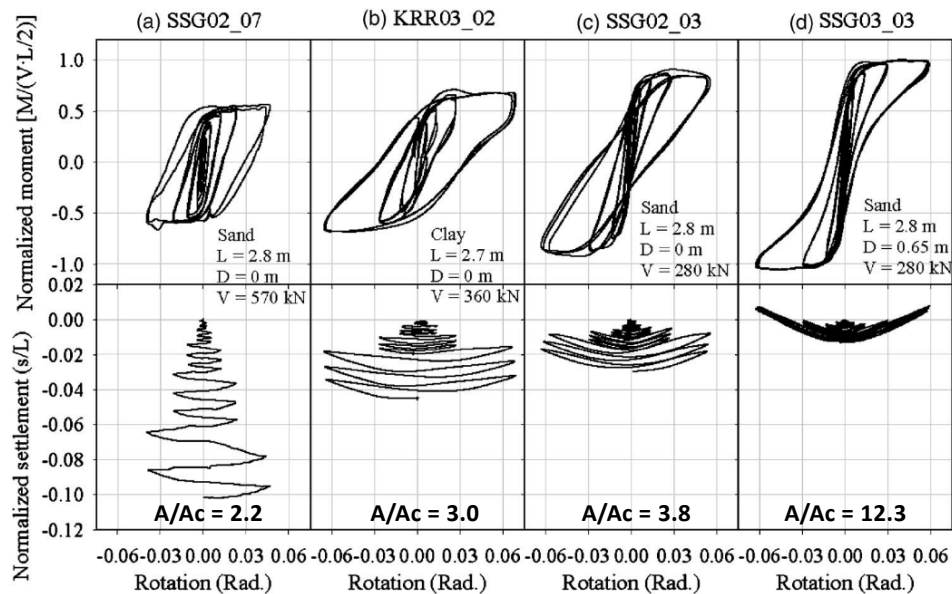


Figure E- 2: The image from Gajan and Kutter, shows the capacity, settlement, and energy dissipation of shallow foundations subjected to rocking. Specimen (d) the right, has the desired behavior for a near-elastic rocking hinge, due to the low bearing pressure and rocking contact area. See reference 24.

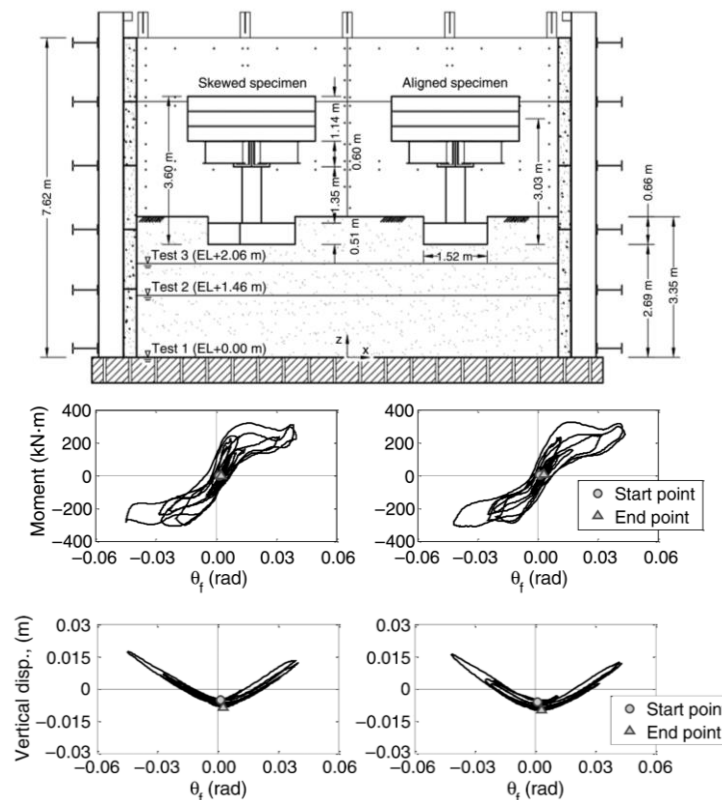


Figure E- 3: The images from Antonellis, Garvras, Panagiotou, Kutter, Guerrini, Sander, and Fox, shows the near-elastic nonlinear rocking response of bridge structures under shake table loading.