NEHRP Recommended Seismic Provisions for New Buildings and Other Structures

Volume II: Part 3 Resource Papers

FEMA P-1050-2/2015 Edition

FEMA

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NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provisions

for New Buildings and Other Structures (FEMA P-1050-2)

2015 Edition

Volume II: Part 3 Resource Papers

Prepared for the Federal Emergency Management Agency of the
U.S. Department of Homeland Security
By the Building Seismic Safety Council of the
National Institute of Building Sciences

BUILDING SEISMIC SAFETY COUNCIL
A council of the National Institute of Building Sciences
Washington, D.C.
2015

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

The BSSC is an independent, voluntary membership body representing a wide variety of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings.

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For further information on Building Seismic Safety Council activities and products, see the Council's website (www.bssconline.org) or contact the Building Seismic Safety Council, National Institute of Building Sciences, 1090 Vermont, Avenue, N.W., Suite 700, Washington, D.C. 20005; phone 202-289-7800; fax 202-289-1092; e-mail bssc@nibs.org.

Copies of this report on CD Rom may be obtained from the FEMA Publication Distribution Facility at 1-800-480-2520. Limited paper copies also will be available. The report can also be downloaded in pdf form from the BSSC website at www.bssconline.org .

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INTRODUCTION

The 2015 edition of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* is a new knowledge-based resource of technologies and procedures for improving seismic design and building practices in the nation. Starting with the 2009 edition, the Provisions began to focus on serving as a resource document aimed at translating research into practice. In this process, the earlier practice of containing a full set of seismic design requirements was eliminated. This approach is continued with the 2015 Provisions. The new changes in the 2015 NEHRP Provisions are based-on extensive results and findings from research projects, problem-focused studies, and post-earthquake investigation reports conducted by various professional organizations, research institutes, universities, material industries and NEHRP agencies.

Consistent with the approach used in the 2009 edition, the national standard ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures, Chapters 11-23, including Supplement No. 1 and the Expanded Commentary, has been adopted by reference for the Provisions. Modifications and additions to the Standard that passed BSSC's evaluation and consensus approval process appear in Part 1 of the Provisions. These recommended changes are intended for consideration and adoption in the next edition of ASCE/SEI 7. Each proposed Part 1 change is accompanied by a corresponding change to the ASCE 7-10 Commentary, which is contained in Part 2 of the Provisions. Parts 1 and 2 together with the adopted chapters of ASCE/SEI 7-10 and the references cited therein constitute Volume 1 the 2015 Provisions. Part 3 of the Provisions presents Resource Papers in a separate Volume 2.

Work on the 2015 Provisions began in October 2009 when the National Institute of Building Sciences, the BSSC's parent organization, entered into a contract with FEMA for initiation of the 2015 Provisions update effort. In consideration of balancing geographical and design practices, providing expertise in a broad range of subject areas, focusing on key areas of code improvement, and collaborating with national standards and building codes, 21 individual experts were selected to serve on the 2015 Provisions Update Committee (PUC). The PUC, with input from the earthquake engineering community, identified technical issues considered most critical for improvement of the U.S. seismic design practice, and formed Issue Teams for developing change proposals to the ASCE Standard. The following topics were investigated in the 2015 Provisions cycle: incorporation of P-695/P-795 methodologies for qualification of new systems and components; evaluation of performance objectives for seismic design and re-evaluation of seismic design categories; anchorage to concrete based-on ACI 318 Appendix D; nonlinear response history analysis; diaphragm issues; foundations on liquefiable soil and other site-related issues; soil amplification factors; triggers for site-specific spectra, design mapping issues based-on the U.S. Geological Survey's 2014 national seismic hazard maps; base isolation, energy dissipation systems; soil-structure interaction, and modal response spectrum analysis.

Between March 2010 and February 2015, the Issue Teams, members of the PUC, and the BSSC's Simplified Seismic Design Project developed 47 change proposals that were evaluated by the PUC in seven ballots, and subsequently evaluated by the Membership Organization representatives in four ballots. The consensus approved proposals from these four ballots were accepted by the BSSC Board of Direction for incorporation into the 2015 Provisions. The 2015 Provisions include extensive new changes, affecting significant parts of the seismic design sections in ASCE 7-10, including replacing four entire chapters.

All changes in Parts 1 and 2 of the Provisions are submitted to the ASCE/SEI 7 Standard committee for consideration of adoption. With some further improvements on the code language, most of these new changes are expected to be accepted in ASCE/SEI 7-16. The Standard is expected to be adopted by reference in International Building Codes (IBC) 2018.

The 2015 Provisions are divided into two volumes. For Part 1 of the Provisions in Volume 1, its Table of Contents lists only those sections and subsections of ASCE/SEI 7-10 that have been changed by approved proposals of the Provisions. For Part 2 in Volume 1 and Part 3 in Volume 2, the Table of Contents lists all chapters and up-to the fourth level of subsection headings.

A separate companion Provisions CD includes proposed maps for ASCE/SEI 7-16, IBC 2018 and IRC 2018 and issues and research recommendations for developing the 2020 Provisions.

Part 3 is a collection of resource papers that introduce new procedures or provisions not currently contained in the referenced standards for consideration and experimental use by the design community, researchers, and standards- and code-development organizations. Part 3 also represents Issue Team efforts on substantive proposals for topics that require further consideration by the seismic design community and additional research before being submitted to the BSSC membership for consensus approval for Parts 1 and 2 in the 2020 Provisions. Part 3 provides useful guidance on the application of Part 1 requirements, either as a discussion of an overall approach or as a detailed procedure and clarify some aspects of the Provisions requirements in Part 2. Part 3 consists of the following resource papers:

- Resource Paper 1, New Performance Basis for the Provisions
- Resource Paper 2, Diaphragm Design Force Level
- Resource Paper 3, Diaphragm Design: Current Practice, Past Performance and Future Improvements
- Resource Paper 4, Updated Maximum-Response Scale Factors
- Resource Paper 5, One-Story, Flexible Diaphragm Buildings with Stiff Vertical Elements

Specifically, the five resource papers include:

- Proposals for code and standard changes reflecting new and innovative concepts or technologies that are judged, at the time of publication of this edition of the Provisions, to require additional exposure to those who use codes and standards, and to possibly require systematic trial use. Some of these potential future changes are formatted for direct adoption while others discuss only the thrust of the proposed change.
- Discussions of topics that historically have been difficult to adequately codify. These papers provide background information intended to stimulate further discussion and research and, eventually, code change proposals.

Resource Papers 2, 3 and 4 also contain further proposed modifications to Parts 1 and 2 of the Provisions.

Feedback on the resource papers is encouraged. Comments and questions about the topics treated in these Part 3 resource papers should be addressed to:

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RESOURCE PAPER 1
NEW PERFORMANCE BASIS FOR THE PROVISIONS

RP1-1 ABSTRACT

This proposal provides background on performance based earthquake design and then proposes a potential new basis for Performance Goals in the Provisions. The proposed performance goals extend the risk targeted performance targets that was first introduced in the 2009 Provisions for Risk Category I and II structures to Risk Category III and IV structures, proposes probabilistic function loss targets for Risk Category IV structures, and proposes probabilistic performance targets for nonstructural components.

RP1-2 HISTORY OF PERFORMANCE-BASED DESIGN

Concepts of modern earthquake design provisions and their relation to performance expectations, stated and implied, have evolved over several decades. In the current provisions, buildings are assigned to Risk Categories1 based upon the consequence of their failure. A set of Seismic Design Categories is defined by combining those Risk Categories with the amplitude of design ground motion at the site. Key design provisions including those addressing required lateral strength and drift limits vary by Seismic Design Category. While the Provisions define four Risk Categories, there is only consideration of three unique ones since the first two – unoccupied structures and ordinary occupancy structures – have common seismic design criteria. There are additional provisions and higher design forces for most school buildings, structures housing large numbers of people, and other important but non-essential structures. Finally, there are a host of additional design requirements and even higher design forces for essential facilities, such as hospitals, fire stations, and emergency operations centers. This approach is assumed to achieve a number of performance objectives, some of which are stated in terms of probabilities associated with specific scenarios, as provided in C.1.3 and specifically in Tables C.1.3.1a and C.1.3.1b of ASCE 7.

The earliest performance intention of U.S. earthquake design provisions was to protect against building collapse. Following the earthquake and fire of 1906, San Francisco imposed a lateral force requirement of 30 pounds per square foot on building design with the expectation, based on empirical observations, that this would lead to adequate protection against earthquake collapse. At about this same time, several countries including Japan and Italy required design for lateral forces computed as a fraction (10%) of the building’s weight. In the U.S., lateral force requirements computed based on an assumed lateral acceleration did not enter the code until 1927, in response to the Santa Barbara earthquake of 1925. The California State Chamber of Commerce saw the need for a building code “dedicated to the safeguarding of buildings against earthquake disaster”. No distinction was made in performance objectives based on occupancy until the passage of the Field Act in California in 1933 in response to the Long Beach earthquake, whereby oversight of school construction was vested with California’s Division of Architecture. At that time it was assumed that a lateral force requirement of 2% to 10% of the building’s weight, depending on foundation conditions, was sufficient to preclude collapse.

In 1943, Los Angeles implemented provisions that recognized the effect of building flexibility on lateral response, and in 1948 the Joint Committee on Lateral Forces of the San Francisco section of ASCE was established to develop lateral force provisions that prescribed coefficients corresponding to the building fundamental period. In 1957, the Seismology Committee of the Structural Engineers of California began work on recommended changes to the Uniform Building Code for California and other

[1] In previous editions of the Provisions, Risk Categories were called Occupancy Categories, however the change was made based on changes that were made in ASCE 7-10. Refer to the commentary of Chapter 1 for discussion of the term Risk Category.

regions affected by earthquakes, and concepts of equivalent static force procedures still used today were developed at that time. Following the 1971 San Fernando Earthquake and the poor performance of some hospitals, there was recognition that certain types of facilities should be designed to preserve their pre-earthquake functionality following a major earthquake.

The commentary of the 4th Edition of the SEAOC Recommended Lateral Force Requirements, published in 1974, contains general language on performance, noting that the provisions should result in structures that resist minor earthquakes without damage, moderate earthquakes without structural damage but some damage to nonstructural components, major earthquakes with substantial structural and nonstructural damage and the most severe earthquakes ever anticipated to occur without collapse. The Commentary notes that the primary function of building codes is to provide minimum standards to assure public safety, and that the SEAOC provisions were intended to "provide criteria to fulfill life safety concepts". It was also observed that certain seismic provisions, such as those regulating school construction in California, were intended to minimize property damage in addition to protecting occupants. (SEAOC Blue Book – Fourth Edition) Major, moderate, minor and most severe earthquake ever anticipated were never quantitatively defined. Seismic designs were carried out to provide a minimum lateral strength and compliance with system-specific prescriptive detailing provisions. Early codes, which were based on the "Blue Book," addressed only structural design, but later, the codes specified minimum forces for anchorage and bracing of nonstructural components. Higher performance for important buildings was achieved first by requiring more stringent detailing provisions, by requiring higher design forces, and mandating more stringent quality assurance.

In the early 1970's the concept of probabilistic ground motion mapping was developed. The level of ground shaking hazard associated with the design requirements was determined to be roughly consistent with a 10% chance of being exceeded in 50 years (approximately a 475-year return period). Maps in ATC 3-06 (1978) and early versions of the NEHRP Provisions (1985) were based on that hazard, which provided a roughly equivalent seismic design base shear to that in the building codes at the time. However, the ATC 3-06 maps were adjusted in a major way from the first generation probabilistic map (475-year return period) produced by USGS. The USGS map had a contour at 0.6 g and a footnote indicating that along the San Andreas and Garlock fault system within that contour the peak value of PGA would be 0.8 g. The ATC map cut the top value down to 0.4 g and then pushed all the contours out further from the seismic sources, as well as raising the floor in the low spots. Overall, the total seismic hazard on the ATC 3 map was more than on the USGS 475-year map, but in areas of highest seismic hazard it was considerably less.

In the mid-1990s, two projects conducted at about the same time, the ATC-33 project which led to development of the FEMA 273/274 (1997) rehabilitation guidelines and the SEAOC *Vision 2000* (1995) publication made major advances in the definition of the expected performance of buildings designed to conform to the building code. Both projects defined performance in terms of discrete levels termed: Fully Operational (FEMA 273 defined Operational), Operational (FEMA 273 defined Immediate Occupancy), Life Safe and Near Collapse (FEMA 273 defined Collapse Prevention), each representing progressively more severe levels of damage from negligible to near total. The *Vision 2000* publication included Figure 1 which linked the discrete performance levels to quantitative seismic hazard levels at which they should be achieved for structures of different occupancy categories. The figure indicated that the design requirements for ordinary occupancy buildings would provide "Life Safe" performance at the "Rare" 475-year earthquake ground motion which by then had been defined in the building codes as the design earthquake (DE). The figure indicated code-conforming buildings would provide "Near Collapse" performance for "Very Rare" earthquake ground motions, then defined as having a return period of 970 years. Essential facilities were assumed capable of providing "Fully Operational" performance for 475-year earthquake ground motion.

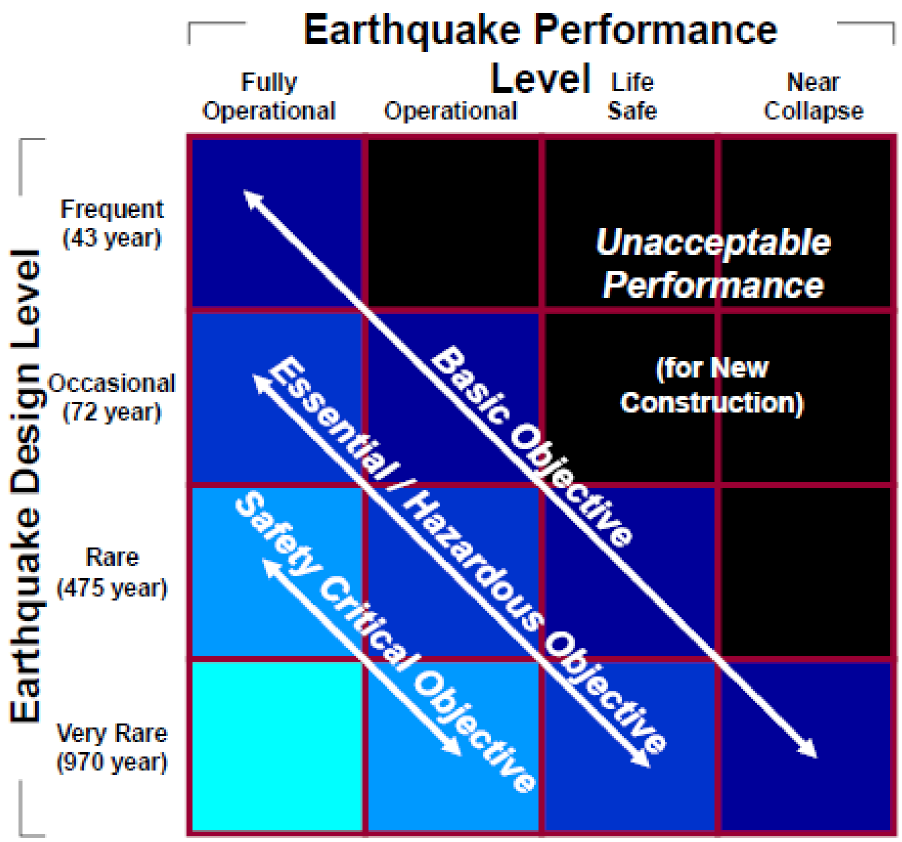

FIGURE 1 SEAOC Vision 2000 Performance Matrix

The 1997 NEHRP Provisions (FEMA 302/303) updated the older concept of avoiding collapse for the most severe earthquake intensities that had ever been experienced by setting a target of collapse prevention performance, similar to Near Collapse, for rare shaking termed Maximum Considered Earthquake (MCE) shaking. MCE shaking was defined as having a 2% probability of exceedance in 50 years (2,475-year return period), instead of the 975-year return period earthquake recommended in SEAOC *Vision 2000*. The design response parameters were not adjusted upward to correspond to the decision to target collapse in the extreme event as the design goal. Instead design shaking was taken as 2/3 of the MCE levels based on belief that well-designed structures would incorporate a margin against collapse of at least 1.5.

Maximum Considered Earthquake shaking was defined as having a 2%-50 year exceedance probability because it was desired to capture hazards associated with very large, very rare earthquakes on known faults outside of California, such as the New Madrid seismic source near Memphis and the Wasatch Fault near Salt Lake City. It was recognized that the 10% in 50 year exceedance probability would not be sufficient to capture such events. In part, 2% - 50 years was selected because USGS had already produced maps for this hazard level, and it was agreed that this would capture the very rare events described above. However, in regions near major active faults, MCE shaking was capped at 150% of the mean deterministic hazard at the specific site. This cap only applied where two-thirds of the 2% in 50-year shaking exceeded values that would produce forces equivalent to the highest design forces in the 1994 edition of the Provisions (the 1994 UBC gave very similar maximum values).

The Commentary of the 1997 NEHRP Provisions provided an updated version of the *Vision 2000* performance, shown in Figure 2, adopting the FEMA 273/274 performance level definitions and the newly defined earthquake hazard levels. In this graphic, "Operational" replaces the term "Fully Operational" from *Vision 2000* and "Immediate Occupancy" replaces "Operational."

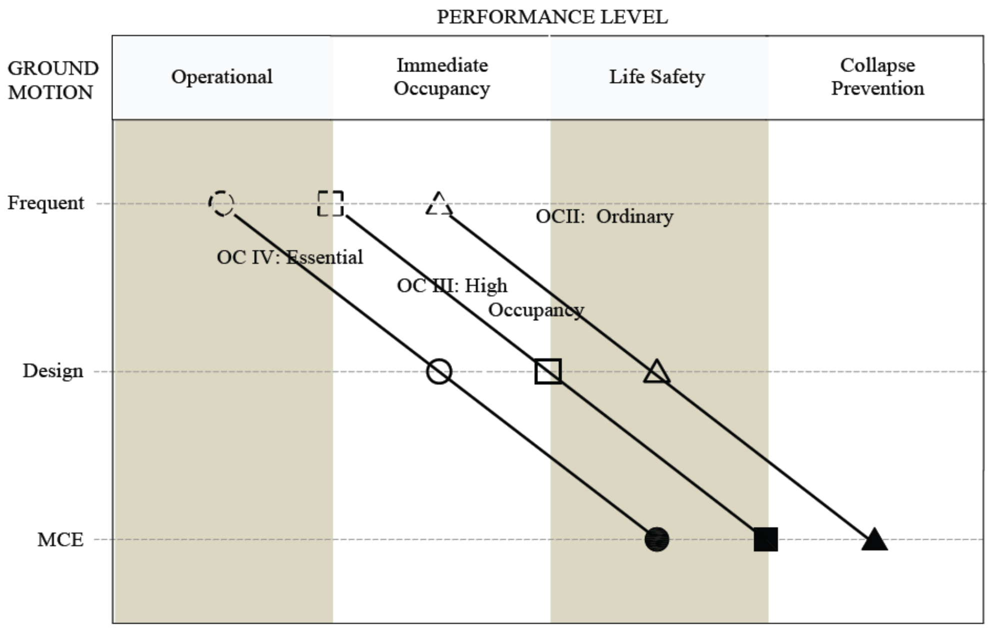

FIGURE 2 1997 NEHRP Performance Matrix

The next major update to seismic performance definitions came in the 2009 NEHRP Provisions with the introduction of risk targeted seismic hazard parameters. Several FEMA-funded projects (FEMA 350, 2000 and FEMA P-695, 2009) had introduced the concepts that seismic performance could not be predicted absolutely, but rather that there is considerable uncertainty associated with the prediction of performance. These uncertainties relate to variability in the intensity and character of ground shaking, imprecision in the design requirements, as-constructed quality, and other factors. The FEMA 350 document produced by the SAC Joint Venture targeted performance at 90% confidence of less than a 10% chance of collapse, conditioned on the occurrence of MCE shaking. Later, the FEMA P-695 document combined uncertain and random behaviors to define code performance as simply a 10% chance of collapse given MCE shaking.

FEMA P-695 provides a rational basis for determining global seismic performance factors, including the response modification coefficient (the R factor), the system overstrength factor (Ω_0) and the deflection amplification factor (C_d) that, when properly implemented in the seismic design process, will result in "equivalent safety against collapse in an earthquake, comparable to the inherent safety against collapse intended by current seismic codes, for buildings with different seismic-force-resisting systems," (FEMA 2009). The primary acceptance criterion of FEMA P-695 is that the seismic-force-resisting system be shown to have not more than a 10 percent probability of collapse conditioned on MCE_R ground motions (i.e., conditional collapse criterion subsequently adopted by 2009 Provisions). The Methodology is intended for use with model building codes and standards to set minimum acceptable design criteria for code-approved seismic-force-resisting systems when linear design methods are applied. It also provides a basis for evaluation of current code-approved systems and their ability to meet the seismic performance intent of the code.

Figure 3 shows the collapse probability results of FEMA P-695 evaluations of ten commonly used seismic-force-resisting systems. The significant overall observation is that there is a common period-dependent trend in the probability of collapse regardless of the type of system suggesting that the R factor and other aspects of seismic design methods could be improved. This trend includes the observation of higher collapse probability for short-period systems, relatively low collapse probability in the mid-period range, and potentially higher collapse probabilities for systems with very long periods. The relatively high collapse probabilities of short-period systems is believed to be largely due to limitations of current analytical methods used to model these buildings, since low-rise buildings have not shown a propensity for collapse in recent United States earthquakes. In general, the results of FEMA P-695 evaluations shown in Figure 3 support the life safety objective adopted by the 2009 Provisions of not more than a 10 percent probability of collapse conditioned on MCE_R ground motions, with the exception of building with very short periods. Because past earthquake experience does not correlate with this apparent short period collapse issue it was decided not to create any specific procedures for short-period structures and to further research the issue.

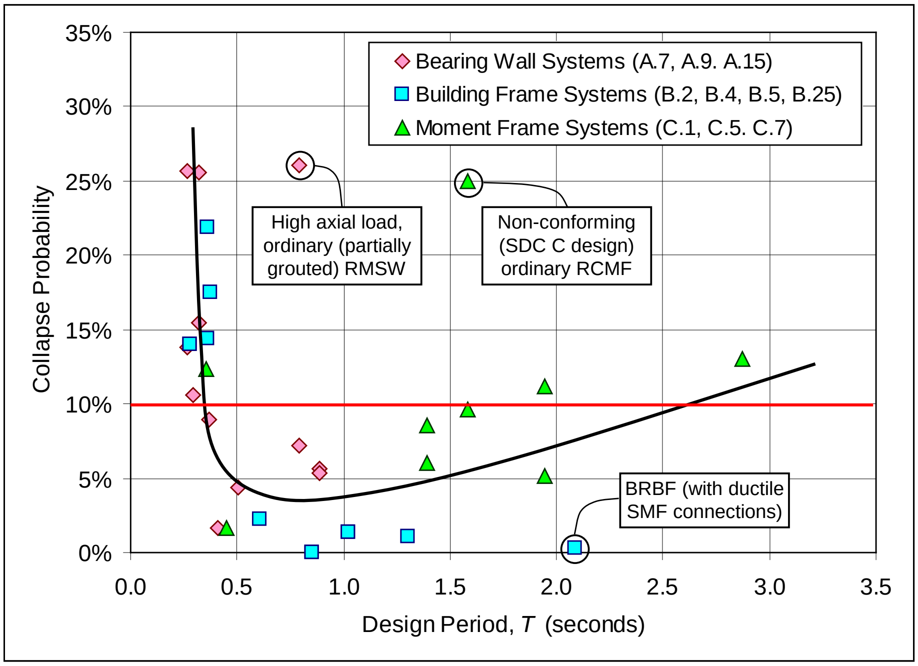

FIGURE 3 Plot showing the trend (and three circled outliers) in the probability of collapse of selected systems as a function of design period MCE_R ground motions. Data shown in this figure are based on results of prior studies of FEMA P-695 and NIST GCR 10-917-8 (copy of Figure 2-1 of NIST GCR 12-917-20)

The 2009 Provisions adopted the FEMA P-695 concepts in regions of the U.S. in which MCE shaking was probabilistically defined, selected the MCE motions such that when the site hazard is convolved with a standard structural fragility having a 10% collapse probability at MCE shaking and a dispersion of 0.6, a 1% probability of collapse in 50 years would result. This resulted in MCE shaking having different exceedance probabilities, generally slightly in excess of 2% in 50 years, throughout the U.S. The resulting MCE motions were named Risk-Targeted Maximum Considered Shaking, denoted by the symbol MCE_R. In areas of the U.S. close to major active faults, a deterministic definition of MCE_R shaking continued to be used, but changed to be shaking having an intensity one standard deviation above the mean motion resulting from a characteristic earthquake on the proximate major fault.

Part of the reason that deterministic limits were imposed is that the Provisions Update Committee believed the probabilistic analysis had flaws that cannot be corrected with our current state of knowledge. In other words, the higher probability of collapse that is computed and accepted is not entirely based upon an assumption that a higher risk is acceptable where the public is relatively well-informed about the earthquake issue; a very real part of that acceptance is the belief the computed probabilistic numbers are not realistic near active faults. In ATC 3-06, the capping was excused with a concept of "effective peak acceleration", which allowed engineering judgment to temper the ground motions predicted by seismologists. Today, the decisions on how to perform a deterministic analysis are still rooted in engineering judgment.

The 2009 Provisions only addressed target probabilities of collapse for normal occupancy structures and was silent on the probability of collapse associated with structures in higher occupancy categories. However, the commentary of the general provisions chapter of ASCE 7-10 did provide the estimated anticipated probabilities of collapse given MCE_R shaking for higher occupancy (risk) categories for structures designed to satisfy the seismic requirements of ASCE 7-10. These were defined respectively as 6% and 3% conditional probabilities of collapse for Risk Categories III and IV. These probabilities were determined by adjusting the standard fragility having a 10% probability of collapse at MCE_R shaking and a dispersion of 0.6, with the Occupancy Importance Factors specified by the Provisions for these categories. These values are expressed in the Table 1 below, reproduced from the commentary of ASCE 7-10.

Table 1 ASCE 7-10 Table C.1.3.1b Anticipated Reliability (Maximum Probability of Failure) for Earthquake

Risk Category I and II	
Total or partial structural collapse	10% conditioned on the occurrence of Maximum Considered Earthquake shaking
Failure that could result in endangerment of individual lives	25% conditioned on the occurrence of Maximum Considered effects
Risk Category III	
Total or partial collapse	6% conditioned on the occurrence of Maximum Considered Earthquake shaking
Risk Category IV	
Total or partial collapse	3% conditioned on the occurrence of Maximum Considered Earthquake shaking
Failure that could result in endangerment of individual lives	10% conditioned on the occurrence of Maximum Considered Earthquake shaking

[1] Refer to the NEHRP Recommended Provisions Seismic Regulations for Buildings and Other Structures, FEMA P-750 for discussion of the basis of seismic reliabilities.

In Table 1 probabilities of failure are expressed conditionally based on the occurrence of MCE shaking. Only structural performance is categorized. Nonstructural performance is not addressed in Table 1 but implied as failure that could result in endangerment of individual lives. Consideration of nonstructural performance solely as a conditional probability against endangerment to life is in conflict with commentary to the NEHRP Provisions. This commentary indicates that the performance of designated seismic systems (nonstructural components that are assigned an $I_p = 1.5$), such as equipment in

Risk Category IV buildings, are to be functional following design earthquake shaking. Additionally, commentary explains the intent to "minimize structural and nonstructural repair costs whenever practicable to do so." There are a number of nonstructural provisions contained within the code requiring restraint of components which do not pose a life safety hazard, because the restraint is deemed something that can be done at low cost and provide a reduction in post-earthquake damage. In part, probabilistic criteria associated with loss of function of associated nonstructural components were not placed in the commentary because a consensus could not be formed as to what the appropriate target reliabilities were or how to verify them. This remains an issue in the present edition of the Provisions.

Additionally, there are other statements in the 2009 NEHRP Provisions commentary that characterize Risk Category IV structures as being intended to maintain function / operation following design earthquake (DE) shaking, expressed as two-thirds of the MCE_R. The ASCE 41-13 Standard, the successor document to FEMA 273, provides a table mapping Risk Category performance to ASCE 41/FEMA 273 Performance Objective(s) shown in Table 2 below in an attempt to further define the performance intentions, albeit in the deterministic performance level vernacular of that standard. The reason that was done in ASCE 41-13 is that engineers are occasionally charged to provide an evaluation or retrofit that provides equivalent performance with a new Risk Category IV building designed to the Provisions.

Table 2 ASCE 41-13 Basic Performance Objective Equivalent to New Building Standards (BPON)

Risk Category	Seismic Hazard Level BSE-1N (DE)	Seismic Hazard Level BSE-2N (MCE_R)
I & II	Life Safety Structural Performance; Position Retention Nonstructural Performance (3-B)	Collapse Prevention Structural Performance; Nonstructural Performance Not Considered (5-D)
III	Damage Control Structural Performance; Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance; Nonstructural Performance Not Considered (4-D)
IV	Immediate Occupancy Structural Performance; Operational Nonstructural Performance (1-A)	Life Safety Structural Performance; Nonstructural Performance Not Considered (3-D)

While the definition of performance of structures assigned to various Risk Categories using the ASCE 41/FEMA 273 vernacular is conceptually similar to the Provisions, there are many issues between them which prevent a declaration of equivalency. The first is that the ASCE 41/FEMA 273 performance objectives are deterministic, while the Provisions performance goals are probabilistic. ASCE 41 states the performance to be "Collapse Prevention" in the BSE-2N (MCE_R) shaking, without any indication of confidence in achieving that objective. Conversely the Provisions declare a 10% probability of collapse in the MCE_R, meaning there is a 90% probability of achieving collapse prevention. Second, few studies have been done to correlate the criteria in ASCE 41 to achieve the performance objective with the probabilities provided by buildings designed by the provisions.

One point that can be gleaned from the ASCE 41 standard's mapping effort is that the performance level targets for Risk Category IV structures cannot be met solely by designing for a conditional collapse probability at the MCE_R hazard level. There is a need to define the target for loss of function. Function protection in context of earthquake design means that a facility will not have sustained structural damage or damage to the nonstructural systems that would prevent restoration of pre-earthquake function following the design earthquake. The Provisions discuss this in its commentary and contain many prescriptive requirements, particularly for nonstructural components that are intended to provide function protection.

RP1-3 FUTURE PERFORMANCE-BASED PROVISIONS

NIST GCR 12-917-30 (ATC, 2012) provided a method to extend the risk targeted methodology to function loss and economic loss. The report proposed absolute and conditional risk targets for function and economic loss, as shown in Table 3. That report used Risk Category II buildings as the anchor point for the functional and economic performance targets in addition to the collapse preference target and defined the performance targets for higher risk categories by adjusting the conditional probability of the performance target being exceeded.

Table 3 Hypothetical Seismic Performance Objectives, Ground Motion Intensities, and Design Factors Illustrating Risk-Based Framework for Future Editions of ASCE 7 (from Table 4-6, NIST 2012)

Generic Risk Subject	Seismic Performance Objectives				Primary Design Parameter[2]		Basis for Values of Design Parameters
	Facility Performance	Probability		Ground Motion Intensity	Structural	Non-Structural	
		50-Year Risk	Conditional (on GM)				
Risk Category I (structures posing low risk to life safety)							
Life Safety	No Collapse	2	20	MCE_R	(R_M/I_e)	(R_M/I_p)	(FEMA P-695)
Economic	None						
Function	None						
Risk Category II (structures not in Risk Categories I, III or IV)[1]							
Life Safety	No Collapse	1	10	MCE_R	R_M/I_e	R_M/I_p	FEMA P-695
Function[3]	Green Tag	5	10	FLE_R			
Economic[4]	10%LR	50	10	SLE_R			
Risk Category III (posing high risk to life safety, economic impact and/or disruption)							
Life Safety	No Collapse	≈ 0.5	5	MCE_R	R_M/I_e	R_M/I_p	(FEMA P-695)
Function[3]	Green Tag	≈ 10	5	FLE_R			
Economic	10%LR	≈ 20	5	SLE_R	Drift[5]	TBD[6]	Hazus/ATC-58[7]
Risk Category IV (essential facilities required to maintain functionality)							
Life Safety	No Collapse	≈ 0.25	2.5	MCE_R	R_M/I_e	R_M/I_p	(FEMA P-695)
Function	Green Tag	≈ 5	2.5	FLE_R	Drift[5]	TBD[6]	Hazus/ATC-58[7]
Economic[4]	10%LR	≈ 10	2.5	SLE_R			

1. ASCE/SEI 7-10 defines probabilistic MCE_R ground motions in terms of (1) a 1 percent probability of collapse in 50 years and (2) a 10 percent probability of collapse given these ground motions occur. The 10 percent conditional collapse probability is based on design of the seismic force resisting system using R (I_e = 1.0) and other design parameters that are consistent with FEMA P-695 acceptance criteria appropriate for Category II structures .

2. Other parameters and criteria include minimum value of the base shear coefficient, C_s, etc.

3. In this example, protection against loss of function is assumed to be provided implicitly by response limits associated with Life Safety criteria for MCE_R ground motions (Risk Category II structures) and by economic loss criteria for SLE_R ground motions (Risk Category III structures).

4. In this example, protection against unacceptable economic loss is assumed to be provided implicitly by response limits associated with Life Safety criteria for MCE_R ground motions (for Risk Category II structures) and by loss of function criteria for FLE_R ground motions (for Risk Category IV structures).

5. In general, current story drift limits of Table 12.12-1 of ASCE/SEI 7-10 do not provide adequate damage control to meet functional and/or economic loss objectives and would require substantial revision.

6. To Be Determined. Design of nonstructural systems to meet functional and economic performance objectives is complex, requiring consideration of structure response (e.g., calculation of in-structure response spectra), special qualification testing of certain components, etc.

7. Structural fragility data of the ATC-58 project (ATC 2011) or the *HAZUS-MH MR1 Advanced Engineering Building Module Technical and User's Manual* (FEMA 2003) may be used to define appropriate system-specific story drift limits for limiting damage to the structural system.

Using Table 3 from the NIST GCR 12-917-30 (ATC, 2012) report and the risk targeted collapse philosophy, Table 4 proposes a possible framework of target performance levels for the development of future editions of the Provisions. It preserves MCE_R shaking as the basis of design for collapse avoidance and probabilistic absolute collapse targets. As with previous editions of the provisions the risk targeting for collapse is done for ordinary structures – Risk Category II. The probability of collapse is reduced for high Risk Category structures and increased for Risk Category I. The reduction in probability of collapse matches with the increase in design forces typically provided for by the provisions. The reduction of collapse probability is made to convey an acceptance of a lower level of safety that might be implemented in the future for Risk Category I structures in a manner similar to how the design of those structures for other environmental hazards is done.

Table 4 introduces the concept of a Function-Level Earthquake (FLE_R) shaking and probabilistic function loss targets for Risk Categories II through IV conditioned on the occurrence of FLE_R shaking. Unlike Table 3, Table 4 set the absolute performance target for function based on a Risk Category IV structure, instead of a Risk Category II structure. This was done because there is a perception within the profession that current Risk Category IV provisions provide function protection for Design Earthquake shaking (currently defined at 2/3rds of MCE_R shaking). In spite of this belief, there has been no study to validate that. Table 4 suggests 2.5% absolute probability of collapse in 50 years and a conditional 10% probability of function loss in the FLE_R for Risk Category IV structures. These specific values are meant to illustrate the relative performance between the different Risk Categories and are not specifically intended to be hard targets. Significant study and likely additional provisions development is required to quantitatively define these performance states.

The risk targeting for the FLE_R is done for Risk Category IV because that is the Risk Category where function protection is commonly assumed to be provided. For Risk Category II and III structures, no conditional probability of function loss based on the FLE_R hazard is provided, indicating a greater probability that there would be loss of function in those facilities. That is not to say that there is not some earthquake level that function protection is assumed, however a function performance target is not proposed as a specific performance goal and may differ for different structural systems based on design and detailing requirements found in those material standards.

Table 5 is a proposed framework for nonstructural components performance targets. The table is broken up into two categories, in the same manner in which the Provisions currently delineate nonstructural components. For most components, the performance level can be described as position retention, whereby the component is restrained in its place to prevent falling hazards, but the internal workings of mechanical and electrical components may be damaged and the component may be unable to function following the design earthquake. Similarly, architectural components may not provide their intended function, such as maintaining weather protection. For essential life safety system components, components containing hazardous materials, and components found in essential facilities, where the provisions currently designate an $I_p = 1.5$, the performance level is operational, however some repairable damage may occur and the component may not return to function until after the earthquake. The components are expected to function following the design earthquake. Since the provisions currently require nonstructural component design at the design earthquake level, it was decided to use the FLE_R as the earthquake for nonstructural consideration.

While Table 5 tethers nonstructural performance to the MCE_R and the proposed FLE_R, those are not the only factors that affect nonstructural components. Structural response, specifically floor acceleration and inter-story, drift affect nonstructural performance. For a given MCE_R or FLE_R, the nonstructural performance may vary greatly based on the structural systems, structure’s dynamic characteristic and ductility.

Table 4 also shows an explicit separation of anticipated performance for Risk Category I and II structures. Although the Provisions have traditionally adopted identical design criteria for these two categories, the differentiation in performance shown here is consistent the performance goals for some other natural hazards adopted in ASCE-7.

Tables 4 and 5 include entries for economic protection which for Risk Category II through IV is simply filled with the term implicit. Implicit means that there will be some protection against damage which would cause economic loss. The reason these lines for economic loss are included at all is for consistency with the prescriptive requirements added to the Provisions and referenced standards to enable damage control. While NIST GCR 12-917-20 provided recommendations on economic loss performance targets, it is recommended that the Provisions should not specify explicit targets for economic loss.

Tables 6 and 7 compare the current Provisions Design Earthquake (DE) shaking of two-thirds of MCE_R with a 10% conditional probability of function loss at the FLE_R. The FLE_R shaking is based on a 5% probability of function loss in 50 years risk target for Risk Category IV facilities and, the resulting FLE_R is slightly less than the design earthquake hazard that is currently in the 2009 Provisions and the 475-year hazard which was previously used.

Tables 4 and 5 represent one proposed framework for the next generation performance-basis for the Provisions. However, an assessment of what probabilistic performance the current Provisions provide is necessitated before any new framework can be adopted. While "functionality" is presumed for Risk Category IV structures), there is not data on the level of functionality currently provisions provide and it is not known what various stakeholders will deem tolerable damage and still be "functional."

In order to extend the risk-targeted concepts to nonstructural components and systems, other Risk Categories, and other performance levels, a method must be developed that permits estimation of structural and nonstructural performance based on design requirements, shaking intensity, and the performance and functionality expectations of the different stakeholders. For example, damage that contributes to the likelihood of an unsafe placard following a given shaking intensity need to be identified and procedures developed to reduce the likelihood of occurrence. This list might include residual drift, damage to structural elements that compromise the performance of the structure in the inevitable aftershocks that follow a sizeable earthquake, and drift limits that are truly protective of nonstructural components. For nonstructural systems, the damage tolerance of occupants in different types of structures must be assessed, a framework is needed for determining what constitutes functionality following the earthquake, which will allow the designer to identify those components and systems that must function to provide the desired level of service, and finally development of design requirements that will deliver the desired performance. In the future, the FEMA P-58 methodology may provide a basis for estimating structural; and nonstructural performance, but substantial further development of the methodology is needed. At the present time available fragility functions are not sufficiently robust. Also the methodology does not presently deal well with the complex interaction between damage to different types of systems and components on building functionality. Further correlation and calibration of the collapse fragilities for structures and life loss and function fragilities for the large universe of nonstructural components is also needed.

Table 4 Possible Structural Performance Targets

Generic Risk Subject	Structural Risk Objective			
	Performance Objective	Risk Probability		Ground Motion Intensity
		Conditional (on GM)	Absolute (% in 50 years)	
Risk Category I (structures posing low exposure for life safety risk)				
Life Safety	No Collapse	20	~2	MCE_R
Economic Loss	Not considered			
Function Loss	Not considered			
Risk Category II (structures not in Risk Categories I, III or IV)[1]				
Life Safety	No Collapse	10	1	MCE_R
Economic Loss	Implicit			
Function Loss	Not considered			
Risk Category III (structures posing high risk to life safety, economic impact and/or disruption)				
Life Safety	No Collapse	5	<1	MCE_R
Economic Loss	Implicit			
Function Loss	Not considered			
Risk Category IV (essential facilities required to maintain functionality)				
Life Safety	No Collapse	2.5	<<1	MCE_R
Economic Loss	Implicit			
Function Loss	Operational	10	5	FLE_R

Table 5 Possible Nonstructural Performance Targets

Generic Risk Subject	Nonstructural Risk Objective			
	Performance Objective	Risk Probability		Ground Motion Intensity
		Conditional (on GM)	Absolute (% in 50 years)	
Position Retention, I_p = 1.0 (All other nonstructural components)				
Life Safety	No falling hazard and egress maintained	25	5	MCE_R
Economic Loss	Implicit			
Function Loss	Not considered			
Operational, I_p = 1.5 (Essential nonstructural components)				
Life Safety	No falling hazard and egress maintained	10	1	MCE_R
Economic Loss	Implicit			
Function Loss	Operational	10	5	FLE_R

Table 6 Proposed FLE_R 0.2 s spectral values and return periods compared to 2014 Provisions DE

		DE		FLE_R	
Region	City (Site Location)			FLE_R (*g*)	RP (yrs)
Southern California	Los Angeles	1.60		1.13	380
Southern California	Century City	1.44		1.03	375
Southern California	Northridge	1.13		1.00	358
Southern California	Long Beach	1.10		0.77	396
Southern California	Irvine	1.03		0.71	413
Southern California	Riverside	1.00		0.94	368
Southern California	San Bernardino	1.58		1.60	348
Southern California	San Luis Obispo	0.74		0.52	405
Southern California	San Diego	0.84		0.51	466
Southern California	Santa Barbara	1.89		1.29	382
Southern California	Ventura	1.59		1.11	382
Northern California	Oakland	1.24		1.45	346
Northern California	Concord	1.38		1.36	358
Northern California	Monterey	1.02		0.72	384
Northern California	Sacramento	0.45		0.33	400
Northern California	San Francisco	1.00		1.04	353
Northern California	San Mateo	1.23		1.11	371
Northern California	San Jose	1.00		1.15	357
Northern California	Santa Cruz	1.01		0.84	369
Northern California	Vallejo	1.00		1.00	353
Northern California	Santa Rosa	1.67		1.44	370
Pacific Northwest	Seattle	0.91		0.64	388
Pacific Northwest	Tacoma	0.86		0.63	375
Pacific Northwest	Everett	0.85		0.58	396
Pacific Northwest	Portland	0.65		0.43	396
Other WUS	Salt Lake City	1.03		0.58	478
Other WUS	Boise	0.21		0.14	407
Other WUS	Reno	1.00		0.73	368
Other WUS	Las Vegas	0.33		0.21	425
U.S.	St. Louis	0.29		0.18	430
U.S.	Memphis	0.67		0.36	488
U.S.	Charleston	0.77		0.36	533
U.S.	Chicago	0.09		0.05	444
U.S.	New York	0.19		0.09	492

Table 7 Proposed FLE_R 1.0 s spectral values and return periods compared to 2014 Provisions DE

		DE		FLE_R	
Region	City (Site Location)			FLE_R (*g*)	RP (yrs)
Southern California	Los Angeles	0.56		0.40	385
Southern California	Century City	0.54		0.38	393
Southern California	Northridge	0.40		0.36	367
Southern California	Long Beach	0.41		0.29	410
Southern California	Irvine	0.38		0.27	413
Southern California	Riverside	0.40		0.36	362
Southern California	San Bernardino	0.72		0.66	361
Southern California	San Luis Obispo	0.28		0.20	392
Southern California	San Diego	0.32		0.20	469
Southern California	Santa Barbara	0.66		0.45	389
Southern California	Ventura	0.60		0.41	392
Northern California	Oakland	0.50		0.53	355
Northern California	Concord	0.49		0.47	368
Northern California	Monterey	0.37		0.26	392
Northern California	Sacramento	0.20		0.15	385
Northern California	San Francisco	0.43		0.41	364
Northern California	San Mateo	0.57		0.45	387
Northern California	San Jose	0.40		0.39	355
Northern California	Santa Cruz	0.40		0.30	369
Northern California	Vallejo	0.40		0.36	359
Northern California	Santa Rosa	0.69		0.58	378
Pacific Northwest	Seattle	0.35		0.24	387
Pacific Northwest	Tacoma	0.34		0.24	380
Pacific Northwest	Everett	0.32		0.22	393
Pacific Northwest	Portland	0.28		0.18	408
Other WUS	Salt Lake City	0.37		0.20	495
Other WUS	Boise	0.07		0.05	391
Other WUS	Reno	0.34		0.24	384
Other WUS	Las Vegas	0.11		0.08	398
CEUS	St. Louis	0.11		0.06	458
CEUS	Memphis	0.23		0.12	517
CEUS	Charleston	0.24		0.10	569
CEUS	Chicago	0.04		0.03	425
CEUS	New York	0.05		0.03	454

RP1-4 REFERENCES

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Applied Technology Council (ATC). (2013). *Tentative Framework for Development of Advanced Seismic Design Criteria for New Buildings.* NIST GCR 12-917-20, prepared by the Applied Technology Council for the National Institute of Science and Technology, Gaithersburg, MD.

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Federal Emergency Management Agency (FEMA). (2000). *Recommended Seismic Design Criteria For Moment-Resisting Steel Frame Structures*. FEMA 350, prepared by the SEAOC, ATC, and CUREE (SAC) Joint Venture for the Federal Emergency Management Agency, Washington, DC.

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Structural Engineers Association of California (SEAOC). (1955). *Recommended Lateral Force Requirements*. Sacramento, California.

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RESOURCE PAPER 2
DIAPHRAGM DESIGN FORCE LEVEL

RP2-1 DIAPHRAGM DESIGN FORCE LEVEL

RP2-1.1 Add the following to the end of the Added Section 12.11.5 DIAPHRAGM DESIGN FORCE REDUCTION FACTOR

The diaphragm design force reduction factor, R_s, for steel deck diaphragms and wood sheathed diaphragms supported by cold-formed steel framing shall be determined in accordance with Table 12.11.5-2.

For diaphragm systems or design methods not listed in Table 12.11.5-1 or 12.11.5-2, R_s shall be taken equal to 1.0 or shall be justified by cyclic testing, rational analysis, and/or comparison of performance with types of diaphragms listed in Table 12.11.5-1 or 12.11.5-2.

Table 12.11.5-2 Diaphragm Design Force Reduction Factor, Rs

<table>
<tr><th colspan="2">Diaphragm System</th><th>Shear-Controlled</th><th>Flexure-Controlled</th></tr>
<tr><td>Untopped steel deck designed in accordance with AISI S100 or SDI RD</td><td>-</td><td>2.0</td><td>NA</td></tr>
<tr><td rowspan="2">Topped steel deck designed in accordance with AISI S100 or SDI C and SDI NC</td><td>Reinforced topped steel deck with shear stud connection to framing</td><td>2.0</td><td>2.5</td></tr>
<tr><td>Other topped steel deck with structural concrete fill</td><td>1.5</td><td>2.0</td></tr>
<tr><td>Wood sheathed designed in accordance with AISI S213</td><td>-</td><td>2.0</td><td>NA</td></tr>
</table>

RP2-1.2 Add the following references to Section 23.1 of Chapter 23 SEISMIC DESIGN REFERENCE DOCUMENTS

SDI

Steel Deck Institute

SDI C-2011

Standard for Composite Steel Floor Deck-Slabs

SDI NC-2010

Standard for Non-Composite Steel Floor

SDI RD-2010

Standard for Steel Roof Deck

RP2-1.3 Add a new Section C12.11.5 to the end of C12.11 STRUCTURAL WALLS AND THEIR ANCHORAGE

C12.11.5 Diaphragm Design Force Reduction Factor

C12.11.5.1 Steel Deck Diaphragms. Diaphragm design force reduction factors, R_s, have been assigned for untopped and topped steel deck diaphragms.

Untopped steel deck diaphragms are currently designed according to the SDI Diaphragm Design Manual (SDI, 2004), the Tri-Services Method (Tri-Services, 1988), AISI S100 (AISI, 2010) and AISI S213 (AISI, 2009), or SDI RD-2010 (SDI, 2010a). Current design methods and standards were developed from hundreds of quasi-static load tests of untopped steel deck diaphragms conducted by researchers and manufacturers over many years. Steel deck diaphragm design currently follows an elastic design approach. The behavior is predictable and well understood.

Topped steel deck diaphragms are designed in accordance with SDI Diaphragm Design Manual (SDI, 2004), the Tri-Services Method (Tri-Services, 1988), AISI S-100 (AISI, 2010) and AISI S-213 (AISI, 2009), SDI C-2011 (SDI, 2011), or SDI NC-2010 (SDI 2010b).

C12.11.5.2.1 Intended mechanism. Untopped steel deck diaphragms are primarily shear-controlled. Topped cold-formed steel deck diaphragms can be shear- or flexure-controlled.

C12.11.5.3.2 Derivation of diaphragm force-reduction factor. Untopped steel deck diaphragms have been tested at Ecole Polytechnique in Montreal, Canada by Robert Tremblay, Colin Rogers, et al. (Essa et al., 2002; Rogers & Tremblay, 2003a; Rogers & Tremblay 2003b) and by Hilti, Inc. (Engleder & Gould, 2010). The research involved cyclic, inelastic load testing of steel deck connections and diaphragm systems with welds, power-actuated fasteners and screws. Untopped steel deck diaphragms are connection-dependent and ductile behavior of the connections (local ductility) translates into ductile behavior of the diaphragm (global ductility). R_s-factors were assigned based on judgment formed partly by an examination of the research findings.

Topped steel deck diaphragms can be shear- or flexure-controlled. Topped cold-formed steel deck diaphragms may be connected with welded shear studs or mechanical shear connectors to framing. Other topped diaphragms are filled with structural concrete without shear connectors. Tests conducted to date have been quasi-static and were used in the formulation of the standards referenced earlier.

R_s-factors were based on judgment formed in part by examination of limited test data that the seismic performance of topped steel deck diaphragms without shear studs is likely to be comparable to that of cast-in-place reinforced concrete diaphragms, while the performance of topped steel deck diaphragms with shear studs is likely to be somewhat superior. This basis has been controversial within the Issue Team. There are members who strongly feel that the performance of topped metal deck diaphragms with shear studs is more likely to be comparable to that of cast-in-place reinforced concrete diaphragms.

C12.11.5.2 Wood Sheathed Diaphragms Supported by Cold-Formed Steel Framing

Table 12.11.5-2 includes an R_s value for wood sheathed diaphragms designed in accordance with AISI S213-07 with Supplement 1-09 (AISI, 2009).

C12.11.5.2.1 Intended mechanism. As with wood-sheathed diaphragms on wood light-frame construction, wood diaphragms on cold-formed steel light frame construction (CFSF) are shear-controlled, with design strength determined in accordance with AISI S213(AISI, 2009) and the shear behavior based on the sheathing-to-framing connections. CFS diaphragm chord members are unlikely to form flexural mechanisms (ductile or otherwise), due to the overstrength inherent in axially loaded members designed in accordance with the applicable standards.

C12.11.5.2.2 Derivation of diaphragm design force reduction factor. An R_s-factor of two is assigned in Table 12.11.5-2 based essentially on judgment. WSP on cold-formed steel framing and WSP on wood framing have similarities as well as differences in their responses to seismic excitation. More information on WSP on CFSF diaphragm behavior and further studies on performance would be ideal.

C12.11.5.3 Other Systems. Although R_s-values are provided in Tables 12.11.5-1 and 12.11.5-2 for most common diaphragm systems and design standards, there are a wide range of other systems and design methods that might be used. Among these are other diaphragm sheathing and fastening combinations, some included in current building codes and others approved through code provisions for alternate materials and methods of construction. In addition, there are alternate construction types that serve the function of diaphragms in some buildings and building systems, most notably horizontal truss systems.

The final paragraph of Section 12.11.5 sets the value of R_s at 1.0 unless specific consideration of the diaphragm force reduction factor is provided. This was judged to be the lowest value applicable to diaphragms, outside of extraordinary systems and circumstances. Where specific consideration is to be provided, this paragraph goes on to describe the characteristics required for a greater R_s-factor. Part 1 Commentary Section C12.11.5 has provided detailed discussion of these characteristics and available methods for defining the R_s-factor.

RP2-1.4 Add the following references to Chapter C12 REFERENCES:

AISI, 2010. North American Specification for the Design of Cold-Formed Steel Structural Members (AISI S100), American Iron and Steel Institute, Washington, D.C.

AISI, 2009 North American Standard for Cold-Formed Steel Framing - Lateral Design 2007 Edition with Supplement No 1. and Commentary (AISI S213), American Iron and Steel Institute, Washington, D.C.

Essa, H. S., Tremblay, R., and Rogers, C., 2002. Inelastic Seismic Response of Metal Roof Deck Diaphragms for Steel Building Structures, *12th European Conference on Earthquake Engineering,* Paper Reference 482, Elsevier Science Ltd., Kidlington, Oxfordshire, U.K.

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SDI, 2010b. *Standard for Non-Composite Steel Floor* (SDI NC-2010), Steel Deck Institute, Glenshaw, PA.

SDI, 2011. *Standard for Composite Steel Floor Deck-Slabs* (SDI C-2011), Steel Deck Institute, Glenshaw, PA.

Tri-Services, 1988. *Seismic Design Guidelines for Upgrading Existing Buildings* (TM5-809-10), Departments of the Army, the Navy and the Air Force, Washington, D.C.

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RESOURCE PAPER 3
DIAPHRAGM DESIGN: CURRENT PRACTICE, PAST PERFORMANCE AND FUTURE IMPROVEMENTS

RP3-1 CHAPTER 1 INTRODUCTION

Resource Paper 10 (RP 10) in Part 3 of the 2009 NEHRP Provisions (FEMA, 2009) addresses the seismic design of topped and untopped precast concrete diaphragms; this resource paper, however, has become out of date in many ways in the intervening years. While discussing necessary updates to RP10, the Provisions Update Committee (PUC) also discussed the current wide diversity in the ways diaphragms of various construction materials (reinforced concrete, precast concrete, steel deck, and wood) are designed. While uniform design methods may be unnecessary and largely unattainable, it was felt worthwhile to review current practices with respect to the design of diaphragms of each major construction material. It was expected that such review would point out opportunities for improvement in current design procedures; therefore Issue Team 6 (IT6) on diaphragms was formed and charged to "reexamine and refine seismic design provisions for cast-in-place concrete, precast concrete (with or without topping), steel, and wood-frame diaphragms, focusing on objectives with regard to performance in the design earthquake."

This resource paper is divided into multiple chapters on:

- Global and ASCE 7 related issues
- Cast-in-place concrete diaphragms
- Precast concrete diaphragms, with and without topping
- Steel deck diaphragms, with and without topping
- Wood-frame diaphragms

The following global or ASCE 7 related topics were identified as requiring IT6 attention:

- Applicability of ASCE 7 Provisions
- Design force level
- Rigid vs. flexible diaphragms
- Diaphragm stiffness
- Diaphragm deflection
- Ramps & stairs

Among the global issues, the diaphragm design force level was identified to require and deserve more attention than the other issues listed above. IT6 worked on this topic for many months and developed a NEHRP Provisions Part 1 proposal to modify the current diaphragm design force level in ASCE 7-10 Section 12.10. At the direction of the PUC, this proposal was later split into a Part 1 proposal that applies to precast concrete diaphragms, cast-in-place concrete diaphragms, and wood-sheathed diaphragms supported by wood framing and a Part 3 proposal that applies to steel deck diaphragms, wood-sheathed diaphragms supported by cold-formed steel framing, and other diaphragms. These two proposals and accompanying commentaries have been approved by the PUC as well as BSSC Member Organizations. A companion proposal for ASCE 7 Section 14.2 has been developed by IT6 in collaboration with precast diaphragm researchers, addressing design provisions for precast concrete diaphragms. The recommended modifications to ASCE 7-10 Section 12.10 appear in Parts 1 and 3 of the 2015 NEHRP Provisions as noted above, and the accompanying commentaries appear in Parts 2 and 3, respectively. The recommended modifications to ASCE 7-10 Section 14.2 also appear in Part 1 of the 2015 NEHRP Provisions, and the accompanying commentary in Part 2. The remaining global issues are addressed in Chapter 2 of this resource paper. An overview of the recommended modifications is also provided at the beginning of Chapter 2.

The material chapters address: current practice, past performance, and areas of potential improvement. IT6 realizes that the material chapters do not address a number of diaphragm types that are in current use or diaphragm systems that may be developed in the future, including but not limited to lumber-sheathed diaphragms on wood framing and a number of proprietary diaphragm systems. Also commonly used are horizontal truss systems acting as diaphragms, including tie-rod bracing in light steel buildings and structural steel and concrete horizontal trusses. These other systems are provided for in the recommended modifications to ASCE 7-10 Section 12.10 in Provisions Part 3. In the concluding chapter of this resource paper, recommendations are provided for needed future developments in the field of seismic design of diaphragms.

RP3-2 CHAPTER 2 GLOBAL AND ASCE 7 RELATED TOPICS

This chapter addresses the following topics, which have an impact on design irrespective of the material of construction:

- Applicability of ASCE 7 Provisions
- Diaphragm design force level
- Rigid versus flexible diaphragms
- Diaphragm stiffness
- Diaphragm deflection
- Ramps & stairs

RP3-2.1 Applicability of ASCE 7 Provisions

The ASCE 7 provisions for determining diaphragm design force-levels can be applied to a vast majority of the buildings that are designed and built in current practice. The provisions are applicable to structures with well-defined, discreet diaphragms where inertial forces and transfer forces can be readily identified and separated. The diaphragm forces are to be applied to each level of the structure independently. Furthermore, they are required to be applied to all diaphragms within the structure, including mezzanines and partial diaphragms.

Unique, complex structures where diaphragms interconnect between multiple levels of a structure may require a more detailed, rational analysis to determine the design forces in the diaphragm. Structures (or portions of structures) such as stadium, theater, and arena sloped seating areas (Figure 2-1) are examples of such structures containing components in which inertial forces and transfer forces are not easily identified and separated. The diaphragms in sloped seating areas will not only develop their own inertial forces, but they will also transfer seismic forces between multiple stories of the structure.

FIGURE 2-1 Stadium seating diaphragms interconnecting multiples levels of a structure

The ASCE 7-10 provisions for diaphragm forces are not applicable to the unique diaphragms within these types of structures.

RP3-2.2 Design Force Level

The seismic design of a structure in ASCE 7-10 is based on an approximation of the inelastic response of the seismic force-resisting system. The approximation reduces the results of an elastic analysis in consideration of the reserve strength, ductility, and energy dissipation capacity inherent in the vertical elements of the seismic force-resisting system. In 1978, ATC-3 provided design force reduction factors based on consideration of inelastic behavior of the vertical elements of the seismic force-resisting system and the performance of structures in past earthquakes. The primary assumption leading to these factors is that yielding in the vertical elements of the seismic force- resisting system is the primary mechanism for inelastic behavior and energy dissipation.

In contrast, the design requirements for the horizontal elements of the seismic force-resisting system (the diaphragms) have been established by empirical considerations, rather than by reduction of the elastic diaphragm forces due to inelastic action. For established diaphragm construction types, this empirical approach has been generally satisfactory. Satisfactory system performance, however, should theoretically require that the diaphragms have sufficient strength and ductility to mobilize the inelastic behavior of the vertical elements.

The level of diaphragm design force from the empirical equations in U.S. codes and standards does not ensure, however, that diaphragms have sufficient strength to mobilize the inelastic behavior of vertical elements. Analysis tools, not available when the empirical rules were established, permit more accurate determination of realistic forces in diaphragms under design or MCE-level earthquake ground motions. Technological advances have also resulted in designs that allow innovation in pursuit of higher levels of efficiency and economy in construction. New code- compliant designs reveal that the level of force currently required for diaphragm design may not ensure development of inelastic mechanisms in the vertical elements of the seismic force-resisting system. This phenomenon was dramatically illustrated by the response of several shear wall structures during the Northridge earthquake (Iverson and Hawkins, 1994).

In order to help achieve the intended seismic performance of structures, the design of horizontal and vertical elements of the seismic force-resisting system need to be made more consistent. Analytical results as well as experimental results from shake-table tests in Japan, Mexico, and the United States have shown that diaphragm forces over much of the height of the structure actually experienced in the design-level earthquake may at times be significantly greater than code-level diaphragm design forces, particularly where diaphragm response is near-elastic. See Part 2 of the 2015 NEHRP Provisions proposal for detailed discussion. However, there are material-specific factors that are related to overstrength and deformation capacity that may account for satisfactory diaphragm performance. The modifications to ASCE 7-10 Section 12.10 in Part 1 of the 2015 NEHRP Provisions tie the design of diaphragms to levels of force and deformation that represent anticipated behavior. These modifications are incorporated as alternate provisions, located in a new section immediately following the Chapter 12 section addressing diaphragms, chords and collectors (ASCE 7-10 Section 12.10). As a result of renumbering of ASCE 7 sections due to other modifications, the ASCE 7-16 section number for the alternate provisions is not know at this time. These will be referred to as the alternate provisions in the following discussion.

The alternate provisions present an elastic diaphragm force as the statistical sum of first mode effect and higher mode effects (Rodriguez et al., 2002). The first mode effect is reduced by the R- factor of the seismic force-resisting system, but then amplified by the overstrength factor, Ω_0, because vertical element overstrength will generate higher first mode forces in the diaphragm. The effect caused by higher mode response is not reduced by the R-factor nor amplified by Ω_0 (Rodriguez et al., 2007). In recognition of the deformation capacity and overstrength of the diaphragm, the elastic diaphragm force from the first and higher modes of response is then reduced by a diaphragm force reduction factor, R_s.

With the modification by R_s, the alternative provisions design force level may not be significantly different from the current diaphragm design force level of ASCE 7-10 for many practical cases. For some types of diaphragms and for some locations within structures, the diaphragm design forces may be significantly different from those of ASCE 7-10, resulting in noticeable changes to construction. Based on data from testing and analysis and on building performance observations, it is believed that these changes are warranted.

The alternative provisions enable the significantly greater forces observed in near-elastic diaphragms and the anticipated overstrength and deformation capacity of diaphragms to be directly considered in the diaphragm design procedure. This should result in an improved distribution of diaphragm strength over the height of structures and among structures with different types of seismic force-resisting systems.

The PUC at its meeting on May 7, 2014 decided by a majority vote that the alternative provisions be mandatory for precast concrete diaphragms in buildings assigned to SDC C, D, E, and F and that it be optional for other precast concrete diaphragms as well as for cast-in-place concrete and wood sheathed diaphragms on wood framing; these are addressed in Part 1 of the 2015 NEHRP Provisions. Steel deck and all other diaphragms are now addressed only in Part 3 of the 2015 NEHRP Provisions.

The effort required for the alternative provisions diaphragm design procedure has not increased to any significant degree from what is currently required.

RP3-2.2.1 Precast Diaphragm Design Options

The value of the diaphragm design force reduction factor, R_s, for precast concrete diaphragms varies depending upon the diaphragm design option selected among three choices that are made available. The proposed modifications to ASCE 7-10 Section 14.2, intended for Part 1 of the 2015 NEHRP Provisions, include specific provisions for the definition and application of these precast concrete diaphragm design options, including the details of qualification of connectors or joint reinforcement used in design.

RP3-2.3 Rigid versus Flexible Diaphragm Modeling Assumptions

For seismic design of structures, the classification of diaphragms as rigid, semi-rigid or flexible is of interest primarily for determination of force distribution to the vertical elements of the seismic force-resisting system. In general, the distribution of the seismic design story shear to the various vertical elements of the seismic force-resisting system is affected by the rigidity of the diaphragm relative to that of the vertical elements. A rigid diaphragm is one that is modeled to distribute lateral forces to vertical elements based only on the relative stiffness of the vertical elements. The general assumption is that the deformation within the diaphragm is not significant relative to the deformation of the vertical system. This assumption implies that the diaphragm is capable of carrying loads to extreme points on the diaphragm, even when there are large differences in stiffness between individual vertical elements. It also implies that the deformation in the diaphragm does not have a significant effect on drift, so that gravity elements remote from the vertical lateral-force-resisting elements are not subject to significantly larger lateral displacements. These assumptions may not be conservative. For flexible diaphragms, the seismic design story shear is distributed to the various vertical elements based on the area of the diaphragm tributary to each line of vertical elements.

The behavior of diaphragms as rigid, semi-rigid, or flexible depends on many factors, including spans, aspect ratio, jointing and connections. It also depends on the relative stiffnesses of the vertical elements of the seismic force-resisting system.

Starting with its 2005 edition, ASCE 7 has permitted certain diaphragms to be "idealized as flexible." These are untopped steel deck or wood structural panel diaphragms supported by rigid lateral force resisting systems such as those consisting of concrete or masonry shear walls or steel braced frames, and untopped steel deck or wood structural panel diaphragms in one and two-family dwellings. Starting with the 2010 edition, ASCE 7 Section 12.3.1 was expanded to allow flexible diaphragm idealization for structures of light-frame construction with diaphragms of untopped steel deck or wood structural panel, provided that the is untopped or has not more than 1-1/2 inch nonstructural topping and the structure meets code required drift limits at each line of vertical elements of the lateral force-resisting system. ASCE 7 also permits certain diaphragms to be "idealized as rigid." These are cast-in-place concrete slabs acting as diaphragms or topped steel deck diaphragms that do not have significant openings in them and that have long side to short ratios not exceeding three. ASCE 7 further permits a diaphragm that is neither "idealized as flexible" nor "idealized as rigid," to be classified as flexible if the computed maximum in-plane deflection of the diaphragm under lateral load is more than twice the average drift of adjoining vertical elements of the seismic force-resisting system over the story below the diaphragm under consideration, under tributary lateral load equivalent to that used in the computation of the in-plane diaphragm deflection. ASCE 7 considers all other diaphragms to be semi-rigid. These other diaphragms, however, are considered to be rigid in the 2012 and prior editions of the International Building Code (IBC).

RP3-2.3.1 Rigid versus Flexible Precast Concrete Diaphragms

Since story drift can vary level to level, classification of diaphragms per ASCE 7 can result in the same structure having diaphragms that are flexible and not flexible at different levels or at the same level for different load directions. For the purpose of analysis and lateral force distribution in precast concrete diaphragms, a more consistent definition is desirable. Fleischman et al. (2002) have proposed an alternate index, the stiffness reduction factor, α, to classify precast concrete diaphragm flexibility for the entire structure:

$$a = \frac{\delta_{dia,mid}}{\Delta_{lat,mid}}$$

Where:

$\delta_{dia,mid}$ = maximum in-plane deflection of mid-level diaphragm

$\Delta_{lat,mid}$ = total lateral system drift at the elevation of the mid-level diaphragm

This provides an approximate measure of the average relative flexibility of the diaphragm and simplifies the determination for the entire structure. The distinction between rigid and flexible diaphragms is important not just for diaphragm design, but also for the design of the entire seismic force-resisting system.

In their research, Farrow and Fleischman (2003a) investigated common layouts of precast parking garages. The characteristics of these structures include long spans between lateral-force-resisting elements and large openings or discontinuities created by separation of floor plates for the traffic ramps. They investigated the effects of diaphragm and lateral element layout, diaphragm dimensions, mechanical connector strengths and cast-in-place concrete topping on diaphragm deformation. Although even topped diaphragms are actually cracked by the effect of joint tooling and shrinkage that accumulates at the joints, the baseline for deflection comparisons was a monolithic diaphragm of identical thickness.

Although a diaphragm may be designed to remain elastic, the effective section will still be weakened by the joints or by cracking at the joints. In addition, shear deformations in diaphragms are a significant portion of the total deformation. In topped diaphragms, Farrow and Fleischman (2003b) found shear deformation to be 20% of the total lateral displacement for the configurations they studied.

Untopped diaphragms experienced elastic deflections more than twice those of topped systems, with shear deformation comprising approximately one half of the total (Farrow and Fleischman, 2003b). The expression for diaphragm deflection included both flexure and shear terms. Expressions were developed for flexibility factors for flexure and shear.

Examples applying these expressions for typical precast concrete parking structure layouts have been developed. The results calibrated to tests have shown that the effective stiffness of a diaphragm constructed with cast-in-place concrete toping can be as low as 10 to 15% of the gross uncracked section, depending on the many variables considered. An untopped diaphragm using mechanical connectors between components often have stiffness half that of the comparable topped deck.

RP3-2.3.2 Rigid versus Flexible Wood-Frame Diaphragms

Historically it has been by far most common for designers to assume flexible behavior of wood diaphragms for distribution of seismic forces, assigning seismic demand based on tributary area or simple span beam models. This assumption received considerable discussion following poor performance of tuck-under parking buildings in the 1994 Northridge, California Earthquake (SEAOC, 1999). Since that time, appropriate assumptions for distribution of forces have remained largely unresolved, and remain an item of contention in design of wood-frame buildings.

Discussion in the SEAOC Blue Book (SEAOC, 1999) and Recommendations for Earthquake Resistance in the Design and Construction of Woodframe Buildings (Cobeen et al., 2004) provide summaries of information available and information lacking at that point in time, detail the complications in identifying diaphragm behavior, and make interim recommendations for design, pending more rigorous evaluation of appropriate design assumptions.

In summarizing issues affecting the classification of diaphragms and establishment of force distribution, the 1999 SEAOC Blue Book discussion identified the following: “In order to keep the issue of relative stiffness in perspective, it helps to recall that there are a number of aspects in which the behavior of wood lateral force-resisting systems differ from the structural models that are analyzed:

- The behavior of wood structural panel lateral force-resisting systems is not linear elastic and is not very well understood beyond the range of design forces. The behavior under actual earthquake loading may invalidate many of the analytical assumptions that are normally made.
- The contributions of partition walls and wall, floor and roof finishes and toppings in wood structures are thought to have a significant influence on the behavior of the lateral force-resisting system, but there is no method available to designers to quantify this influence.
- The load distributed to a shear wall can be significantly influenced by the stiffness of the element on which the wall is supported.
- Equations are not available for calculation of shear wall deflections for bracing materials other than wood structural panels.
- Equations are not available for calculating diaphragm deflections using unblocked wood structural panel diaphragms.
- Behavior of diaphragms may fall somewhere between rigid and flexible.”

Since 1999, additional information has become available for description of load-deflection behavior of wood structural panels, and design equations are available for calculation of deflections of additional shear wall types and unblocked diaphragms in the range of tabulated design capacities. We are, however, still far from being able to identify appropriate flexibility assumptions for seismic design.

In summarizing available information, the CUREE report (Cobeen et al., 2004) addressed the following five items of information:

1. Tests including wood-sheathed diaphragms typical of light-frame construction demonstrate the ability of diaphragms to redistribute forces through diaphragm rotation, as would be anticipated for semi-rigid or rigid diaphragms.
2. Tests with a noticeable imbalance in shear wall stiffness resulted in significant diaphragm rotation.
3. In buildings relying on diaphragms in rotation, shear walls resisting resulting torsional forces experienced significantly degraded strength and stiffness as a result.
4. Strength and stiffness of light-frame structures are very significantly influenced by the presence and type of finish materials installed over the designated structure, and the varying stiffness of both structural elements and finish materials over a given load cycle.
5. Code deflection equations based on bare woodframe structures tend to significantly overestimate maximum building deflections.

The CUREE report (CUREE, 2004) made the following two interim recommendations for design:

- "Based on available data, discussed in Section B.10.3, it appears that better building performance will result when seismic forces can be resisted locally rather than requiring redistribution to other portions of the structure. For this reason tributary area analysis is recommended for the great majority of buildings. Where tributary analysis is used, code drift limits should be applied at each shear wall line, rather than to the story as a whole."
- "For buildings where code drift limits cannot be met at each wall line, and buildings where the distribution of shear walls suggests a very significant torsional irregularity, analysis using rigid diaphragms will be necessary, and special attention should be given to the loading condition of the perpendicular walls. The designer should consider superimposing the torsional loading with the in-plane loading for the perpendicular walls."

The 1999 SEAOC Blue Book had made similar design recommendations. The CUREE report (CUREE, 2004) provided the following discussion of additional study needed: "In order to move beyond interim recommendations and rationally evaluate the appropriate threshold for rigid diaphragm distribution, the next needed step is an analytical study evaluating a range of shear wall and diaphragm stiffness and varying configurations reflecting actual buildings. Because of the wide range of behavior possible from diaphragms and shear walls, the focus of the study will need to be acceptable building behavior rather than "accurate" behavior. The data available from the woodframe project permits initial development of analytical models needed for this study."

The issues of diaphragm flexibility and resulting seismic force distribution have an impact on many designers on a day to day basis, both in determining force distributions for use in design, and in obtaining approval of design assumptions from building departments. New to the 2010 edition, ASCE 7 Section 12.3.1.1, item c permits the designer to idealize as flexible light-frame structures with wood structural panel sheathed diaphragms, provided 1) the diaphragm is untopped or topped with not more than 1-1/2" nonstructural topping, and 2) the drift under seismic loading at each vertical element of the bracing system is found to be less than the permitted story drift. This new exception provides broad ability for the designer to continue past design practice, using flexible diaphragm analysis. The assumption of a flexible diaphragm may not, however be appropriate in all cases; configurations of concern include torsionally irregular structures and structures with large diaphragm cantilevers. For these structures the designer is encouraged to consider rigid diaphragm behavior or enveloping of rigid and flexible diaphragm behavior; requirements in the upcoming 2015 edition of SDPWS are anticipated to address these configurations of concern. The derivation of simplified design guidance based on the complex range of building behavior would be of great assistance to design engineers. As noted in the CUREE report, the design guidance will need to be related to adequate building performance rather than strictly correct solutions.

RP3-2.4 Diaphragm Stiffness and Modeling Guidance

RP3-2.4.1 Cast-in-place Concrete Diaphragm Stiffness and Modeling Guidance

As discussed in Section 2.3, the relative flexibility of a diaphragm in a structural analysis can have a significant impact on the distribution of forces within the diaphragm and how forces are distributed between the vertical elements. Current practice in the design of structures with diaphragms that are categorized as flexible in accordance with ASCE 7 is to distribute the forces to the vertical elements based upon the position and distribution of the masses supported. In common practice, the mass is assigned to each of the vertical elements on a tributary basis without consideration of torsional effects.

Structures with diaphragms that are not categorized as flexible in accordance with ASCE 7 have different requirements. In such structures, ASCE 7 requires that the inherent torsion resulting from eccentricities between the locations of the center of mass and the center of rigidity be considered. Further, the analysis must consider accidental torsion due to an assumed displacement of the center of mass from its actual position.

When the diaphragm cannot be classified as flexible or rigid, the analysis of the structure is required to explicitly include the stiffness of the diaphragm. This is commonly referred to as a semi-rigid diaphragm analysis. The response of many regular structures may not be sensitive to the relative stiffness of a semi- rigid diaphragm. In structures where there are significant changes in mass or stiffness of the vertical elements, or in structures with offsets of the vertical elements, the diaphragm stiffness assumptions can have a significant impact on the response of the structure.

Figure 2-2 shows a common condition in many of today's tall structures. The slender, relatively flexible tower transitions to a more rigid podium and basement. The podium and below-grade diaphragms will transfer significant amounts of the shear between the core and the lateral force-resisting elements in the podium structure, such as the perimeter basement walls.

The amount of force identified by analysis to be transferred at these levels is highly dependent upon the stiffness assumptions used in the modeling of the ground-level diaphragm. As the assumed stiffness of the diaphragms increases, so too does the amount of force that will transfer out of the cores to the other vertical elements. In these cases, it is important to understand the sensitivity of the behavior of the overall structure to the stiffness of the diaphragm carrying transfer forces, and an iterative approach may be necessary.

RP3-2.4.2 Precast Concrete Diaphragm Stiffness and Modeling Guidance

Discussion of flexibility of precast concrete diaphragms and resulting recommendations for building analysis including diaphragm flexibility effects are provided in Nakaki (2000).

RP3-2.4.3 Wood-Frame Diaphragm Stiffness and Modeling Guidance

Most analysis of buildings using light-frame diaphragms for equivalent static force design will tend towards use of flexible or rigid diaphragm assumptions. Flexible diaphragm analyses most often use hand calculations or very simple spreadsheets. Rigid diaphragm analysis is often conducted using analysis spreadsheets. Some analysis programs are available that automate analysis using these simplified models.

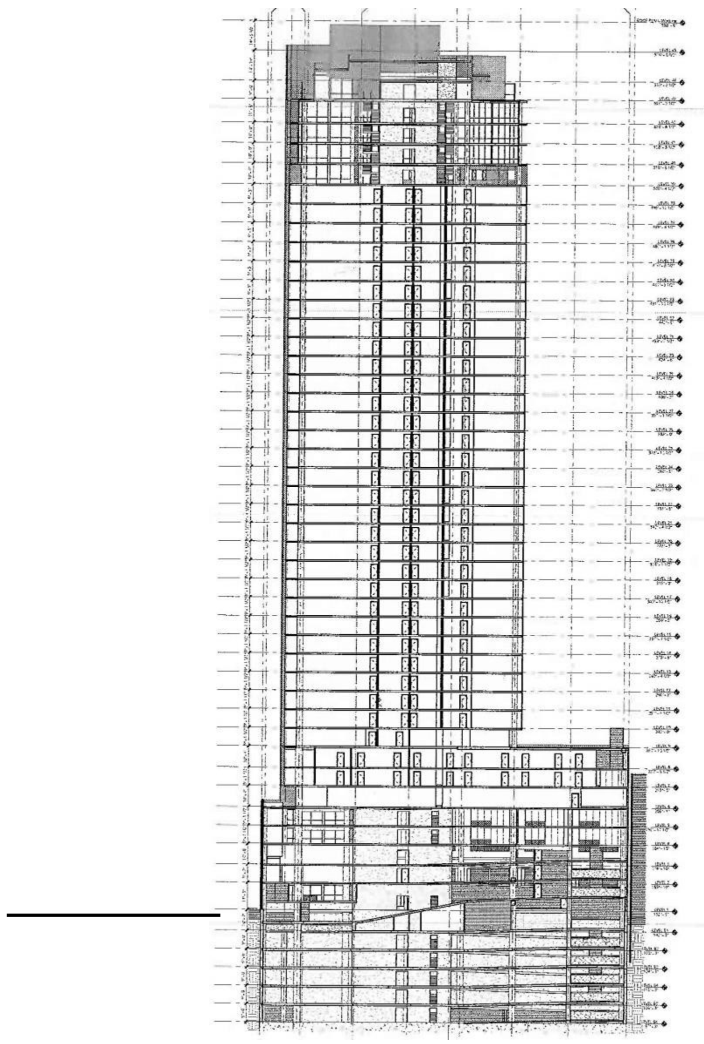


FIGURE 2-2 Flexible tower on a rigid podium structure

Although not common design practice, where detailed computer analysis models using semi-rigid diaphragm properties are developed for seismic design, approaches for linear elastic design include modeling the diaphragm using shell or panel members, or using equivalent diagonal tension and compression struts. In both of these cases, the material properties need to be derived from available load deflection descriptions. Diaphragm deflection equations from AWC's SDPWS Standard (AWC, 2008) can be used for identifying linear stiffness in the range of design capacities, from which shell or diagonal strut properties can be derived. Built into the derivation of the diaphragm equations is the modeling of a simple-span beam in which the shear varies from zero to the design capacity. In the three-part SDPWS deflection equation, the effective stiffness portion will often dominate deflection. Where this is the case, an average effective area and shear modulus can be derived for the shell element, with flexural section properties set to near zero to avoid influence. Diagonal strut properties can be similarly assigned. Where this approach is used, it needs to be recognized that the earthquake performance of the structure will vary from the analysis model behavior due to a number of sources, including varying load level, hysteretic behavior significantly more complex than described, and influence of finish materials including flooring and roofing. Where diaphragm stresses vary notably from design levels, use of testing data published by APA and other sources is recommended. Where semi-rigid descriptions of diaphragms are used for analysis, the user is cautioned to carefully consider the stiffness and potential significant variation in stiffness of the vertical elements of the lateral force-resisting system.

There are only very limited analysis tools in which it is possible or practical to include non-linear behavior of light-frame diaphragms, and those tools are generally only used in a research setting, rather than design applications. One such tool is OpenSees (2000), which permits broad description of nonlinear behavior. Where such nonlinear description is available, the description of hysteretic behavior developed in the CUREE project SAWS Program (Folz and Filiatrault, 2002) and NEESWood SAPWood Program (Pei and van de Lindt, 2010) provide best available descriptions of wood structural panel diaphragm behavior. Commercially available programs are generally not easily adapted to allow adequate descriptions of hysteretic behavior, and should be used with caution.

RP3-2.5 Diaphragm Deflection

Where calculation of diaphragm deflection is not necessary for purposes of diaphragm classification, calculation of diaphragm deflections in design practice occurs under limited circumstances. One such circumstance is where seismic separation joints are required between adjacent structures, and the deflection of the diaphragm is large enough to impact the joint size; this would generally apply to longer span and more flexible diaphragms. Another circumstance is where diaphragm deflection could adversely impact the performance of structural elements (such as non-ductile concrete columns) or components (such as nonstructural walls, cladding systems, or brittle finish materials). Again, this is likely to be of concern with longer span and more flexible diaphragms.

Methods to determine diaphragm deflections are addressed in other chapters of this resource paper.

RP3-2.6 Ramps and Stairs

The behavior of a structure's lateral force-resisting system, including both the diaphragm and the vertical elements, can be significantly affected by ramp and stair framing extending between levels, and by the diaphragm openings that occur at ramps and stairs. Effects of ramps and stairs include discontinuities in the floor geometry, changes in stiffness of both the diaphragm and the vertical elements of the gravity and lateral force-resisting systems, and changes in load paths.

RP3-2.6.1 Ramps

Issues related to ramps were discussed by Moehle et al. (2010): "Ramps and sloping diaphragms can create unique design challenges, especially where they create a connection between different stories of a structure. In some cases, story shear can migrate out of the vertical elements of the seismic force-resisting system through the ramp in the form of shear or axial forces." Lee and Kuchma (2008) further identified that current building codes "do not adequately account for the complicated in-plane force flow associated with ramp cavities." These references only begin to indicate the complexity that may be introduced to floor diaphragms by ramp framing, and similarly by stair framing. It is important to consider the different characteristics in the orthogonal and longitudinal directions.

Ramps are common to parking garage framing. Functionally, there are two distinct types of ramps used in parking garages. One type is called a speed ramp. A speed ramp is used only for the vertical communication of traffic between floors, so it may be both relatively narrow and steep. The second, more common type is the parking ramp, which not only provides the vertical communication but also forms part of the parking surface. This ramp type is limited by the International Building Code (ICC, 2012) to a maximum slope of 1:15. Due to function and slope limitations, this type of a ramp tends to create a large discontinuity in the diaphragm

Consideration of behavior related to ramps should include:

- Discontinuity
- Connection between levels

- The relation of the sloped floor to vertical elements of the lateral force-resisting system, including load paths and changes in stiffness of the vertical elements of both the lateral force-resisting system and of the gravity system

RP3-2.6.1.1 Discontinuity

First, a ramp may form a large discontinuity in the diaphragm. When this discontinuity is very large, it may constitute a Type 3 Horizontal Structural Irregularity. A “Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.” (ASCE, 2010) Although a parking ramp may occupy a large area in a garage, it is only the rare layout that is sufficiently large to meet this criterion. Nonetheless, a parking ramp placed to one side of a parking plan may result in a torsional irregularity as it affects the story drift across the plan. Corkscrew ramp configurations sometimes cause an undesirable overall torsional response (Moehle et al., 2010).

Discontinuities have been considered in the context of the beam analogy, where the distribution of lateral forces is approximated by modeling the diaphragm as a beam spanning across the vertical elements of the lateral force-resisting system. The discontinuity in the floor framing created by a ramp is often considered as a large opening. “Openings in diaphragms, depending on the size of the openings, have a significant influence on the behavior of the diaphragm. Secondary moments are introduced in the diaphragm, resulting in additional tension and compression forces in the diaphragm segment.” (Prasad et al., 2006) “Large openings in the diaphragm result in abrupt change in the stiffness of the diaphragm (additional research is needed in this field)” (Prasad et al., 2006). An elastic study of a parking structure configuration with an interior ramp between outside level bays has been made. “The mid-span deflection decreases significantly as the ratio of ramp landing length (L) to overall floor length (b) increases, a result anticipated by previous work.” (Fleishman and Farrow, 2001) This should be apparent from beam analogy, since the effect of wider end bays is to impose some fixity to the subdiaphragms created by the ramp. This evaluation also reveals that high shear forces develop in the end bays along the lines of the ramp boundaries. These lines occur at beam lines and are called the “seams” in precast concrete framing.

The discontinuity created by a ramp may include the effect of re-entrant corners. These can be locations of accumulating forces not only from lateral forces, but also from volume change strains.

RP3-2.6.1.2 Connections between Levels

For seismic forces parallel to a ramp, the ramp may act as a strut between levels. Early in the development of precast concrete parking garages, designers attempted to use this effect to form bracing as part of a truss in lieu of other lateral force-resisting systems in the longitudinal direction of the garage. When the lateral forces are modest and the structure is only a few stories tall, the accumulated forces can be accommodated. With the advent of more comprehensive seismic considerations in the building code, typical lateral forces in all seismic design categories increased. Larger force requirements made this truss approach infeasible as the area of reinforcement required along the ramp chords became unmanageable except in short structures. Reinforced concrete braced frames are no longer included as a seismic force-resisting system in ASCE 7 Table 12.2-1, so this is no longer an option.

Although the strut behavior is not used as part of lateral design, the requirement for continuous chords creates a load path for seismic forces between levels. This additional force should be considered in the design of sloping diaphragms. The combination of design chord forces and unintentional strut forces may cause yielding; attention to the ductility of the ramp diaphragm chord may therefore be necessary. Where ramps terminate at a rigid foundation, lateral forces can bypass the vertical system. An expansion joint at the base of each ramp can alleviate this problem.

RP3-2.6.1.3 Interface with Vertical Elements

For seismic forces perpendicular to a ramp, the ramp may act as an inclined shear wall, with an effectiveness varying relative to the stiffness of the vertical elements of the lateral force-resisting system. Short columns at the ends of a ramp can accumulate large shear forces. “Under lateral loading, the stiff ramp may limit the building movement and inhibit the movement of the frames as intended. The stiff ramp may short-circuit the frame system and may contribute towards the creation of a weak story condition,” (SEAOC, 2010).

Short columns between sloped ramp and adjacent deck are susceptible to concentrated shear demands. “It is a problem unless special details are used, or lateral force-resisting elements are provided at the vertical offset of the diaphragm.” (FEMA, 1998) For cast-in-place concrete framing, the use of double columns, where one column supports the level diaphragm and one column supports the ramp, can alleviate this short column effect. With precast concrete framing, vertical or horizontal walls with openings are often used to provide vertical and lateral support at the ramps. Unintended out-of-plane forces can be relieved by including flexible, ductile connections between walls and floor diaphragm members on either the ramp or flat side. The connections relieve the short vertical span prying out-of-plane while providing a strong shear force transfer in the plane of the walls. These ramp walls can limit seismic deformation at the discontinuity. Transverse walls at the ends of the ramp can also help limit deformation, but this has the effect of causing the highest diaphragm shear and moment to occur at the same locations.

RP3-2.6.2 Stair and Elevator Core

Framing for stairs and elevator shafts can also create discontinuities and unintended load paths in diaphragms. These features are smaller and usually impose less global effect, but they may be important. There are additional effects with stairs and elevators that should be considered.

Stair and elevator walls may be used as shear walls. The discontinuities or openings may create some isolation of the shear wall from the diaphragm. A load path to the wall may require collectors or drag struts. Some consideration of the limits for direct interface between walls and diaphragms is implied in the FEMA 310 Tier 2 evaluation (FEMA, 1998). In the procedures for diaphragms, it states that “consistent with the beam analogy, a stair or skylight opening may weaken the diaphragm just as a web opening for a pipe may weaken a beam.” (FEMA, 1998) When the openings occur at shear walls or braced frames, the evaluation and acceptance criteria limits openings to 25% of wall length for Life Safety and 15% for Immediate Occupancy.

It is important to consider the effects on the behavior of stairs in altering stiffness and load paths in moment-resisting frame structures. (Cosenza et al., 2008) The effects of intermediate landings include increased stiffness due to inclined beams and short columns. In a modal analysis of the structure, there may be a reduction in building period, and the effect may not be the same in the longitudinal and transverse directions.

Discontinuity caused by stair and elevator framing can form re-entrant corners.

RP3-2.6.3 Analysis and Design

Engineering practice varies with respect to how to treat the above conditions in an analysis model. Idealizing a sloping diaphragm as a flat, continuous element might not correctly identify the forces in that diaphragm, and might lead to over-stating the stiffness of the diaphragm in a particular location. “The potential implications of the modeling assumptions of ramps should be considered when determining whether or not to explicitly include sloping diaphragms in an analysis model” (Moehle et al., 2010). Rational representation of load paths may dictate 3D computer modeling to evaluate ramp effects. The effectiveness of the model, however, is limited by the accuracy in the modeling relative to cracked stiffness, connections and joints, and actual geometry.

Finite element modeling of ramps can be used, but if columns or walls are not modeled at the interface of a ramp and flat diaphragm, then the analysis may not capture the effects of all the axial forces attracted by the ramp. A mesh size of 1/10 to 1/15 of the bay length has been recommended as sufficient in most areas of the model. Section cuts can be used to determine shear distribution. In areas of high force concentration, accuracy may require a finer mesh. For a more accurate determination of stiffness and deformations, there should be a rational reduction in stiffness for cracking. With untopped precast concrete construction, load paths are largely defined by the connections across the joints, and these might be included in the modeling.

Another strategy is to use strut-and-tie models used to identify force paths and reinforcement layouts around discontinuities, but these models may not capture the forces from vertical connection caused by the ramp or stair/elevator framing.

RP3-3 CHAPTER 3 - CAST-IN-PLACE CONCRETE DIAPHRAGMS

This chapter provides background information on cast-in-place concrete diaphragms. See Chapter 4 for precast concrete diaphragms.

RP3-3.1 Building Types and Systems

Cast-in-place concrete construction is utilized in a wide variety of buildings throughout the world. Some of the most common types are multi-unit residential buildings, office buildings, medical buildings, laboratories, retail buildings, parking garages, and manufacturing buildings. Building geometries vary from relatively simple, rectangular layouts to highly complex plan shapes. Similarly, building heights vary from single-story structures to super-tall buildings.

Gravity load-supporting systems within concrete structures vary widely. In general terms, concrete gravity systems fall into two basic types: one-way or two-way systems. In a one-way system, the concrete slab is reinforced to resist flexural stresses in one direction only. One-way slabs are supported by joists or beams that span between building columns, or they are supported directly by bearing walls. Common types of one-way systems include one-way flat slabs, and narrow- and wide-module pan joist systems. Two-way slab systems are reinforced to resist flexural stresses in two, typically orthogonal, directions. Common types of two-way systems include flat plates, flat slabs, and waffle slabs. Many buildings contain some combination of these gravity load-supporting system types. Slab reinforcement can be non-prestressed (conventional mild reinforcing), or post-tensioned (PT) for any of these common types. Column and wall reinforcement is most often not post-tensioned. Figure 3-1 illustrates examples of one-way and two-way concrete gravity load-supporting systems.

Non-post-tensioned gravity load systems are all required to contain a minimum amount of reinforcement. In a concrete slab, this is most often provided as a continuous mat of reinforcing steel placed at the bottom of the slab to help control the size of cracks that develop as the concrete shrinks during curing. The required amount of this reinforcement is equal to 0.0018 (0.18 percent) of the slab cross section. This temperature and shrinkage steel also resists flexural demands. Where flexural demands are high, additional steel is provided as necessary. Many slabs do not contain continuous top reinforcement. Top reinforcement is provided in slabs in regions where flexural demands result in tension on the top of the slab and is usually concentrated over the columns.

Post-tensioned slab systems differ from non-prestressed slabs in that the concrete is put into a state of compression by post-tensioning the slab tendons. This compression is induced in the slab while the slab is still supported by a shoring system. Post-tensioning is usually applied before the concrete reaches its full, specified, design compressive strength. The post-tensioning serves two purposes. First, it replaces the temperature and shrinkage reinforcement that would normally be required in a non-prestressed slab. Second, the post-tensioning helps resist flexural loading. Post- tensioning tendons can be bonded or un-bonded. Most post-tensioned slabs used in buildings in the United States are unbonded. Bonded post-tensioning is used in other parts of the world and in some cases in the United States for precast beams and girders.

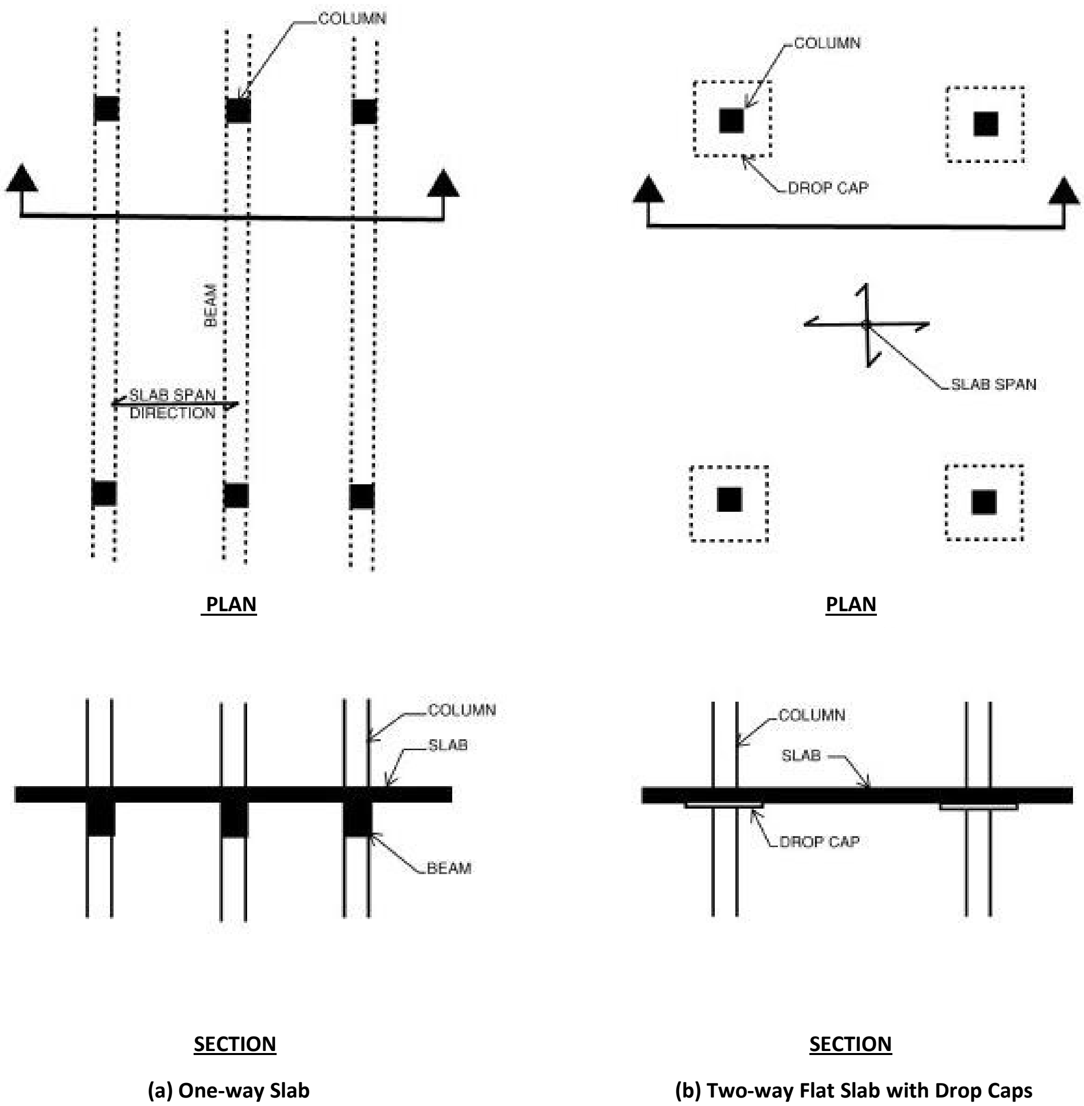


FIGURE 3-1 Examples of Concrete Systems

FIGURE 3-2 Post-Tensioned Slab Reinforcement at Column

In an un-bonded post-tensioned two-way slab, ACI 318-11 Section 18.3.3 limits the tensile stress under service loading conditions to $6\sqrt{f'c}$. The modulus of rupture of concrete is often considered to be approximately $7.5\sqrt{f'c}$. Thus, an un-bonded post-tensioned slab is intended to remain uncracked under service loading. Non-prestressed reinforcement is only required by ACI 318 over the columns and in other locations where tensile stresses exceed certain limits. As a result, there is very little non-prestressed reinforcement in a PT slab compared to a non-prestressed slab.

Lateral forces are typically resisted by concrete moment frames, shear walls, or cantilevered columns. Many times, the vertical elements of the seismic force-resisting system contain combinations of frames and walls – as the resulting seismic force-resisting system may be a dual system or a building frame system. In high seismic design categories, restrictions and limitations are placed on the use of some of these systems. For example, current building codes require that reinforced concrete shear wall buildings be supplemented with intermediate or special moment-resisting frames for buildings that exceed 160 feet in height in Seismic Design Categories (SDC) D through F. These limitations can be increased to 240 feet if certain prescriptive requirements are met. Further, prescriptive reinforcement detailing requirements must be complied with to ensure that the structure can meet required ductility demands.

In most concrete buildings, the slab, which supports gravity loads, is also the diaphragm. This is true in both one-way and two-way systems. These diaphragms are also required to perform several other secondary functions within a concrete building. For example, concrete slabs often serve as lateral bracing for columns and walls. Slabs provide resistance to out-of–plane forces from wind, seismic, hydrostatic, and other lateral forces that act on the cladding of the structure. Slabs also resist the inertial forces that develop during an earthquake or wind storm and distribute these forces to the vertical elements of the lateral force-resisting system.

FIGURE 3-3 A shear wall in a concrete structure

RP3-3.2 Diaphragm Forces and Intended Behavior

At all locations of the diaphragm, it must be designed to resist all of the forces acting on it, including those due to gravity, wind, soil loading, etc. The scope of this document is limited to diaphragm design for seismic forces.

Seismic forces in concrete diaphragms are determined using ASCE 7 (ASCE, 2010). The seismic design force equations are covered in more detail in Section 2.2 of this report.

The provisions governing the design and detailing of cast-in-place concrete diaphragms are written with the intent that the diaphragm remains elastic (Cleland and Ghosh, 2012). A diaphragm is not intended to act as an energy dissipating mechanism for the overall structure. Significant inelastic response should occur in the vertical elements of the seismic-force resisting system.

During a design-level earthquake, it is possible that some limited inelastic response of the diaphragm occurs if the forces within the diaphragm are higher than those predicted using the ASCE 7 equations. The extent to which this may occur is not well understood. Cast-in-place concrete diaphragms are typically designed with an assumed, discrete load path for the diaphragm forces. Due to their monolithic nature, concrete diaphragms may often have the ability to re-distribute forces within the diaphragm, depending upon geometry and reinforcement configurations. Further, energy may be dissipated within the diaphragm in the form of cracking and yielding of reinforcement.

On the capacity side of design equations, both concrete and reinforcing steel often have capacities that are greater than the specified capacities. Although this over-strength is variable, it is not uncommon for concrete to achieve compressive strengths as much as 30% greater than the specified compressive strength. Actual yield stress in reinforcing steel can be as high as 25% higher than the specified minimum yield strength. This over-strength, however, cannot be explicitly relied upon in diaphragm design practice.

While material over-strength can enhance the actual capacity of a concrete diaphragm, over-strength can have adverse effects on the demand side of design equations. Consider a coupled concrete shear wall where the primary seismic mechanism is yielding of the coupling beams. If the actual yield strength of the as-built coupling beam longitudinal reinforcement is 25% higher than specified, the entire structural system could be subjected to forces that are significantly higher than the forces the system was designed to resist.

Many concrete diaphragms are effectively rigid, according to the definition of a “rigid” diaphragm in ASCE 7. When the aspect ratio of a diaphragm exceeds 3 to 1, or when “horizontal irregularities” defined in ASCE 7 exist, the relative stiffness of the diaphragm must be considered when distributing seismic forces between the vertical elements of the seismic force-resisting system. Modern finite element analysis software allows for the stiffness of the diaphragm to be considered in the overall building analysis. Cracking in the concrete slab is considered using reduced in-plane stiffness values that typically fall in the range of 15% to 50% of the gross section properties (Cleland and Ghosh, 2012). It is possible for a concrete diaphragm to be classified as a calculated flexible diaphragm in certain instances, such as a long, rectangular parking garage.

RP3-3.3 Analysis and Design

RP3-3.3.1 Methods of Analysis

Analysis of concrete diaphragms can range from simple hand calculations to complex finite element analyses. The choice of method of analysis depends on the relative complexity of the structural systems and loads involved. Diaphragms are typically, but not always, modeled separately from the overall structure on a floor-by-floor basis. Prasad et al. (2008) describes methods of analyzing forces within diaphragms.

When structures have regular configurations, have just two lines of lateral force-resisting elements in each direction, and there are no horizontal offsets in the lines of lateral force-resisting elements, it may be appropriate to use an equivalent beam model to analyze floor diaphragms. When there are more lines of lateral force-resisting elements with no horizontal offsets, then an equivalent model of a beam supported on elastic springs may be appropriate.

Another way to analyze the forces in the diaphragm is to use the distribution of forces in the vertical elements determined from a computer model to back-calculate the distribution of the inertial forces in the diaphragm, and subsequently determine the shears and moments in the diaphragm. Moehle et al., (2010) describe the details of this method of analysis.

If there are significant diaphragm discontinuities, then strut-and-tie modeling or complete three-dimensional finite element modeling of the diaphragm may be appropriate. Strut-and-tie modeling is particularly useful for determining reinforcement configurations in portions of diaphragms around openings and re-entrant corners.

More complex methods of analysis are needed when the structure has vertical, horizontal, or torsional irregularities, or when there is significant transfer of forces from one vertical element to another occurring within the diaphragm. In these cases, the overall building model may consider the flexibility of the diaphragm in order to identify the transfer forces. Modern finite element software can provide sophisticated tools to model a complex diaphragm in great detail.

Regardless of the method of analysis used, the distribution of forces determined in the analysis of the diaphragm must be consistent with distribution of forces assumed in the analysis of the overall structure. In some cases, diaphragm analysis is based on bounding the analysis between two of the above methods.

RP3-3.3.2 Material Standard

For structures assigned to Seismic Design Categories B or C, the diaphragm design shall satisfy all of the relevant provisions in Chapters 1-18 of the American Concrete Institute (ACI) Standard 318-11. For structures assigned to Seismic Design Categories D, E, or F, the diaphragm design shall also satisfy the provisions of Chapter 21. Concrete diaphragm design follows the strength design philosophy described in ACI 318-11.

RP3-3.3.3 Diaphragm Shear

Shear design of concrete diaphragms is relatively straightforward. For SDC B and C, shear design provisions are found in Chapter 11 of ACI 318-11. In most cases of diaphragm design, the shear strength equations found in Section 11.9 for walls are also appropriate for use in diaphragm design.

The diaphragm shear strength is the shear strength of the concrete, VC, plus the additional strength provided by the shear reinforcement, VS. Thus the total shear strength can be expressed as:

$$\phi V_n = \phi\,(V_C + V_S)$$

Where:

ϕ = 0.75 or 0.60

$V_C = 2\lambda\sqrt{f'_c}hd$

$V_S = A_s f_y d/s$

The concrete shear strength, VC, is a function of the concrete density, specified compressive strength, thickness of the slab, and the effective depth of the diaphragm. The steel strength, VS, is a function of the area, spacing, and yield strength of the shear reinforcement and the effective depth of the diaphragm.

The total shear strength of the diaphragm, including the contributions of both steel and concrete, is limited to a maximum of:

$$\phi V_n = 10\sqrt{f'_c}hd$$

For structures assigned to SDC D through F, ACI 318-11 Section 21.11 contains explicit diaphragm shear strength equations. The diaphragm shear strength is defined in Equation 21-10 as:

$$\phi V_n = \phi A_{cv}(2\lambda\sqrt{f'_c} + \rho_t f_y)$$

This equation can be derived from the Chapter 11 provisions by setting Acv = hd and setting ρt = AS/sh. Therefore, Equation 21-10 could be used for the design of diaphragms in all Seismic Design Categories.

Shear failure in concrete members is brittle in nature, characterized by a sudden, rapid loss in load-carrying capacity. Therefore, a diaphragm that is designed to exhibit elastic response to seismic forces cannot be permitted to fail in shear. ACI 318 Sections 9.3.4 and 21.11 provide an enhanced level of reliability in diaphragms of structures assigned to SDC D through F and with certain structural systems that are known to have significant over-strength.

ACI 318-11 Section 9.3.4 states, "For structures that rely on intermediate precast structural walls in SDC D, E, or F, special moment frames, or special structural walls to resist earthquake forces, E, ϕ shall be modified as given in (a) through (c):

a. For any structural member that is designed to resist E, ϕ for shear shall be 0.60 if the nominal shear strength of the member is less than the shear corresponding to the development of the nominal flexural strength of the member. The nominal flexural strength shall be determined considering the most critical factored axial loads and including E;

b. For diaphragms, ϕ for shear shall not exceed the minimum ϕ for shear used for the vertical components of the primary seismic-force-resisting system.

c. For joints and diagonally reinforced coupling beams, ϕ for shear shall be 0.85.

The commentary for section 9.3.4 states that 9.3.4 (a) applies to the shear design of brittle members including diaphragms. This is because it can be difficult to reinforce a concrete diaphragm to provide enough strength to resist the shear that can be developed when a diaphragm reaches its nominal flexural strength.

Further, Section 21.11.9.2 limits the nominal shear strength of concrete diaphragms in structures assigned to SDC D through F:

$$V_n \leq 8A_{cv}\sqrt{f'_c}$$

Thus, it is possible for a concrete diaphragm in a low seismic design category with a low-R system to be designed to a maximum shear strength equal to $7.5\sqrt{f'c}$, whereas in a high seismic design category with a high-R system, the maximum unit shear stress may be as low as $4.8\sqrt{f'c}$.

The shear reinforcement (not shear friction) is uniformly distributed horizontally across the width of the slab and is oriented perpendicular to the diaphragm flexural (i.e. chord) reinforcement. In many structures with a regular configuration and concrete diaphragms, the concrete alone is adequate to resist diaphragm shear forces, and additional reinforcement is not usually required, even in PT slabs. When irregularities are present that result in transfer of concentrated forces, then diaphragm shear reinforcement is more likely to be required. Diaphragm shear reinforcement is also more likely to be required in regions of a diaphragm near openings, reentrant corners, or other discontinuities.

RP3-3.3.4 Diaphragm Flexure

Flexure in reinforced concrete diaphragms is commonly resisted by reinforcement concentrated near the edges of the diaphragm, called chord reinforcement. The tension chord force, T_u, is typically calculated as follows:

$$T_u = M_u/d_{eff},$$

Where:

M_u = Factored moment

d_{eff} = Effective depth of the diaphragm section

For nonprestressed reinforcement, the required area is then determined as follows:

$$A_s = T_u / \phi f y$$

ACI 318-11 Section 21.11.8 permits designers to distribute flexural reinforcement across the width of the diaphragm. This can have implications on the shear stress distribution. Thus, the diaphragm shear and flexural design assumptions must be compatible.

When diaphragm flexural (chord) reinforcement is located near the extreme flexural edges of the diaphragm, a uniform shear stress through the depth of the diaphragm is a reasonable assumption. In practice, it is recommended that the flexural reinforcement be placed in the outer quarter of the diaphragm for this assumption to be appropriate (Moehle et al., 2010).

Bonded tendons may be used to resist diaphragm flexure. In Seismic Design Categories D, E, or F, ACI 318-11 Section 21.11.7.2 limits the stress due to design earthquake forces to 60,000 psi. Unbonded tendons are not allowed to resist diaphragm flexure directly, however the precompression in a diaphragm from unbounded tendons can be considered if a complete load path for all diaphragm actions is provided.

Unbonded tendons that provide precompression are typically proportioned to resist full factored gravity loads. Depending upon the actual dead load to live load ratio, not all of the precompression is required to resist vertical loads in load combinations that include seismic forces. According to Moehle, et. al. (2010), for the case of DL = 120 psf, LL = 40 psf and reducible by 40% and a minimum prestress of 125 psi, approximately 15 psi may be available in reserve to resist diaphragm flexure. This was determined by using the ratio of the load factors for load combinations that include only gravity loads, relative to load combinations that include seismic loads. Therefore, the moment that can be resisted by the precompression is 15 psi multiplied by the gross-section modulus of the diaphragm section. Moment above this value must be resisted by additional bonded tendons.

Previous versions of ACI 318 up to ACI 318-05 required transverse reinforcement (confinement) of the compression chord for structures assigned to Seismic Design Categories D, E, or F if the compressive stress exceeded 0.2f'c. In ACI 318-08 onwards, this is no longer a requirement for compression chords.

RP3-3.3.5 Collectors

Collector forces can be resisted by bonded reinforcement, bonded tendons, or precompression from unbonded tendons. As is the case with tension chords, ACI 318-11 limits the stress due to collector forces in bonded tendons to 60,000 psi for structures assigned to Seismic Design Categories D, E, or F. Similar to chord reinforcement, the location and extent of collectors within a concrete diaphragm must be consistent with the assumed overall distribution of flexure and shear.

Unconfined concrete in compression exhibits brittle behavior, much like concrete in shear. A sudden loss of capacity in a collector could result in a sudden redistribution of forces within the diaphragm. The addition of confining reinforcement enhances the ductility of concrete in compression.

Transverse reinforcement is required in collectors if the compressive stress in the collector exceeds $0.5f'c$ for structures assigned to Seismic Design Categories D, E, or F per ACI 318 Section

21.11.7.5. The transverse reinforcement needs to satisfy ACI 318-11 Section 21.9.6.4(c). This assumes that seismic load effects including overstrength factors govern the design of the collector. If the basic seismic load combinations without overstrength factors govern, then transverse reinforcement is required if the compressive stress exceeds $0.2f'c$.

For an assumed rectangular cross section, transverse reinforcement is required to consist of rectilinear hoops with or without crossties. The total cross-sectional area of the transverse reinforcement, A_{sh} shall satisfy Equation 21-5 of ACI 318-11:

$$A_{sh} = 0.09sbcf'c/fyt$$

Where:

s = spacing of hoops along the length of the collector

bc = core dimension perpendicular to the tie legs that constitute A_{sh}

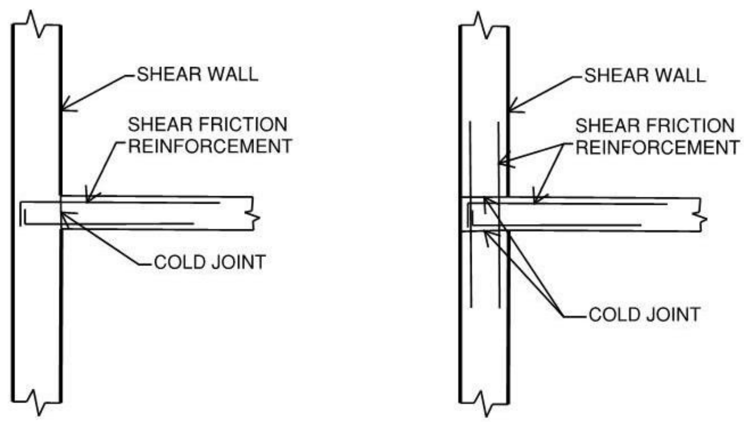
Each corner of the hoop and end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Crosstie spacing within a cross section of the boundary element shall not exceed 14 in. (350 mm) and consecutive cross ties shall be alternated end for end. The minimum spacing, s is defined by ACI 318-11 Section 21.6.4.3.

For a collector within the depth of a thin concrete slab, it can be difficult to meet the confinement requirements due to reinforcement congestion. The slab thickness can be increased, or a beam can be introduced until the compressive stress is below the confinement trigger, or the member is sufficiently large to facilitate placement of the transverse reinforcement.

Alternatively, the assumed width of a collector can be increased, and the collector reinforcement distributed horizontally over a broader effective slab width. Currently, there are no prescriptive code definitions for the effective width of collectors. SEAOC (2005) provides some recommendations that have been used in practice. It is important to note that distributing collector reinforcement can often result in local stress concentrations due to eccentricities. These stress concentrations must be addressed in the design of the diaphragm.

RP3-3.3.6 Connections to the Vertical Members of the Seismic Force-Resisting System

Diaphragm shear is typically transferred to the vertical members of the seismic force-resisting system via shear friction. The shear friction coefficient, μ should give due consideration to the location of construction joints and the roughness of the shear friction plane. See Figure 3-4. Note the different cold joint locations which would result in different shear friction coefficients at the vertical shear plane of the slab-to-wall connection. The unit shear force in the diaphragm at the connection should be consistent with the unit shear force in the diaphragm at the connection location, and it should not exceed the unit shear capacity of the diaphragm.



(a) Walls cast ahead of slabs (b) Walls cast level-by-level

FIGURE 3-4 Common cast-in-place concrete wall to slab connections

Collector forces should be directly transferred to the vertical members of the seismic force-resisting system, wherever possible. Tensile forces are transferred via direct anchorage of the collector reinforcement into the vertical member. Compressive forces can be transferred via bearing between the vertical member and the collector. Any reinforcement present can be considered when evaluating the strength of the interface, as long as combined actions due to gravity and seismic forces are considered.

Collector reinforcement must extend into the vertical member to develop the force in the collector as a minimum. It shall also be embedded far enough that the collector force can be transferred into the vertical member. This length often exceeds the development length of the collector reinforcement.

RP3-3.3.7 Detailing and Dimensional Requirements

ACI 318-11 Section 21.11.6 specifies a minimum thickness for cast-in-place concrete diaphragms of 2 in. (50 mm). This requirement is commonly associated with topping slabs on metal deck or precast deck, but can apply to cast-in-place systems with thin slabs like waffle slabs and joist systems.

ACI 318-11 Section 21.11.7 defines the reinforcement requirements for diaphragms. This section requires minimum diaphragm reinforcement for temperature and shrinkage control as defined in Section 7.12. Spacing of diaphragm reinforcement can't exceed 18 in. (450 mm), except for post-tensioned slabs, where the minimum average prestress limits crack widths. Diaphragm shear reinforcement must be continuous and uniformly distributed across the shear plane.

All reinforcement used to resist collector forces, diaphragm shear, or flexural tension is required to be developed and spliced for fy. (ACI 318 Section 21.11.7.3). Mechanical splices used to connect the diaphragm to the vertical elements of the seismic force-resisting system are required to be Type 2 (ACI 318-11 Section 21.11.7.4).

Longitudinal reinforcement for collector elements at splices and anchorage zones are required to have a minimum spacing of three longitudinal bar diameters, but not less than 1½ in. (40 mm). Additionally this reinforcement is required to have a minimum cover of 2.5 bar diameters, but not less than 2 in. (50 mm). In lieu of these spacing and cover requirements, confinement reinforcement may be provided (ACI 318, Section 21.11.7.6).

RP3-3.3.8 Special Considerations

Diaphragm forces must be transferred around all slab openings. The strut-and-tie method is a useful tool to determine force paths and reinforcement layout around the openings. This type of modeling is also useful when designing for local stresses that occur at other discontinuities such as reentrant corners. If the strut-and-tie design is implemented, then the struts are subject to the confinement triggers of ACI 318-11 Section 21.11.7.5.

Openings that are adjacent to the vertical elements of the seismic force-resisting system should be avoided during planning, if possible. When openings are required for architectural or other reasons, the load path and reinforcement layout for the force transfer to the vertical elements must be appropriately designed.

Diaphragm design must consider the transfer of force at vertical offsets such as ramps, steps, or depressions. Undesirable torsion and/or vertical forces can result at steps and depressions. Additional reinforcement designed to resist these actions must be provided. In the case of ramps that may tie two or more stories of a structure together, the analysis of the structure should consider the potential impact of the ramp.

Structural elements that are not explicitly part of the seismic force-resisting system must be designed for the stresses resulting from displacement of the diaphragm due to its inertial forces. The diaphragm displacements must be considered in conjunction with overall story drifts. These displacements can induce shears and moments in gravity columns, for example. Similarly, non-structural components such as cladding or mechanical systems must be capable of withstanding these displacements.

Reinforcement congestion can be a concern in concrete diaphragms, especially in collectors and at connections to the vertical elements of the seismic force-resisting system. Collectors often connect to the boundary zones of concrete shear walls. Careful study of reinforcement detailing is recommended to ensure that all of the intersecting reinforcement can fit in a manner that maintains minimum spacing and cover requirements.

The placement of diaphragm reinforcement can impact the overall design of the structure. For example, diaphragm shear reinforcement placed in the bottom layer of a two-way slab will resist gravity loads in combination with lateral loads. Another common challenge occurs when there are perimeter moment frames of a dual-system or a special moment frame system, and the chord or collector reinforcement is placed within (or near) the moment frame beam. The diaphragm reinforcement can increase the maximum probable moment capacity of the beam, thus impacting the shear design of frame joints.

The intended behavior of the diaphragm can be altered by the presence of conduits, floor-boxes, plumbing rough-ins, and other items that are cast into the slab. Designers should coordinate the diaphragm design with specifiers and installers of these items.

Diaphragms that are required to resist large transfer forces usually require the most attention in analysis, design, and detailing. The relative stiffness of the diaphragm that is used in the analysis can have a significant impact on the magnitude of the transfer forces that develop. This is especially true in transfer diaphragms with horizontal offset irregularities. In a finite element analysis, the diaphragm gross section properties are typically modified to account for cracking. There are no prescriptive requirements for the level of cracking that is to be assumed in the analysis. However, concrete transfer diaphragms are typically modeled with a stiffness up to 50% of the gross section (Moehle et al., 2010). Softer diaphragms will transfer less force than a stiffer diaphragm. It may be prudent to study the sensitivity of the structure's response to the diaphragm stiffness to arrive at the most appropriate value.

RP3-4 CHAPTER 4 - PRECAST CONCRETE DIAPHRAGMS, TOPPED AND UNTOPPED

RP3-4.1 Common Building Types

Precast/prestressed concrete components are used in many building types and for many functions. A review of common building types provides a context for the consideration of diaphragm analysis, design and behavior.

RP3-4.1.1 Parking Garages

Parking garages are one on the most common building types constructed with precast/prestressed concrete. These structures can range from small, one-level decks to very large multi-story facilities. They are most commonly constructed with simple span precast concrete double tees on gravity load framing consisting of simple span beams and vertically continuous precast concrete columns. The typical bay spans range from 55 ft. (17 m) to 65 ft. (20 m), with narrower bays sometimes used for speed ramps. To accommodate the vertical movement of vehicles, these structures include ramps unless the building site has sufficient slope to permit separate access at each framed level. Ramp layouts may be single helix or double helix when the slope is sufficiently gradual to permit parking on the ramped bays. Steeper ramps can be used when they accommodate only traffic lanes.

Unless steeper outside ramps are used, parking garages generally must have two adjacent bays, but there may be more. As unheated structures, there is typically a dimensional limit of 300 ft (90m) to 350 ft (105 m) for the structural unit (between expansion joints), depending on climate and connection characteristics.

Interior gravity framing is formed with shallow inverted tee beams that permit traffic to cross between adjacent bays or with ramp walls. Exterior gravity framing is made with deep, slender spandrel beams or walls. The lateral force-resisting system is usually limited to only those elements required for lateral bracing of the design forces, and no more. The lateral force-resisting system most commonly consists of shear walls. The shear walls may be exterior walls, stair and elevator core walls, ramp walls, transverse walls at interior ramps, or a combination of these. Moment- resisting frames are also used, but they are much less common. Most parking garages are in the three- to six-story range, but one- and two-story garages are common and some garages may exceed 10 stories.

RP3-4.1.2 Industrial/Food Processing Facilities

Precast/prestressed concrete framing is also used for industrial facilities and food processing buildings. The inherent fire resistance and the hard, washable surfaces with minimum ledges are advantages in these applications. These buildings are usually constructed with single- or two-story framing, except for process towers. Roof framing typically ranges from 60 ft. (18 m) to 80 ft. (24m) spans to provide more column-free interior space. They frequently include perimeter insulated flat slab wall panels. Perimeter framing may be simple span beams on columns with the walls as cladding, or the walls may be designed as loadbearing walls without additional framing. The interior gravity system usually consists of rectangular, inverted tee, or pocketed inverted tee beams on columns. Interior support may be walls at stairs and elevators, fire walls, or interior shear walls. Roof framing is completed with precast concrete double tees with 2 in. (50 mm) or 3 in. (75 mm) flanges.

RP3-4.1.3 Data Centers

In recent years, the special requirements for hardened facilities with high resistance to extreme environmental loading have been met with precast/prestressed concrete framing. These buildings are typically single-story, except two-story framing may be used to support emergency generators. The framing is similar to that for industrial buildings, except load criteria from wind load or earthquakes reflect higher risk categories with higher importance factors. Loads are often specified beyond minimum code requirements to ensure that a higher level of performance is achieved. Roof framing uses precast concrete double tees with 2 in. (50 mm) flanges and 2 to 3 in. (50 to 75 mm) cast-in-place concrete topping for additional diaphragm integrity. When the design wind speed exceeds 160 mph (260 km/h), an additional ballast slab above a membrane roof may be used to resist uplift that exceeds the dead weight of the precast framing.

RP3-4.1.4 Warehouses

When fire resistance and durability are important to the safe storage of materials, precast/prestressed concrete is used for warehouse framing. These are usually buildings with single-story framing similar to industrial buildings.

RP3-4.1.5 Residential – Multi-Family Buildings (with loadbearing walls)

Multifamily residential, high-rise condominiums, and hotels are uses that are well adapted to precast/prestressed concrete. These buildings are framed with hollow core slabs on loadbearing precast concrete walls or loadbearing concrete masonry walls.

Hollow core slabs typically have no structural topping in low and moderate seismic design categories, but topping is used with high seismic design categories to develop diaphragm behavior when the in-plane lateral forces are larger. The lateral force-resisting system most commonly consists of loadbearing and non-loadbearing shear walls, with additional specific requirements for details for structural integrity and disproportionate collapse. Structural integrity details include interior and perimeter ties in the floors that may contribute to diaphragm behavior.

RP3-4.1.6 Office Buildings

Although not as common as parking garage uses, there are regions in the United States where precast and prestressed concrete is also used for office building framing. Office framing is similar to parking garage framing, except that there is no requirement for ramping, interior bay spans may be shorter, and cast-in-place concrete topping over double tees is used to provide level floors, due to the natural camber in these prestressed floor members. The perimeter gravity support may be spandrel beams on columns or loadbearing and non-loadbearing walls. Interior framing may include simple span beam and column framing and walls at stair and elevator cores.

RP3-4.2 System Descriptions

Precast concrete diaphragm systems have distinct characteristics based on the horizontal framing components used to form the floor or roof: hollow core units or double tees. In precast concrete systems, the connections in the plane of the diaphragm have primary importance. The character and behavior of the connections or topping reinforcement between the precast gravity components will determine the overall strength, stiffness and ductility of the system. These connections define load paths in a more fundamental way than the components themselves.

Hollow core diaphragms may be untopped, with grouted keyway joints and grouted end joints with chord reinforcing. Hollow core diaphragms may also include cast-in-place concrete topping slabs that contain the reinforcement for shear-friction transfer across joints and end chord reinforcement. The typical manufacturing processes for hollow core units is such that the top surface may be sufficiently rough to bond with the topping concrete, but may be deemed not to conform to the requirements of ACI 318-11 for high seismic design categories. ACI 318-11 Section 21.11.4 requires the surface to be "clean, free of laitance, and intentionally roughed." This does not define an enforceable measure of surface roughness, and has led to disagreements. Hollow core diaphragms with cast-in-place concrete topping are usually considered non-composite when used in Seismic Design Categories D, E and F.

Double tee floor framing may be constructed to form diaphragms in three ways. The flanges of the double tees may be used to support a cast-in-place reinforced concrete topping slab. When the topping is made composite with the double tee flange, it must have a thickness of at least 2 in. (50 mm). When the slab is designed not to rely on composite action, it must have a thickness of at least 2 ½ in. (65 mm) (ACI 318-11 Section 21.11.6.) This distinction seems somewhat artificial, because whether the topping is composite or not, the weakest locations and the locations that must accommodate most of the deformation or strain are at the joints between precast elements. At these joints, only the topping slab thickness exists, and this thickness is usually reduced by tooling control joints in the cast-in-place concrete to prevent random shrinkage cracks. For high seismic design categories, only shear friction is considered in the strength of shear transfer across the joint. The added strength that may be provided by welded or other mechanical connections is not considered. When the reinforcement for shear friction is welded wire reinforcement, ACI 318-11 requires a minimum spacing between wires parallel to the joint to be 10 in. (250 mm) in SDC D, E and F to avoid fracture by providing sufficient length for strain distribution. When the spacing of joints exceeds 10 ft. (3 m), however, calculations indicate that temperature change strain concentrated at the joints may be sufficient to yield the reinforcement.

The flanges of the double tees may be connected at the joints with welded mechanical connections that have sufficient strength and ductility to transfer shear even as the joint experiences some joint opening from in-plane tension. The chord tension can be resisted with transverse reinforcing embedded in narrow cast-in-place concrete strips that cross the ends of the doubles tees. These "pour strips" are made composite with a reduced thickness part of the flange that is intentionally roughened to ensure bond between precast can cast-in-place concrete. The pour strip thickness may also be increased as a wash or a curb to provide better cover for corrosion protection and development of the transverse reinforcement. The width of these pour strips may be as small as 2 ft. (0.6 m) to 3 ft. (1 m). Where the double tee floors are supported on inverted tee beams and the diaphragm is made continuous across the supporting beams, the pour strips cross the beams and are reinforced transverse to the double tee end joints for shear that can develop along this seam.

In regions with low or moderate seismic design categories, it may also be feasible to develop double tee diaphragms with only mechanical connections to transfer shear and the end chord tension and compression across the joints. These untopped systems are sometimes called "dry" systems, because they require no cast-in-place concrete to complete the diaphragm. These systems are currently limited to lower seismic demand because reliable ductile connections for the chords subject to field tolerances have not been demonstrated. These systems generally must be designed for the chords to remain elastic.

RP3-4.3 Diaphragm Design Forces and Current Recommended Practice

The primary loading that governs the design of the components in the floors that form diaphragms is the gravity load. Dead and live gravity loads determine the requirements for the sections, the amount of reinforcement, and the amount of prestressing. These requirements also drive the layout, spans, and jointing.

Once the gravity load system has been selected, it is necessary to adapt this system to provide the function of the diaphragm. "Seismic design procedures for diaphragms involve determination of diaphragm design forces (lateral loads); transforming these loads to internal in-plane forces in the diaphragm; and providing sufficient reinforcement at each location to sustain the combination of these in-plane forces with gravity load carried by the floor system." (fib, 2003)

Loads that are considered in the design of diaphragms include wind loads, seismic forces, and soil and fluid loads. Recent changes in the model codes have emphasized the consideration of seismic forces. Concrete buildings with the characteristic of higher dead-load mass are frequently governed by seismic forces even in many regions of relatively low seismic hazard.

Floor and roof diaphragms must be designed to resist the design seismic force, F_{px}, given in ASCE Eq. 12.10-1, 12.10-2 and 12.10-3. Any forces due to offsets in the vertical seismic-force-resisting system or changes in lateral stiffness of the vertical elements must be added to the force determined from ASCE 7-10 Eq. 12.10-1.

In general, collector elements transfer seismic forces from the diaphragm to the vertical elements of the seismic-force-resisting system. Collectors (or drag struts) are required, for example, when shear walls do not extend the full length of the diaphragm in the direction of loading. It is essential that the seismic forces are transferred to the shear walls in order to guarantee a continuous load path. Therefore, ASCE 7-10 Sec. 12.10.2.1 requires that for structures assigned to SDC C or higher, collector elements, splices, and their connections be designed to resist the seismic load effect with overstrength factor in IBC 1605.4, ASCE 7-10 Section 12.4.3.

Due to the poor performance of some cast-in-place concrete topping diaphragms in the 1994 Northridge earthquake, several research projects on precast concrete diaphragms have been conducted. The largest and most extensive project spanned more than a decade. Even before this research was completed, it was recognized that current code prescription for equivalent lateral forces may fail to adequately address dynamic and system effects under high seismic excitation. There is an implicit assumption in the codes that the vertical elements of the seismic force-resisting system will limit system response by yielding. The Response Modification Factor, R, is based on the properties and behavior of the vertical elements of the seismic force-resisting system alone (Nakaki, 2000). The assumption implicit in this approach is that the energy dissipation and post- yield deformation will be controlled by the characteristics of the vertical system. To then be consistent, the diaphragm components of the system need to have both the strength and the deformation capacity to ensure that the primary inelastic mechanism is developed. This means that the diaphragm may need to be designed to remain elastic as it develops and transfers forces to the vertical components. For diaphragms assumed to act rigidly, there is an additional reason to avoid yielding in the floor: This may compromise the load paths that can redistribute loads to stiffer components.

The diaphragm must also continue to provide lateral support in the weak directions of vertical system components and for components not part of the seismic force-resisting system. The code-prescribed diaphragm force provisions do not require that elastic behavior in the diaphragm be generally maintained. Several research reports based on dynamic analysis have identified problems with this approach (Rodriguez et al., 2002 and Fleischman et al., 2002). "Seismic resistant designs for building structures typically prescribe strength levels well below that needed to sustain elastic behavior during a seismic event." (Farrow and Fleischman, 2003a)

An important observation is that the dynamic force distributions produced by the investigated structures can differ significantly from those prescribed in ASCE 7. "For a rigid diaphragm structure following the formation of a base plastic hinge, the instantaneous effective centroid occurs well below the centroid implied by the equivalent lateral force pattern. This downward shift is due to the nature of higher modes in the instantaneous deformation state." (Fleischman et al., 2002) As an interim step in improving design guidance for precast concrete diaphragms, several recommendations were offered in the 7th Edition of the PCI Design Handbook (PCI, 2010).

For structures of low and moderate seismic risk, the dynamic effects are less pronounced. If every floor diaphragm is designed for the force at the uppermost level, additional load factors are not recommended for elastic diaphragm response under the design earthquake.

In regions of high seismic risk, special moment frames of reinforced concrete are sufficiently flexible to limit the direct transfer of ground acceleration to diaphragms at lower levels or the development of significant higher mode effects. Again, if every floor diaphragm is designed for the force at the uppermost level, additional load factors are not recommended.

In regions of high seismic risk, shear-wall buildings are most vulnerable to higher accelerations in diaphragms. As the buildings get taller, the effect becomes more severe. For most precast concrete buildings that are less than 80 ft (24 m) tall, it is recommended to apply a diaphragm load factor of 2.0 to the force at the uppermost level, and to design each floor for that force.

In calculating diaphragm deflection, it is important to make a reasonable estimate of the effective section properties. A detailed analysis that considers the effects of joints, connections, and chord and web reinforcement may be made. A reasonable, but conservative, estimate can be made by taking the effective section as 10% to 15% of the gross section for topped systems and between 5% and 10% of the gross section for untopped and pretopped systems.

Precast concrete floors, topped or untopped, experience the strains of deformation by opening at the joints. These strains cannot be disregarded. Chord reinforcement should be proportioned to limit joint opening to the capacity of the shear reinforcement or connections as well as for strength. Joint reinforcement or flange connections should have sufficient ductility to maintain required capacity while they undergo moderate joint strains.

See Section 2.2 of this report for further discussion of the diaphragm design force level.

RP3-4.3.1 Analysis by Beam Analogy

The most common method of analysis for precast concrete diaphragms models the floor plate as a horizontal beam in the plane of the floor. This beam analogy implies a combination of flexural and shear behaviors. The components of the diaphragm according to the beam analogy are illustrated in Figure 4-1. "In the procedure, it is common practice to design the chord steel to carry the entire calculated in-plane bending moment; and design the web reinforcement to carry the entire in-plane shear across panel joints parallel to the load." (Cleland and Ghosh, 2012) Chord forces are calculated from a flexural analysis using a tension/compression couple between the floor edges. For simple-span diaphragms, the highest chord forces are developed midway between supports. For diaphragms in buildings with multiple vertical systems that create multiple spans, the maximum moments may occur at these supports or at other locations. The shear generally accumulates to its largest magnitude at the supports provided by the vertical systems.

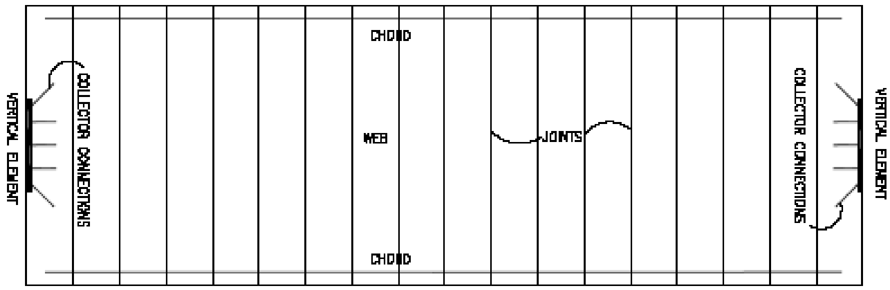


FIGURE 4-1 Precast concrete diaphragm components

In precast floors and roofs without composite topping, the individual components comprising a floor or roof diaphragm must be connected together to transmit the shear and flexure. Joints between precast components, which are parallel to the lateral-force-resisting system, must contain connections to resist the diaphragm shear forces as well as the chord tension/compression forces at the edges of the diaphragm. Joints between the precast components that are perpendicular to the lateral-force-resisting system must include connections that transfer forces across the joint, analogous to horizontal shear (VQ/I) in a composite beam. This is illustrated in Figure 4-2.

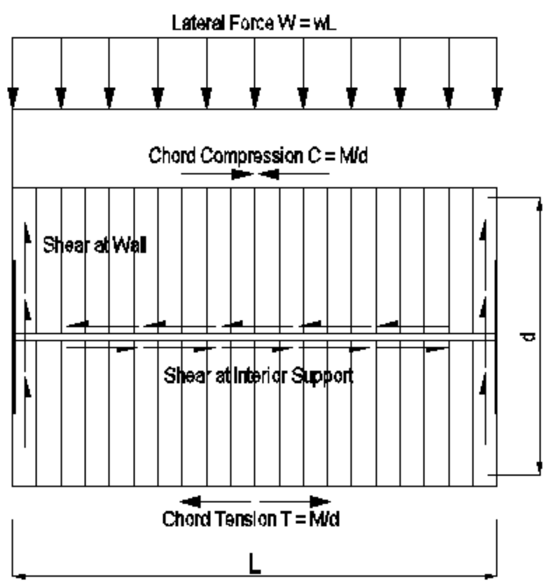


FIGURE 4-2 Diaphragm forces

RP3-4.3.2 Defining Rigid or Flexible Diaphragms by Analysis of the Characteristics

ASCE 7-10 defines diaphragm flexibility (12.3.1.2): "Diaphragms … are permitted to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under the lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system …" (ASCE, 2010) The definition of flexible diaphragm suggests that this classification is local to the story with the diaphragm, and so it is possible for the same configuration to be flexible, semi-rigid and rigid within the same building. For flexible diaphragms, the seismic design story shear is distributed to the various vertical elements based on the area of the diaphragm tributary to each line of resistance.

A rigid diaphragm is one that is capable of lateral force distribution to vertical elements based only on the relative stiffness of those elements. The general assumption is that the deformation within the diaphragm is not significant relative to the deformation of the vertical system. This assumption implies that the diaphragm is capable of carrying loads to extreme points even when there are large differences in stiffness between individual vertical elements. It also implies that the deformation in the diaphragm does not have a significant effect on drift, so that gravity elements remote from the vertical lateral-force-resisting elements are not subject to significantly larger lateral displacements. These assumptions may be unconservative.

The distinction between rigid and flexible diaphragms is important not just for diaphragm design, but also for the design of the entire lateral-force-resisting system.

RP3-4.3.3 Rigid-flexible Considerations for Statically Indeterminate Configurations

A flexible diaphragm is defined as one where the maximum displacement in the diaphragm is twice the maximum displacement of the vertical elements of the lateral force-resisting system. In earlier versions of the load standard, ASCE 7, a rigid diaphragm was defined as one that is not flexible. This simple distinction made analysis simple, but it was not always correct. There are configurations of walls or frames that place such a high demand on the in-plane strength and stiffness of the floor that it cannot be designed to ensure load distribution based on stiffness, and so it must be considered semi-rigid. Semi-rigid diaphragms make the application of the beam analogy questionable.

The behavior of diaphragms as rigid or flexible depends on many factors, including spans, aspect ratio, jointing and connections. It also depends on the relative stiffnesses of the vertical elements of the lateral force-resisting system. Consider a precast structure with shear walls at several lines of lateral support. Figure 4-3 shows a parking structure layout with stiff end walls and interior cruciform walls. The layout includes an interior ramp introducing a large interior discontinuity in the diaphragm.

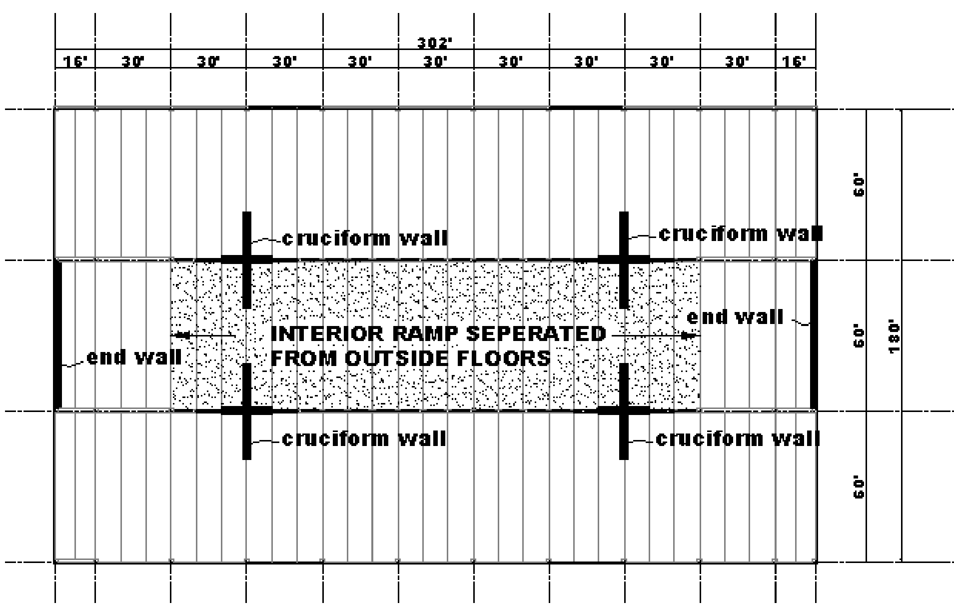


FIGURE 4-3 Illustration of diaphragm layout with multiple walls

It is not uncommon to design the length and width of the walls in such a system to mobilize the resistance to overturning using the direct dead load imposed on the walls. Such a design that uses gravity load resistance to overturning as the primary consideration could result in the interior cruciform walls having significantly less stiffness than the end walls. When the diaphragm is assumed to be rigid, the application of the equivalent lateral floor force must include the effect of accidental torsion, so it would be inconsistent to simply apply the force as a uniform load in evaluating the shear and moment in the diaphragm. The offset of the center of force from the geometric center of the floor plan can be approximated using a combination of uniform and triangular (uniformly varying) load with the resultant acting at the same eccentricity as that was used in the analysis of the vertical elements of the lateral force-resisting system.

Reactions of the diaphragm at the walls can be determined using the results of the lateral distribution. Since the forces at a particular floor or roof level from the distribution of base shear are different from the diaphragm design force, proportional adjustments must be made in the magnitude of the reactions so that they are statically equivalent to the total diaphragm force. The determination of shear and bending in the statically-indeterminate horizontal beams can be made by considering the interior walls as applying counteracting loads rather than acting as supports. Since the interior wall forces in this example would be low, due to their lower relative stiffnesses, the moment diagram is close to one of a uniformly-loaded beam spanning end to end. This is shown as an overlay to the framing on Figure 4-4. For direct comparison of the moments, the variation of uniform load due to the torsional correction has not been included in this illustration. Since the ramp discontinuity separates each bay for most of the length, they share the loads as equal but separate beams. The required chord reinforcement for flexure in such a system is excessive.

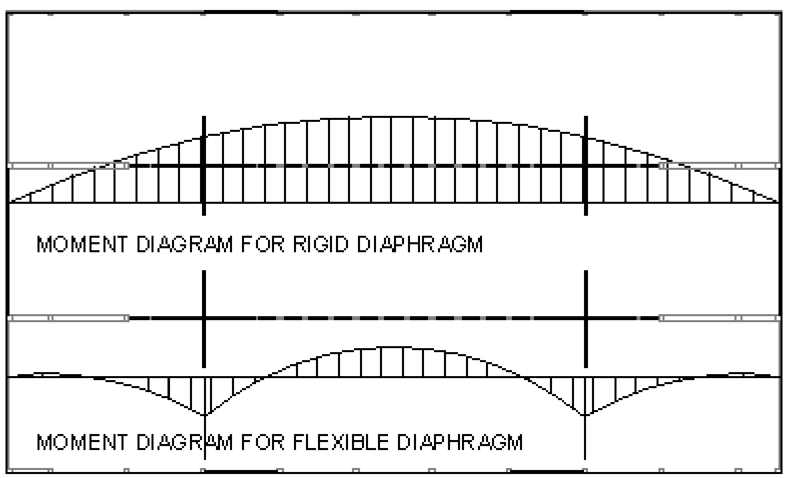


FIGURE 4-4 Comparative flexible and rigid diaphragm moments

For the same system, consider the effects if the diaphragm is flexible. In this case, the equivalent beam is a continuous beam on four supports and the reactions to the walls are support reactions. The moment diagram for this assumption is quite different. The maximum moments for this approach are significantly less than for the rigid diaphragm assumption. The chord reinforcement requirement is also much less.

Unfortunately, diaphragm design is not usually as simple as selecting one or the other approach. There is concern for local overloads. Redistribution of forces from overloaded elements requires some diaphragm rigidity. Flexibility can reduce the demand on the vertical elements and the shear and moments in the diaphragm, but actual rigidity inherent in the floor layout for the structure to serve its function may make this reduction unsafe. If the design includes sufficient chord reinforcement to avoid yielding, the rigid model is safer because redistribution is ensured.

Unless a precast diaphragm clearly qualifies as rigid or flexible, Section 12.3.1 of ASCE 7-10 requires that “the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e. semi-rigid modeling assumption.)" One way to address this requirement is an envelope approach of considering the most severe effects of both rigid and flexible diaphragm assumptions.

Steps in the design method should include: 1) An analysis with rigid diaphragm as a primary assumption; 2) A check on the distribution of the lateral forces and diaphragm forces for a flexible diaphragm assumption; 3) A comparison of the shears and moments resulting from the two approaches; 4) An evaluation of effective section properties and check on diaphragm deformation with respect to drift limits and the permissible deflection of the attached elements; and 5) adjustments in vertical element stiffness and placement to draw the results of analyses based on rigid and flexible diaphragm assumptions closer together and to limit drift to an acceptable magnitude.

Optimum design of diaphragms is interactive with the design of the vertical elements of the lateral force-resisting system. Figure 4-4 shows the results when the shear walls in the plan of Figure 4- 3 are designed only considering the mobilization of dead load resistance to overturning and sliding. In this design, the end walls were taken as continuous horizontal walls from face to face of the columns. With the flexible diaphragm analysis, the end walls receive much less lateral force and the interior cruciform walls receive much more. When the end wall design is changed by substituting separate 10 ft long loadbearing walls at the end bay column locations, the relative stiffness of the walls on these end lines is much less than the interior cruciform walls. When this system is analyzed using a rigid diaphragm assumption, the shear and moment diagrams are much different, as seen in Figure 4-5. This analysis indicates that the modification of the shear wall configuration brings the rigid diaphragm solution closer to the flexible diaphragm assumption.

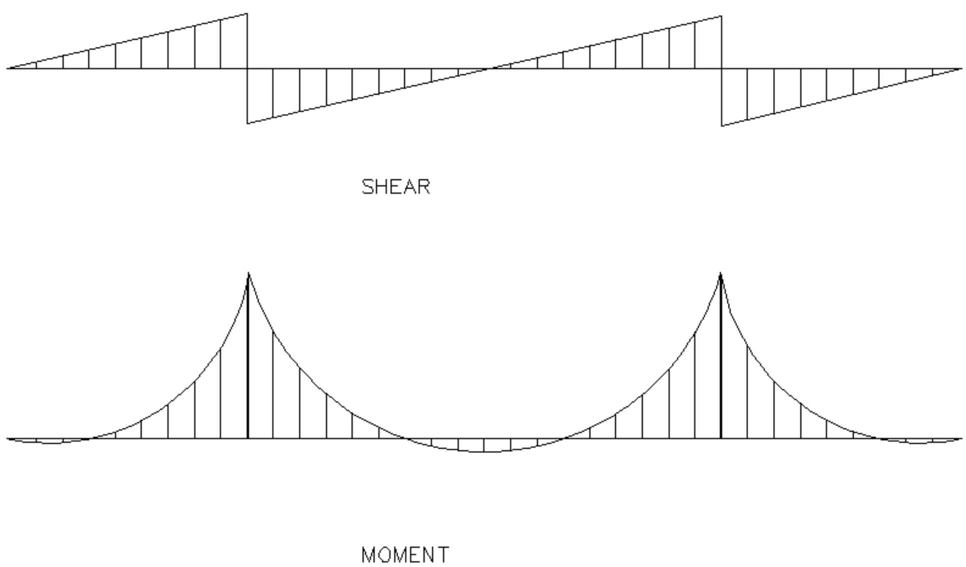


FIGURE 4-5 Shear and moment diagrams for reduced end walls

To bring the rigid and flexible diaphragm solutions even closer together, the stiffness of the ends walls should be increased. To increase the end wall stiffness, the total wall length is increased to 16 ft. With the stiffer walls, the rigid diaphragm analysis shear and moment diagrams are shown in Figure 4-6. With this solution, the results from the flexible assumption and the rigid assumption are nearly identical.

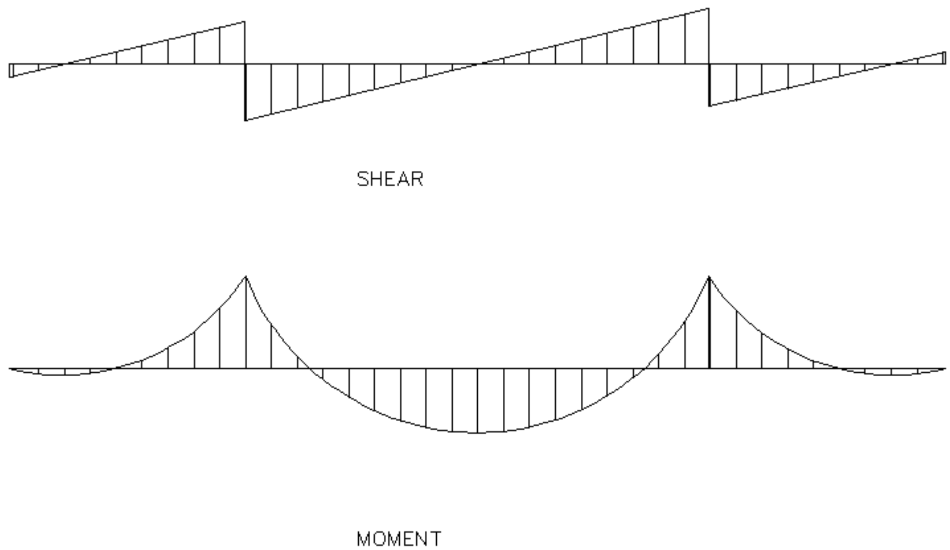


FIGURE 4-6 Shear and moment diagram for balanced shear walls

By example, it is shown that diaphragm analysis and design should not be considered separately from the overall design of the lateral force-resisting system.

RP3-4.3.4 Flexible Diaphragms in Common Precast Concrete Parking Structure Configurations

In their research, Farrow and Fleischman (2003b) investigated common layouts of precast parking garages. The characteristics of these structures include long spans between seismic force-resisting elements and large openings or discontinuities created by separation of floor plates for the traffic ramps. They investigated the effects of diaphragm and lateral element layout, diaphragm dimensions, mechanical connector strengths and cast-in-place concrete topping on diaphragm deformation. Although even topped diaphragms are actually cracked by the effect of joint tooling and shrinkage that accumulates at the joints, the baseline for deformation comparisons was a monolithic diaphragm of identical thickness. For a three-bay parking structure 320 ft (97.5 m) long and 186 ft (56.7 m) wide with a central ramp and shear walls at the ends, the monolithic diaphragm deflection was 2 ½ times shear wall displacement at the roof level. When the topped diaphragm was considered cracked, the deflection increased to about 3 ¼ times the shear wall displacement. When shear deformation equal to 20% of the flexural deformation was considered, the deflection grew to over 3 ½ times the shear wall displacement. Deflection in the untopped diaphragm with cast-in-place pour strips was nearly twice that of the topped system, at 7 ½ times the shear wall deformation. Other configurations of shear walls that included interior placement at the ends of the ramp reduced the diaphragm displacements but increased the force effects at some critical locations. Even with sub-diaphragm aspect ratios less than 2, these systems were all fully flexible.

Although precast concrete diaphragms have traditionally been taken as rigid as diaphragms composed of concrete slabs, the jointed nature of precast concrete construction significantly reduces their stiffness relative to monolithic concrete. This reinforces the importance of design of the overall seismic force-resisting system so that the difference between rigid and flexible modeling is small.

RP3-4.3.5 Strut and Tie Modeling

Appendix A of ACI 318-11 provides an alternate method for analysis of concrete structures that have regions where the plane sections assumption for linear distribution of strain is not valid. This method has been applied to deep beams and other members with regions of large discontinuities. Two volumes of examples using strut-and-tie models (STM), SP-208 (ACI, 2002) and SP-273 (ACI, 2010), have been published by ACI, but none of these has directly considered the modeling of diaphragms. Since diaphragms may, by the beam analogy, be considered deep beams with large discontinuities, strut-and-tie modeling can be adapted to diaphragms. The Commentary to ACI 318-11 Section 21.11.8 states "Strut-and-Tie models are potentially useful for designing diaphragms with openings."

The STM requires some idea of the lateral distribution to the vertical elements so that the model can be developed with struts parallel to the orientation of initial cracking. "A truss formulated in this manner also will make the most efficient use of the concrete because the ultimate mechanism does not require reorientation of the struts." (Fu, 2001)

The development of strut-and-tie models for precast concrete diaphragms must include consideration of the jointing and the load paths created by connections. In diaphragms formed with cast-in-place concrete topping, the placement of reinforcement is guided by the locations of tension ties in the model.

RP3-4.3.6 Finite Element Analysis

Although traditionally perceived as a research tool rather than as a design aid, Finite Element Analysis (FEA) has more recently found a place on the desktop computers in many engineering offices. In some cases, the power and simplicity of finite element modeling may make it a practical and useful alternative when a diaphragm design problem is not simple.

The key to successful diaphragm analysis using finite elements is in the modeling. When the focus of the analysis is the in-plane behavior, shell elements without transverse loads can be used to create the model. With precast systems, it is important to adequately define and model the joints between adjacent precast components. This can be done by developing a layout that represents the flange of only one unit. The size or spacing of the elements should match flange connection spacing, so the nodes can be used as connection sites, as illustrated in Figure 4-7. The model is then copied or arrayed with spacing that leaves joints between the units. At beam lines, the depth of the beam might be represented by increasing the thickness of the shell element. A view of part of one sample layout is shown in Figure 4-8.

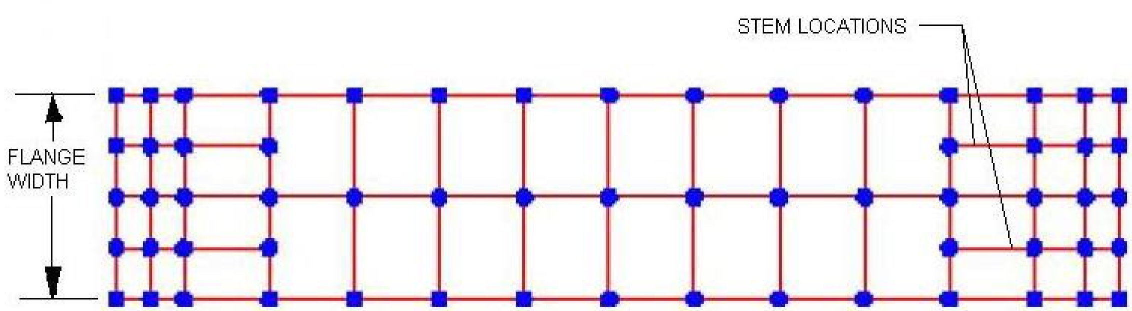


FIGURE 4-7 Detailed view of finite element model of a double-tee flange

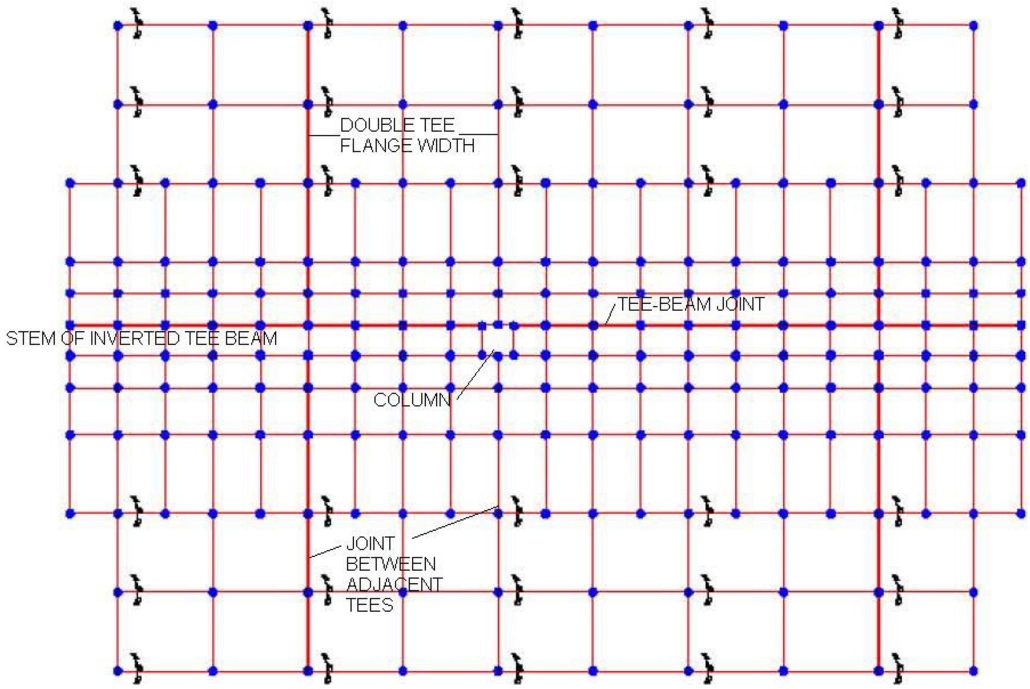


FIGURE 4-8 Finite element mesh for double-tees supported on an inverted T-beam

It is equally important to add connections between the units, which adequately model the connection characteristics. Some FEA programs provide member releases that can reduce or eliminate tension forces from beam elements used to model the shear connectors. Some programs also permit the definition of non-linear springs so that test deformations can be modeled. Nonlinear elements such as compression truss can also be used to model concrete pour strips that transfer forces in compression but not in tension. In applying any beam or spring elements to a model with plate elements, it is important to consider the continuity of force transfer that may not be captured by simple node-to-node attachment. Connections with reinforcement or anchorage to concrete develop into the body of the component or topping, and the model should include the extension of this anchorage. The degrees of freedom in some plate elements may not properly express moment continuity at the edge of a plate. In some cases, it may be appropriate to introduce plate elements with more degrees of freedom (“drilling degrees of freedom”) to permit moment and torsional continuity when these are needed to properly characterize the connection behavior. When possible, it is best to calibrate the model techniques to known experimental data.

Simple modeling of planar behavior, as indicated above, can provide significant results and insight into areas with high load concentrations or deformations. In some cases, the prominence of shear deformation as a large component of deflection is evident. It may also be possible to identify deformation patterns that are more characteristic of tied arch behavior than of beam bending. Chord forces and connection forces are the primary results that can be used in design.

RP3-4.3.7 Precast Concrete Diaphragm Construction Details

Since the diaphragm is composed of many separate gravity load-resisting units, it is necessary to add connections between these for lateral resistance and continuity. The types of connections used to connect precast units together to form diaphragms vary, depending upon the seismic design category, construction method, the required connection strength, strain capacity to accommodate expected joint movement, and the preference of the precast supplier manufacturing and erecting the precast units.

Construction methods for precast diaphragms were described above and include composite topped diaphragm construction, non-composite topped diaphragm construction, or pretopped (untopped) diaphragm construction.

Composite topped construction uses a cast-in-place concrete slab placed over the precast floor units with sufficient surface roughness to transfer horizontal shear, so that the flexural strength of the component is enhanced by the added structural depth of the topping. This approach provides the advantage of both strengthening the gravity system and completing the diaphragm. The primary connections in this method come from the reinforcement placed in the topping. Commonly continuous chord reinforcement is laid at the perimeter to resist chord tension forces and welded wire reinforcement is used for shear transfer across the interior joints. Even though the topping may be continuous, shrinkage cracks will tend to form over the precast joints and these systems will express most of their deformation in the form of strains at these joints. When used as floors of parking structures, the topping is tooled above the joints to control the regularity of the cracking and to provide a location for sealant. Shear reinforcement can be determined by shear-friction analysis, as in Section 11.6.4 of ACI 318-11.

For seismic design categories D, E and F, the shear strength of topping slab diaphragms is determined by Equation 21-11 in Section 21.11.9.3 of ACI 318-11.

$$Vn = Avf\, fy\, \mu$$

where Avf = the area of the shear friction reinforcement within the topping slab, including both distributed and boundary reinforcement and fy = yield strength of shear reinforcement, which may not exceed 60,000 psi for bars and plain welded wire reinforcement or 80,000 psi for deformed welded wire reinforcement. The coefficient of friction, μ, is 1.0λ, where λ is the lightweight concrete factor.

Although it has been the practice in some parts of the United States to omit mechanical connections between the precast units, it is common practice in most regions to include some mechanical connections with double tee construction. These connections provide for erection stability and safety, lend a degree of additional strength and redundancy, and their inclusion is recommended. Connections between members often serve functions in addition to the transfer of lateral loads. For example, weld plates in flanged members are often used to adjust differential camber. Grout keys between hollow-core planks may be utilized in the joints of deeper sections to distribute concentrated vertical loads.

Pretopped (untopped) construction is often favored for double tee parking garages and hollow-core framing in SDC A, B and C. In parking garage construction, the primary advantages are plant-controlled quality and concrete durability and the reduction of field operations to complete the structure.

With untopped double tees, chord reinforcement may be provided as continuous reinforcing bars laid in narrow sections of cast-in-place concrete topping (pour strips) at the ends of the tees, or welded connections that link reinforcing bars embedded across the tee ends, making them continuous.

For double tee untopped systems, the flange connectors are designed to transfer the diaphragm shear as well as to assist in the vertical alignment of the flanges. For many years, plant-fabricated shear connections have been successfully used in this type of construction. Figure 4-9 shows a typical configuration of one of these connectors. Note that in this connection, the weld slug is intentionally offset from the anchoring bars. The intent of this detail is to provide for the transfer of shear parallel to the joint but to afford some flexibility in the connection to shed forces perpendicular to the joint. The actual design or execution of some of these details may not attain this flexibility. Tests on some of these connections have indicated a significant loss of shear strength when loaded in conjunction with tension, and poor ductility. (Pincherira et al., 1998)

Commercial inserts have also been used for flange connectors for shear transfer. An example of one connector is shown in Figure 4-10. This P-11 embed is a “bent wing” that is stamped into a blunt “V” shape with legs projecting back into the tee flange and presenting a flat weld surface parallel with the flange. Figure 4-11 shows a connection made with this insert. The insert can be oriented as shown in the figure or turned to provide a sloped pocket for the weld slug. The weld length is limited to a small part of the parallel surfaces. This connection has only limited tension strength and the eccentric location of the weld allows some plate deformation that offers some tension compliance. With the bent wings at 45 degrees, however, tension across the joint tends to pull on the thin sections of concrete that form between the connector and the edge of the flange at the ends of the parallel surfaces. This can result in cracking or spalling in the flange and may compromise the sealant system.

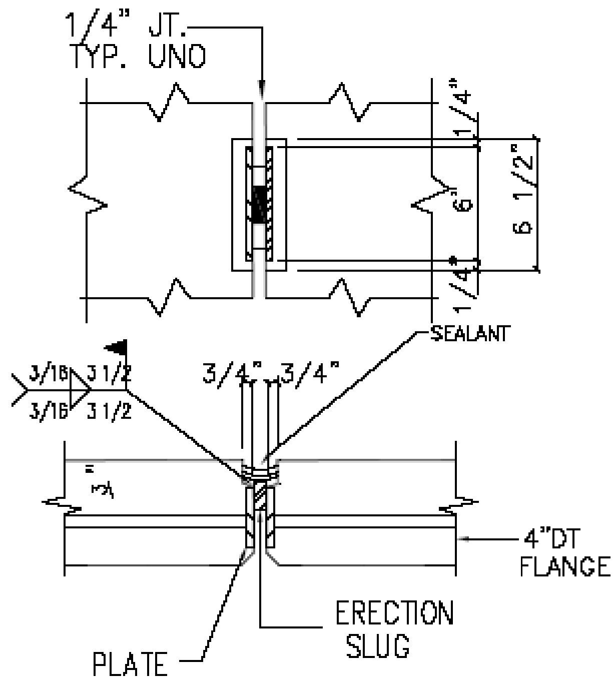


FIGURE 4-9 Flange connector

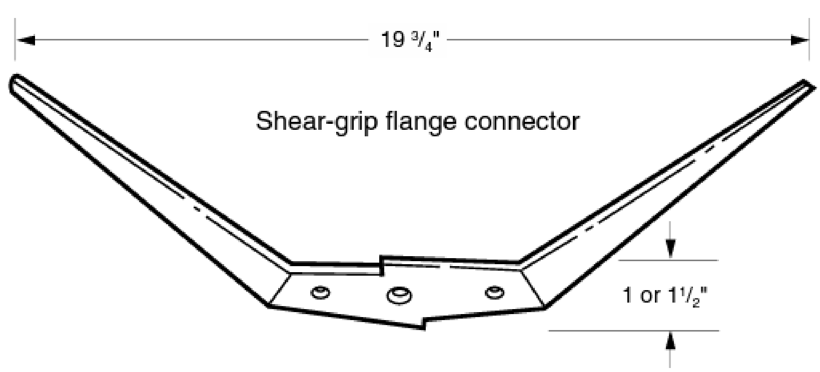


FIGURE 4-10 Commercial flange connection insert

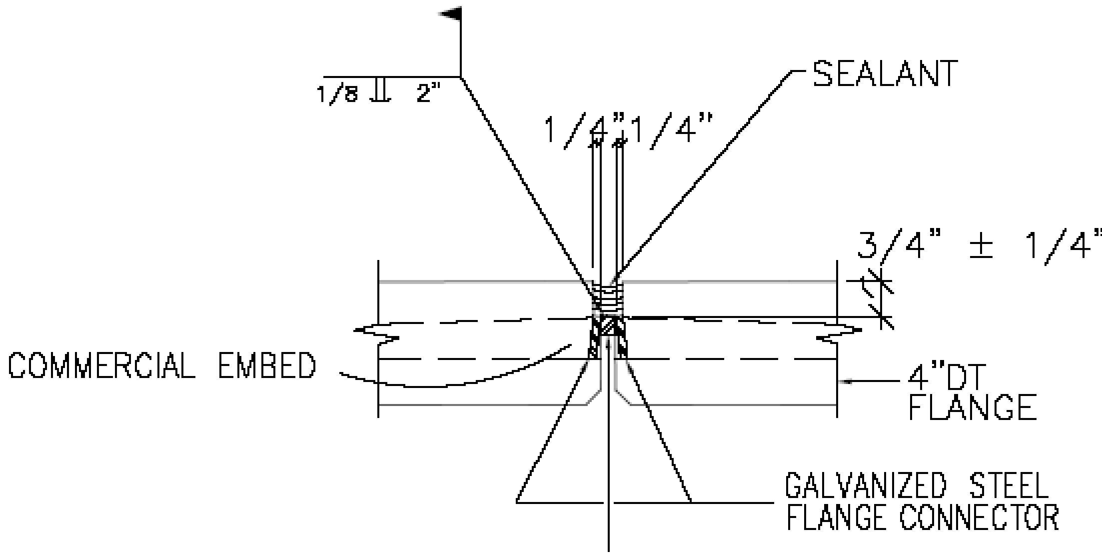


FIGURE 4-11 Bent-wing flange connector

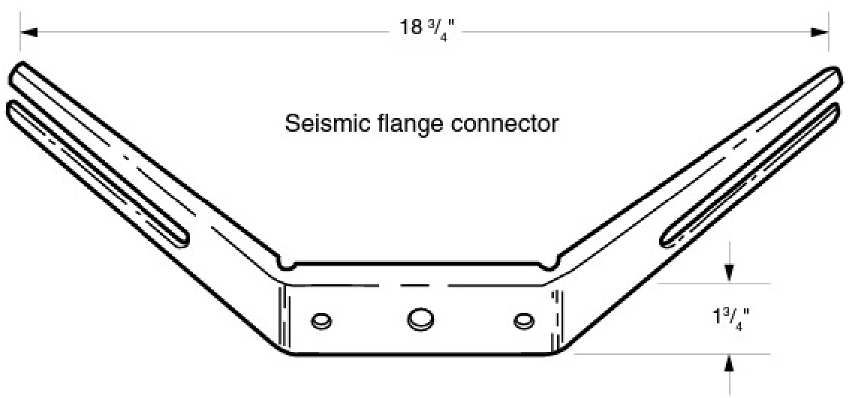


FIGURE 4-12 Seismic bent-wing flange connector

FIGURE 4-13 Vector connector

Improved commercial inserts have been developed. A version of the P-11 intended for seismic application is shown in Figure 4-12. Another improved commercial insert is shown in Figure 4-13. A connection detail using this embed is shown in Figure 4-14. Tests conducted on this connection show improved behavior. After the initial bond with the surrounding concrete is broken, the shear resistance decreases and the displacements increase, but in a ductile fashion. Seismic application of this connection should consider the shear strength after loss of the initial concrete confinement. This “connector is compliant and ductile under tensile load, if the joint slug is welded near the top of the face plate.” (Oliva, 2000)

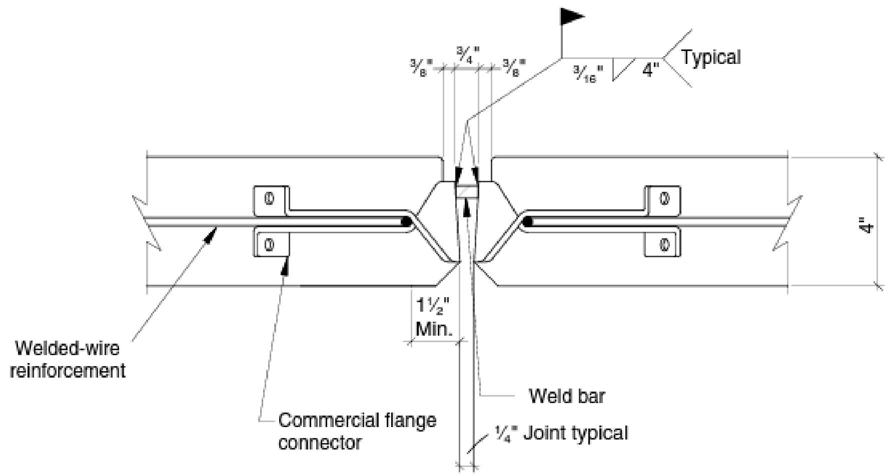


FIGURE 4-14 Vector connection detail

Figure 4-15 shows a section at a perimeter loadbearing spandrel beam with a cast-in-place pour strip with a thickened wash for cover ad development of the chord reinforcement. Figure 4-16 shows a section at an interior inverted tee beam. The pour strip here is also raised, for cover and for drainage.

The transverse steel (#4 @ 12 in. or 13-mm dia. @ 300 mm) crossing the pour strip and the joints between the double tees and the beam is also important for providing shear transfer across the seam in the diaphragm that is created by the discontinuity. Similar details may also be appropriate when the diaphragm is completed with a full cast-in-place concrete topping.

With systems using untopped hollow-core, the chord is frequently made from continuous reinforcement placed in a cast-in-place strip beyond the ends of the plank that also provides shear-friction steel to hold the joints together and permits shear transfer through the grouted joint. Hollow core plank are fabricated with grout keys and are connected by grouting the joints. For members connected by grout keys, a conservative value of 80 psi (55 kPa) can be used for the design shear strength of the grouted key.

Connections that transfer shear from the diaphragm to the shear walls or other vertical elements of the seismic force-resisting system carry the diaphragm reactions. For rigid diaphragms, the reaction forces need to be determined with consideration of the maximum effects of torsion in the plane. In untopped systems, areas adjacent to the vertical elements of the seismic force-resisting system may also be designed with pour strips to permit the use of embedded reinforcement as part of the connection. Relative movements imposed on the diaphragm by the vertical systems from effects like elongations in the plastic hinge zones of frame members or rocking of walls must be accommodated by these connections (fib, 2003).

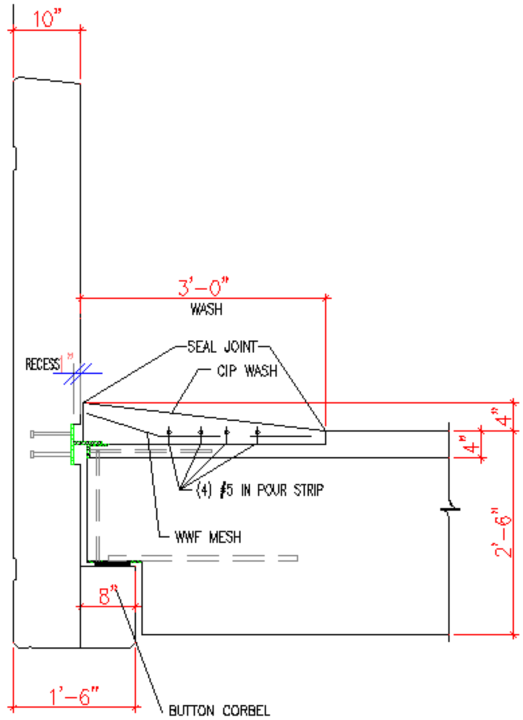


FIGURE 4-15 Perimeter section with pour strip

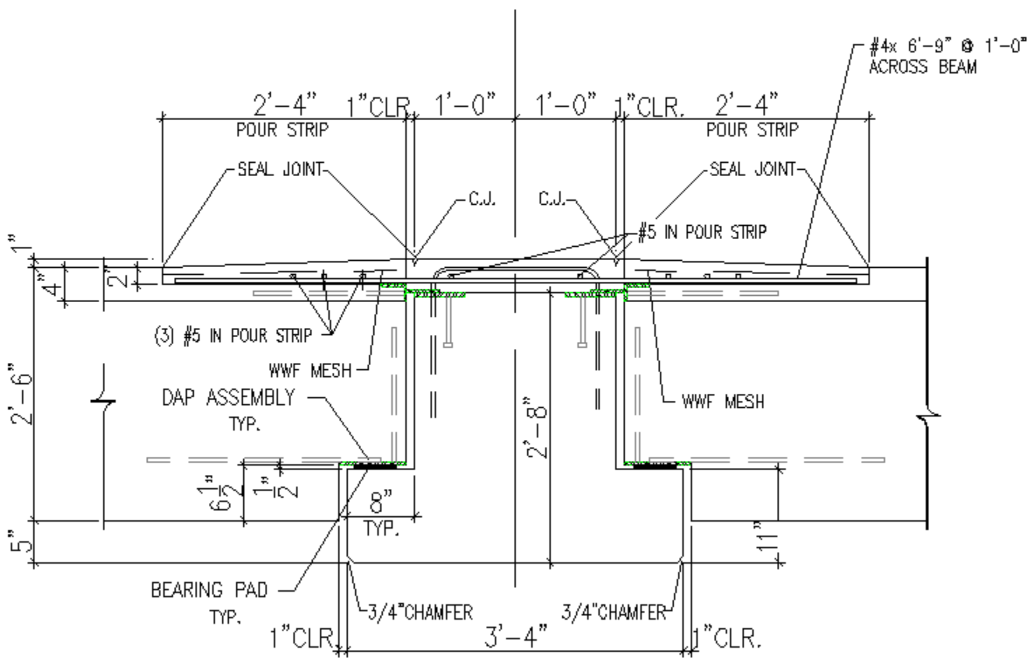


FIGURE 4-16 Interior beam section with pour strip

RP3-4.3.8 Past Performance

In general, there have not been reports of widespread problems with precast concrete diaphragm performance in earthquakes. There were, however, some notable precast concrete diaphragm failures during the 1994 Northridge earthquake. Iverson and Hawkins (1994) reported their observations of extensive damage in some large-plan parking structures. Damaged garages included the Northridge Fashion Center, California State University, Northridge, and the Glendale Civic Center, which had diaphragm damage or failure. They reported that "One characteristic feature of several of the prefabricated and site precast garages that collapsed was an apparent lack of adequate ties connecting precast floor elements to one another and to the lateral load resisting system."

To a significant degree, the "lack of adequate ties" can be attributed to detailing practices that were used in the precast concrete garages. The diaphragms depended on the cast-in-place concrete topping and the reinforcing in that topping alone for collection and transfer of forces. The practice in most of the United States outside California is to provide additional mechanical connections between the flanges of the double tees and the double tees and their supporting beams. The primary purpose of these additional connections is for stability during erection and for alignment of double tee flanges when there is differential camber between members. These connections tie the diaphragm together during construction, but they are not removed or abandoned in the final construction even though there is no formal recognition to their contribution to strength in ACI 318. They provide for some redundancy and some ductility in the diaphragm. In Northridge, as in much of California, these connections were not used because only the reinforcing in the topping slab can be considered for strength.

The reinforcement detailing used in the cast-in-place topping slabs was not effective. The shear reinforcing in the diaphragm was welded wire reinforcement. These wire are drawn to increase the yield strength from about 42 ksi (290 MPa) to 65 ksi (450 MPa) or higher. This process of cold- working the wire through dies significantly reduces the strain capacity of the steel. The wire is then assembled into mats with closely spaced cross wires that develop the strength rather than depending on bond to concrete for development. It was learned from the earthquake that the closely spaced cross wires of high strength wire fabric is not compatible with the behavior of the precast concrete system. Although the topping slab may be continuous, the underlying precast system has regularly spaced joints between the double tees, and additional joints at the interior beams. These precast joints act as control joints by providing planes of weakness. It was learned long ago that it is necessary to tool control joints in toping slabs above the precast joints to avoid irregular cracking in the topping slab that generally follows the precast joints. The deformation in the diaphragm is almost entirely concentrated in the movement of these joints. Even the temperature change movements concentrate at the joints. A temperature change of only 50^{o} F (28^{o} C) can cause concrete between joints spaced at 12 ft (3.66 m) to change length by 0.043 in. (1 mm) (based on a coefficient of thermal expansion of 6 x 10^{-6} in./^{o}F or 2.74 x 10^{-4} mm./^{o}C). If welded wire reinforcement with wire spacing is used, and the thermal movement concentrates in one wire space across the joint, the resulting strain would be greater than 0.007 in./in. (mm/mm), corresponding to a strain of more than twice the yield strain. These high strains also develop when the diaphragm is deformed by lateral earthquake movement unless chord reinforcement prevents joint opening. Another detailing practice common in California is the use of unstressed prestressing strand to provide chord reinforcement. Although this material has a very high yield strength, that strength cannot be attained without high strains. Even when prestressing strand is considered to have a lower tensile strength when proportioned for this use without prestressing, there remains the question of bond and stiffness that are also needed to prevent joint opening. There were reports of chord strand buckling and the material is less effective than conventional reinforcing in restraining joint opening. Failure in the welded wire related to joint opening was noted in the Northridge collapses. Although subsequent research reveals that the inertial forces in diaphragms are greater than considered for design in ASCE 7-10, the contribution of detailing practice to these failures cannot be discounted.

There were two significant results from the damage observed in the Northridge earthquake. First, ACI 318-99 included a new section of provisions for diaphragms that separated diaphragm design from structural walls and included changes in minimum topping thickness and reinforcement detailing. In high seismic applications, it is no longer permitted to use welded wire reinforcement with cross wire spacing less than 10 in. (250 mm), to provide greater strain capacity. The second result was the initiation of extensive research into the behavior of precast concrete diaphragms. The initial research ultimately led to a comprehensive program sponsored by PCI, NSF, and the Charles Pankow Foundation to develop Diaphragm Seismic Design Methodology (DSDM).

RP3-4.3.9 Diaphragm Seismic Design Methodology (DSDM)

The initial recommendations and reports of the DSDM research program were submitted for review by the steering committee and the PCI Research & Development committee in 2012.

Among the results of this research are a new concern with diaphragm flexibility, a requirement for sufficient design forces to ensure that inelastic behavior occurs in the vertical elements, and ductility demand in the diaphragm for elastic design or for performance-based design. An important observation is that the dynamic force distributions produced by the structures investigated can differ significantly from those provided by the equivalent lateral force (ELF) patterns prescribed by the Codes "For a rigid diaphragm structure following the formation of a base plastic hinge, the instantaneous effective centroids occur well below the centroid implied by the ELF pattern. This downward shift is due to the nature of higher modes in the instantaneous deformation state." The methodology also provides a realistic assessment of diaphragm deformation considering tension, shear and flexure and behavior at joints.

In general, the methodology requires an increase in diaphragm design forces. In part, the increase considers the effects of flexibility on response and the effects of flexibility on ductility demand. More importantly, it includes the effects of higher modes of response of the structure that are not diminished by the inelastic response to the first mode. The DSDM research and recommendations were developed in the context of ASCE 7 diaphragm design force level and detailing requirements. As the work of the BSSC Issue Team (IT) 6 on diaphragms progressed to a comprehensive recommendation for change in the code-prescribed design force level, it was recognized that the DSDM recommendations should also reflect the new diaphragm design force level in Part 1 of the 2015 NEHRP Provisions. As a result, the recommendations of the DSDM report for the design of precast concrete diaphragms have been adjusted to that design force level, which is entirely consistent with the research.

The DSDM project focused on three basic objectives: 1) Diaphragm design force levels; 2) Diaphragm deformations for precast concrete systems; and 3) Precast concrete diaphragm reinforcing and connection details required to sustain the forces at the expected deformations. The research demonstrated viable systems applicable to high seismic design categories (PCI, 2012).

"The research project, led by the University of Arizona, accomplished these goals through an integrated experimental-analytical research program using the NSF Network for Earthquake Engineering Simulation (NEES) Sites at Lehigh University and the University of California, San Diego (UCSD).

"The key outcomes of the research are:

The diaphragm design force levels required to keep diaphragms elastic in the design earthquake.

The relationship between precast diaphragm strength and anticipated diaphragm reinforcement deformation demands for different diaphragm geometries and seismic hazard levels.

The required increase in diaphragm shear strength, with respect to diaphragm flexure strength, to prevent undesirable high inelastic shear deformations in precast diaphragms.

The key characteristics of several typical precast diaphragm connectors and joint reinforcement under cyclic tension and shear, including stiffness, strength and reliable deformation capacity.

New diaphragm reinforcement concepts that provide improved cyclic performance.

The research also provided new information on: (1) The response of precast diaphragm connectors and joint reinforcement under combined tension and shear; (2) The seismic diaphragm force profiles that occur in multi-story structures; (3) The force paths that develop in precast floor systems, including parking structures and office buildings; (4) the inelastic deformation patterns that develop in the diaphragm reinforcement, including concentrated demands at columns lines. This information informed the design methodology.

The key research outcomes listed above are embodied in the design procedure as follows:

Diaphragm design force amplification factors (Ψ) that are calibrated to different performance targets for precast diaphragms.

Diaphragm connector and joint reinforcement classifications (Low, Moderate and High Deformability Elements or LDE, MDE, HDE) based on inelastic deformation capacity.

Diaphragm shear overstrength factors (Ωv) to protect the diaphragm from undesirable shear mechanisms." (PCI, 2012)

Under the original scope, diaphragm design factors for force level adjustment and diaphragm shear overstrength factors were developed and calibrated to diaphragm design forces calculated from the equivalent lateral force method in ASCE 7-10. With the development of the Modifications to ASCE 7-10 Section 12.10 in Part 1 of the 2015 NEHRP Provisions, based on the Restrepo - Rodriquez formula that includes first mode overstrength and higher mode contribution factors, diaphragm response modifications factors have been derived that are calibrated to the research. All these factors are related to a three-tier design approach which is demand and performance- based.

From the DSDM Part 1 report, these options are described as follows:

An Elastic Design Option, where the diaphragm is designed with the highest set of amplification factors, calibrated to keep the diaphragm elastic, not only for the design earthquake, but also in a rare maximum event, but, in exchange for the high diaphragm design force, permits the designer to detail the diaphragm with ordinary (LDE) connectors or joint reinforcement that need not meet any deformation requirements.

This option is limited in its use through the introduction of Diaphragm Seismic Demand Levels, which are based on building height, diaphragm geometry, and seismic hazard level, and preclude the use of the Elastic option for High diaphragm seismic demand.

A Basic Design Option, in which the diaphragm is designed with amplification factors calibrated to keep the diaphragm elastic for the design earthquake, which are therefore lower than the amplification factors for the Elastic Design Option, but requires MDE diaphragm connectors or joint reinforcement, whose classification provide an inelastic deformation capacity sufficient to survive the anticipated deformation demands in a rare maximum event.

This option, and the next, require the use of the diaphragm shear overstrength factor to assure that a non-ductile shear failure does not occur prior to the reinforcement reaching its intended inelastic target deformation. Note that inelastic deformation is associated with joint opening due to diaphragm flexure, not joint sliding deformation due to shear.

A Reduced Design Option, in which the diaphragm is designed for the lowest level of amplification factors. Because these factors are lower than the amplification factors in the Basic Design Option, some yielding in the diaphragm is anticipated in the design earthquake. These factors have been calibrated to keep the diaphragm inelastic deformation demands within the allowable deformation capacity for the highest classification of precast diaphragm reinforcement, termed HDE details.

The methodology promotes the design of the precast reinforcement at the joint level, in which all the reinforcement across the joint is accounted together to carry the design forces, whether shear, flexure, collector or some combination.

The steps in the proposed design procedure derived from the DSDM Report, Part 1 are reproduced here, with modifications noted for the Restrepo – Rodriquez design force proposal as appropriate.

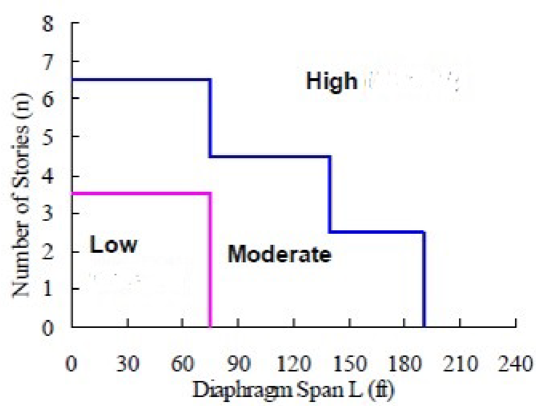
Current code forces are based on the first mode while maximum diaphragm forces are related to higher mode effects, primarily the second mode (Rodriguez et al., 2002). One issue with scaling the diaphragm design forces to current code is that the relationship between the first and second mode design spectrum ordinates changes with building period. This fact is particularly apparent when considering tall structures. These tall structure, however, are not common in current precast/prestressed construction.

RP3-4.4 Seismic Design Procedure for Precast Concrete Diaphragms

Step 1: Determine diaphragm seismic demand level

1. Three diaphragm seismic demand levels are defined: Low, Moderate, and High
2. Diaphragm seismic demand level is based on SDC, number of stories, n, diaphragm span, L and aspect ratio, AR:

    − Determine SDC from risk category of structure and SDS and SD1 at the site per ASCE 7-10 Section 11.6
    − For structures assigned to SDC B or C, the seismic demand level is low. For structures assigned to SDC D, E or F, the seismic demand level shall be determined in accordance with the figure below.



Note 1: If AR>2.5 and the diaphragm seismic demand is Low according to Figure above, the diaphragm seismic demand level shall be changed from *Low to Moderate*.
Note 2: If AR<1.5 and the diaphragm seismic demand is High according to Figure above, the diaphragm seismic demand level shall be changed from *High to Moderate.*

3. Diaphragm span on a floor level is defined as the larger value of:

    − maximum interior distance between two LFRS elements
    − twice the exterior distance between the outer LFRS element and the building's free edge

4. Diaphragm span for the structure, L, is selected as the maximum diaphragm span on any floor in the structure in any direction.
5. Diaphragm aspect ratio, AR is defined as the diaphragm span-to-depth ratio, using the diaphragm span determined in steps (3), (4) above. Depth is defined as the floor diaphragm dimension,

perpendicular to diaphragm span, between the pair of adjacent chord lines for the diaphragm or portion of diaphragm.

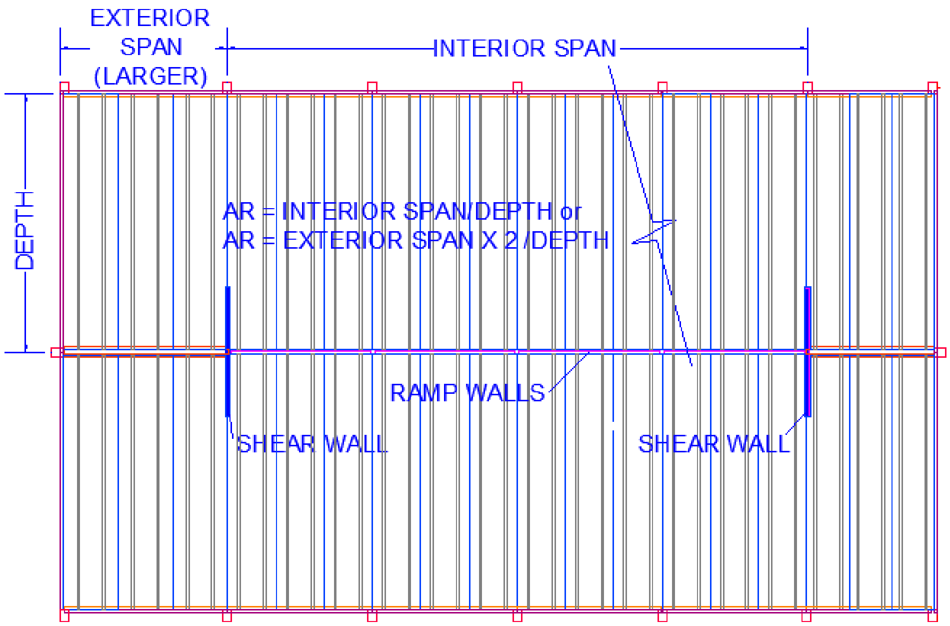


FIGURE 4-17 Aspect ratio (AR) from layout

Step 2: Select diaphragm design option and appropriate diaphragm connector or joint reinforcement classification

1. Three diaphragm design options are defined: Elastic, Basic and Reduced.
2. Diaphragm design option applicability is based on diaphragm seismic demand level, see Table 4-1.
3. Three diaphragm reinforcement classifications are defined: High-, Moderate-, and Low-deformability elements.
4. Diaphragm connection applicability is based on diaphragm design option, see Table 4-2. The design option selected shall be consistent with the lowest connection classification.

Table 4-1 Diaphragm design option

Design Option	Diaphragm Seismic Demand level Low	Diaphragm Seismic Demand level Moderate	Diaphragm Seismic Demand level High
Elastic	Recommended	With Penalty*	Not Allowed
Basic	Alternative	Recommended	With Penalty*
Reduced	Alternative	Alternative	Recommended

*Diaphragm design force shall be increased by 15%.

Table 4-2 Required diaphragm reinforcement classification

Design Option	Diaphragm Seismic Demand level Low	Diaphragm Seismic Demand level Moderate	Diaphragm Seismic Demand level High
Elastic	Recommended	Allowable	Allowable
Basic	Not allowed	Recommended	Allowable
Reduced	Not allowed	Not allowed	Recommended

A cast-in-place equivalent diaphragm is a topped precast diaphragm with the topping designed to act as the diaphragm according to ACI 318-11 section 21.11.5 or a pretopped precast diaphragm where all chords, collectors, and joints between precast elements are constructed of field-cast reinforced concrete with details consistent with the requirements of ACI 318-11.

USER NOTES:

- An Elastic Design Option (EDO):
  - The EDO targets elastic diaphragm behavior in the maximum considered earthquake (MCE).
  - The diaphragm force reduction factor, Rs, is the lowest for the EDO.
  - The EDO allows the use of low-deformability (LDE) connectors or better.
- The Basic Design Option (BDO):
  - The BDO targets elastic diaphragm design in the design earthquake (DE).
  - The diaphragm force reduction factor, RS, has a value between those for the EDO and the RDO.
  - The BDO requires the use of moderate deformability (MDE) connectors or better.
- A Reduced Design Option (RDO):
  - The RDO permits diaphragm yielding in the DE.
  - The diaphragm force reduction factor, RS, is the highest for the RDO.
  - The RDO requires the use of high deformability (HDE) connectors or joint reinforcement.
- High deformability element (HDE): An element that demonstrates a reliable and stable maximum joint opening deformation capacity of greater than 0.6 in.
- Moderate deformability element (MDE): An element that demonstrates a reliable and stable maximum joint opening capacity of between 0.3 in. and 0.6 in.
- Low deformability element (LDE): An element that does not meet moderate or high deformability element requirements.
- Classification of a given diaphragm connector or joint reinforcement element is determined through testing of individual elements following the cyclic testing protocols in Section 14.2.4 (see Modifications to ASCE 7-10 Section 14.2 in the 2015 NEHRP Provisions.

Step 3: Determine diaphragm design force and diaphragm internal forces.

Determine diaphragm design force at each level as:

$F_{px} = C_{px} w_{px} / R_s$ (See Section 14.2.4 in Part 1 of the 2015 NEHRP Provisions)

where:

w_{px} is the weight tributary to floor at level x F_{px} is the diaphragm design force at level x

C_{px} − See Section 14.2.4 in Part 1 of the 2015 NEHRP Provisions

R_s = 0.7 for EDO

= 1.0 for BDO

= 1.4 for RDO

For cast-in-place equivalent diaphragms, R_s is the same as for cast-in-place diaphragms.

For design of collectors, amplify the design force by 1.5 R_S. Diaphragm shear overstrength factor, Ω_V = 1.4 R_S

Determine diaphragm internal forces

Reference ACI 318-14 Diaphragm Chapter – Section 12.4.2.4:

Diaphragm in-plane design moments, shears, and axial forces shall be consistent with requirements of equilibrium and with design boundary conditions. It shall be permitted to determine design moments, shears, and axial forces in accordance with one of (a) through (e), as appropriate:

a. using a rigid diaphragm model for cases in which the diaphragm can be idealized as rigid;
b. using a flexible diaphragm model for cases in which the diaphragm can be idealized as
c. using a bounding analysis in which the design values are the envelope of values obtained by assuming upper bound and lower bound in-plane stiffnesses for the diaphragm in two, or more, separate analyses;
d. using a finite element model considering diaphragm flexibility;
e. using a strut-and-tie model in accordance with the provisions of 18.5.

USER NOTES:

- The rigid diaphragm assumption has historically been applied to precast concrete diaphragms with aspect ratios less than 3 and without major openings. With a rigid diaphragm model, it is common to use simplified analysis by beam analogy. This method permits the diaphragm, or segments of the diaphragm, to be idealized as an equivalent beams with spans between the vertical elements of the seismic force- resisting system. The support elements are not taken as rigid supports, but instead provide resisting forces in proportion to the relative stiffness of each element at the diaphragm level with respect to the base of the structure. In this case, the sum of the forces for the beam supports is equal to the total force at the level, but an imbalance in moment that may occur is resolved by resistance provided by the vertical elements in the orthogonal direction taking the actual and accidental torsion moments.
- The beam analogy permits a simple calculation of moments and shears as if the diaphragm were a beam. This model does not include the development of diaphragm tension forces that may occur in semi-rigid models. The beam analogy does not consider deep beam effects, and it will produce conservative estimates of the lateral force actions on the diaphragm.

Step 4: Design diaphragm connections for required strength

1. Select diaphragm connectors or joint reinforcement based on required diaphragm connection and joint reinforcement classification.

   − Diaphragm connectors and joint reinforcement are classified using the cyclic testing protocols in Section 14.2.4 in Part 1 of the 2015 NEHRP Provisions.

2. Establish diaphragm connection properties required for design including:

   a. Elastic stiffness in tension and shear: k_t, k_v
   b. Yield strength in tension and shear: t_n , v_n

   − See (Pankow, 2014) for determination of properties of diaphragm connections.

3. Diaphragm connections at each joint between precast elements shall possess sufficient total strength (N_n, V_n, M_n) to resist the diaphragm internal forces. The following general interaction formula is permitted to be used for diaphragm joint design:

$$\sqrt{\left(\frac{|M_u|}{\phi_f M_n} + \frac{N_u}{\phi_f N_n}\right)^2 + \left(\frac{\Omega_v V_u}{\phi_v V_n}\right)^2} \leq 1.0$$

where strength reduction factors are as given in the relevant material standard

USER NOTES:

- In most diaphragms, there are areas strongly dominated by either shear or bending where the interaction of the structural effects is not significant and the dominant action can be compared directly to the strength of the connectors or joint reinforcement in the joints.
- When the rigid diaphragm assumption is used, the application of the connection interaction equation can be simplified. As noted above, the beam analogy does not produce separate tension in the diaphragm, so the interaction equation can be written as:

$$\sqrt{\left(\frac{|M_u|}{\phi_f M_n}\right)^2 + \left(\frac{\Omega_v V_u}{\phi_v V_n}\right)^2} \leq 1.0$$

  In the case of an untopped diaphragm with tension-compliant shear connections, the diaphragm moment can be considered as resisted by only the tension-compression couple between the chord reinforcement or chord connections. Since compression by concrete contact can supplement the steel compression in the connection, it is common to take $\phi M_n = 0.9 f_y A_s d$, where d is the distance between the chord connections. ACI 318-11 Section 21.11.9.3 permits the total area of reinforcement transverse to the joints to be included in the calculation of shear strength, including the boundary reinforcement. When the boundary reinforcement is included in the strength calculation, then the interaction equation should be applied. When the full shear strength requirement is met by mechanical shear connectors or shear reinforcement, it is permissible to consider the boundary, chord reinforcement as resisting the moment only.

- Deformation capacity should be considered for other connectors and joint reinforcement in the floor system, including those at diaphragm-to-spandrel connections, spandrel-to-column connections, and interior beam connections not included in the design.

4. The nominal strength of the joint for the interaction equation can be calculated as follows:

$$N_n = \sum t_n$$

$$V_n = \sum v_n$$

   where t_n and v_n are the nominal tension and shear strengths of individual connection elements crossing the joint. The appropriate nominal flexural strength M_n of the joint depends on the diaphragm design option:

- For elastic design option: $M_n = M_y$
- For basic design option: $M_n = \frac{1+\Omega_d}{2} M_y$
- For reduced design option: $M_n = \Omega_d M_y$

where:

M_y is the yield moment of the precast diaphragm joint;

Ω_d is the diaphragm joint flexural overstrength factor, defined as the ratio of the diaphragm joint plastic moment to the diaphragm joint yield moment, M_p/M_y , conservatively taken as *1.0* for a pretopped diaphragm, and *1.25* for a topped diaphragm. Ω_d can alternately be determined from a strain compatibility analysis or pushover analyses.

USER NOTES:

- Primary diaphragm joints include those between precast floor units, diaphragm-to-LFRS joints in both orthogonal direction, and internal beam joints between chord lines.
- A rational method is provided for calculating the diaphragm flexural strengths M_y and M_p and has been embedded in a design aid program. See "Diaphragm Joint Strength Calculation" and "Design Aids for Diaphragm Design: Spreadsheet Program" in PART 3 of (Pankow, 2014).
- It is noted that diaphragm-to-LFRS connections require a different set of required deformation characteristics than those defined by the diaphragm connector and joint reinforcement classification.
- So-called "secondary" connections in the floor system, including diaphragm-to-spandrel connections, spandrel-to-column connections, non-spandrel beam connections outside the chord lines and ramp-to-lite wall non- diaphragm connections, DO NOT require a seismic strength design.

Step 5: Determine diaphragm stiffness and check gravity system drifts if applicable.

1. A diaphragm stiffness calculation is required if:
   - a semi-rigid diaphragm computer structural analysis model is used, or
   - a gravity system drift check is required.
2. For a semi-rigid diaphragm model, determine a diaphragm effective elastic modulus (E_{eff}) and shear modulus (G_{eff}) considering cracking as well as the jointed nature of the diaphragm.
3. A check for the gravity system drift is required for certain design cases as specified in Table 4.3.

Table 4.3 Design cases requiring gravity system drift check

Design Option	Cases	Requiring	Drift
	$n \le 3$	$3 < n \le 5$	$n>5$
EDO	-	-	AR>3.
BD	AR>3.	AR>3.	5
O	8	5	AR>3.

RP3-4.5 Comparison of the DSDM Design methodology with the Restrepo-Rodriquez Equations

DSDM diaphragm design force levels are those of ASCE 7-10 Section 12.10, amplified by ψE, ψD, and ψR factors corresponding to EDO, BDO, and RDO, respectively. Figures 4-18 through 4-21 compare these force levels with the Restrepo-Rodriguez (Rodriguez et al., 2002; Rodriguez et al., 2007) diaphragm design force levels, unreduced by any RS factor, for four different locations within the United States. DSDM has been modified, so that it works with the Restrepo-Rodriguez diaphragm design force level reduced by an RS factor, as adopted in Part 1 of the 2015 NEHRP Provisions. The RS factor, as has been noted before, has different values for the three different design options.

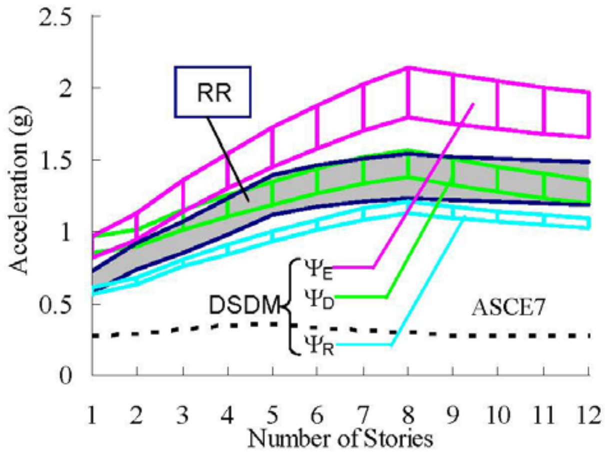


FIGURE 4-18 Comparison of DSDM and Restrepo-Rodriguez force levels for Berkeley, CA

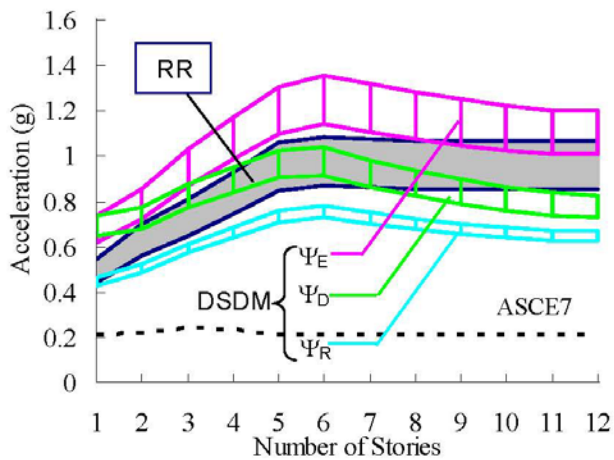


FIGURE 4-19 Comparison of DSDM and Restrepo-Rodriguez force levels for Seattle, WA

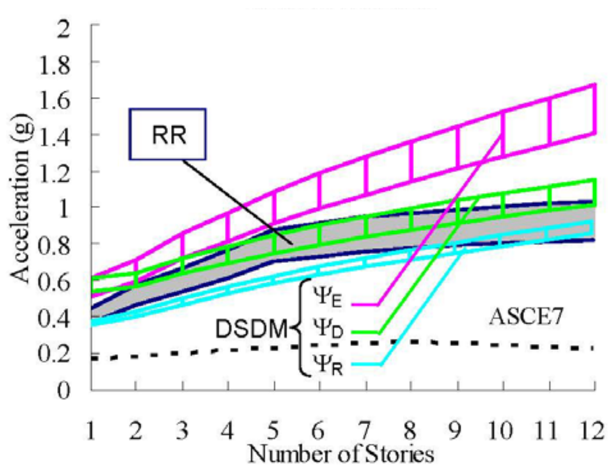


FIGURE 4-20 Comparison of DSDM and Restrepo-Rodriguez force levels for Charleston, SC

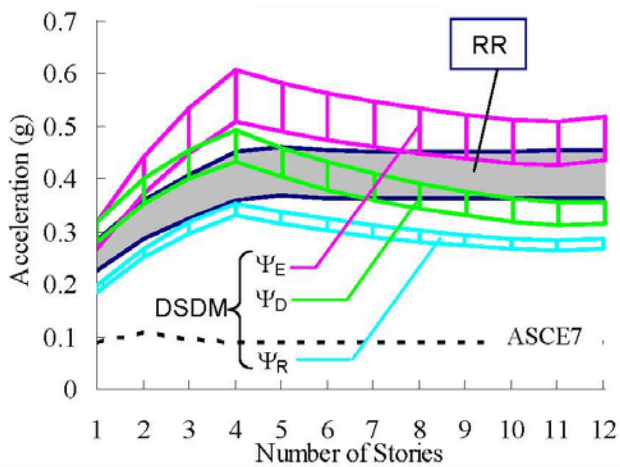


FIGURE 4-21 Comparison of DSDM and Restrepo-Rodriguez force levels for Knoxville, TN

In DSDM, the vertical distribution of diaphragm forces is controlled by the diaphragm vertical force distribution factor, α_X. The story-based α_X factor is intended to provide a reasonable distribution of the diaphragm forces along the height of the structure. It is noted that the α_X factor provides a different vertical distribution that that produced by F_{px} in Section 12.10 of ASCE 7-10.

It is also noted that in the past, researchers endorsed a constant design force distribution along the height of the structure (Fleischman et al., 2002). The introduction of the α_X factor is in recognition of the results of earthquake simulations using more improved analytical models that incorporate certain features into the structural models not included previously, e.g. gravity system columns.

In general, a simple but potentially overly conservative alternate approach is to use a value of α_X =1.0 at each floor. However, in order to provide a more economical design option, α_X values have been determined on the basis of results from three-dimensional nonlinear transient dynamic analysis (Zhang and Fleischman 2012a). It is noted that the αx values are based on a fairly limited sample of analyses as described next and could benefit from a more comprehensive examination.

A distinction is made between the distribution of general building structures and parking structures as a different distribution was obtained for each. The α_X values for general building structures are found in the DSDM Report (Pankow, 2014) PART 1: Appendix 1. These factors were determined from the average distribution of 5 earthquake simulations for 24 separate design cases using a simple evaluation structure, and verified by a single earthquake simulation of a realistic 8-story office building under a bi-directional ground motion

For mid-rise parking structures, the diaphragm force is maximum at the roof, and reasonably bounded by a constant reduced value for the remainder of the lower floors (Figure 4-22). The α_X factors for the parking structure were established based on the results of five total analyses of realistic 4-story parking structures under bidirectional earthquake ground motions: three with exterior transverse shear walls and lite walls (along the ramp); one with interior transverse shear walls and lite-walls; and one with perimeter shear walls in both directions (see DSDM Report PART 5: Appendix A2). Note that in using the α_X factor for a parking structure, it is necessary to assign a level for a ramp sub-diaphragm. In this procedure, such an element is considered part of the uppermost level to which it connects (Pankow, 2014).

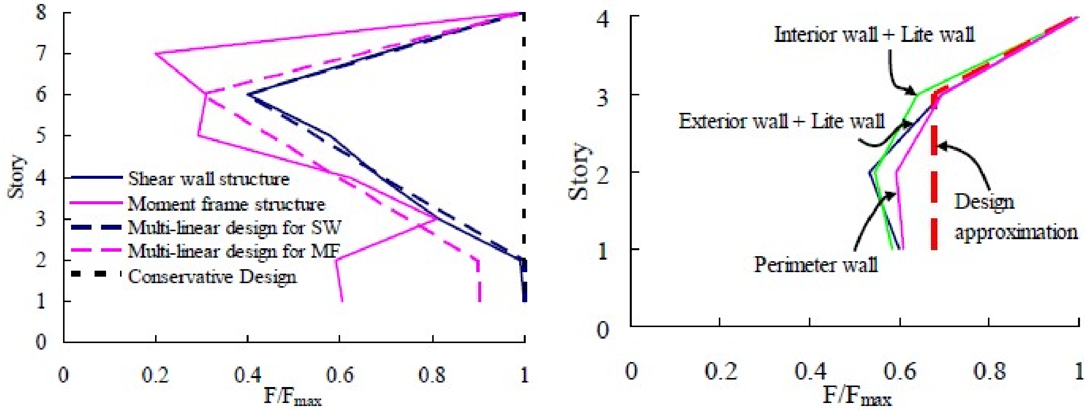


FIGURE 4-22 Distribution of diaphragm forces along height of building (a) 8-story office building, (b) 4-story parking garage

The diaphragm design force distribution along the height of the structure, as shown in Figure 12.11-1 and Section 12.11.2, has been adopted in Section 12.11. Alternative provisions for diaphragms including chords and collectors, in Part 1 of the 2015 NEHRP Provisions.

RP3-4.6 Modifications to ASCE 7-10 for Precast Concrete Diaphragms

The 2015 NEHRP Provisions include recommended modifications to ASCE 7-10 Sections 11.2, 11.3, 12.2-1, 12.3.1.3, Figure 12.3-1, Sections 12.3.3.4, 12.3.4.1, and 12.10 for the level of diaphragm design force. These provisions are offered as an alternative to make the design of horizontal and vertical elements of the seismic force-resisting system more consistent. The proposal, however, makes these provisions mandatory for precast concrete diaphragms in structures assigned to SDC C, D, E, and F.

The calculation of in-plane seismic design forces for diaphragms includes the term Rs, the diaphragm design force reduction factor in Table 12.11.5. The value of this factor for precast concrete diaphragms varies depending upon the diaphragm design option selected.

Issue Team 6 has also proposed modifications to ASCE 7-10 Section 14.2, adding a new Section 14.2.4, which provides specific provisions for the definition and application of these precast concrete diaphragm design options, including the details of qualification of connectors or joint reinforcement used in design. These modifications, when approved, will also be in Part 1 of the 2015 NEHRP Provisions. Section 14.2.4 does not prescribe the step-by-step diaphragm design method or analysis requirements given above. These are typically determined by the licensed design professional based on the specific requirements of the project.

RP3-4.7 Conclusions

Precast concrete diaphragms share one fundamental characteristic: their behavior is largely characterized by the jointed nature of the construction. Historically, the prescriptive design requirements for precast concrete diaphragms in high seismic design categories have been restricted to cast-in-place concrete topping slabs. The use of mechanical connections at the joints has not been recognized. The experience of the 1994 Northridge earthquake showed that any reinforcement crossing the joints can be ineffective if it does not include the strength or strain capacity to match the opening of joints. ACI 318 made changes to provide greater strain capacity in welded wire reinforcement used in topping slabs, but did not address the alternative of deformation-tolerant mechanical connections at these joints. As the research has shown, simply fixing the traditional design approach to address a perceived deficiency misses the broader understanding of forces and deformation demands.

Modifications to Chapter 12 of ASCE 7-10, approved for Part 1 of the 2015 NEHRP Provisions, correct the force calculation for diaphragm design to more accurate predictions based on analysis and testing. Companion modifications have been proposed to Chapter 14 of ACE 7-10 to clearly define those requirements needed for diaphragms to perform as intended by the designer, when subject to the modified design force levels. With a clear understanding of real lateral force demands and precast diaphragm behavior in earthquakes, a new design protocol has been developed following the DSDM research. The new design may provide solutions that include tested and proven mechanical connections substituting for joint reinforcement to provide for enhanced diaphragm performance. The research, in addition to pointing to needed revisions in the equivalent force required for diaphragm design, has provided the basis for qualifying mechanical connectors and joint reinforcement to meet performance requirements for precast concrete diaphragms.

RP3-5 CHAPTER 5 - STEEL DECK DIAPHRAGMS, TOPPED COMPOSITE, TOPPED NON-COMPOSITE AND UNTOPPED

RP3-5.1 Current Practice

RP3-5.1.1 Common Building Types

Gravity and lateral force-resisting systems often utilize steel deck diaphragms. Steel deck can be bare (or untopped) deck as used on roofs, topped with structural concrete in floor applications, or topped with insulating concrete in some roof deck applications. Typical buildings with steel deck diaphragms range from multi-story structural or cold-formed steel frame construction to single story concrete shear wall or concrete masonry unit (CMU) construction with open web steel joists or hot rolled steel roof framing. Occupancies include business, mercantile, multi-family residential, “big box” retail structures, schools, hospitals, industrial and warehouse construction. Building sizes range from a few thousand square feet to over one million square feet for large warehouses. These very large structures are often identified as “Diaphragm Response Dominated Buildings.”

RP3-5.1.2 System Description

When utilized as part of the lateral force-resisting system, the diaphragm system components include the steel deck as a stressed skin, fasteners, and the chords and collectors.

RP3-5.1.2.1 Steel Deck

Roof Deck. Steel roof deck typically varies from 16 to 22 gauge in thickness, and 1-1/2 to 3 inches in depth, however long-span profiles with thicknesses ranging up to 10 gauge and depths up to 12 inches are occasionally used. Steel roof deck does not form the exposed roof surface; rather, the deck is covered with insulation and finish roofing materials. The steel deck attached to the supporting frame acts as a stressed skin in the diaphragm. In these situations, the governing standard for the design of steel roof deck is the Steel Deck Institute, ANSI/SDI RD-2010, *Standard for Steel Roof Deck* (SDI, 2010a).

Exposed steel roofing panels can also be used as a diaphragm, as is often the case in metal building systems. Steel roof sheathing panels may be either through-fastened or standing seam type. Most through-fastened panels are 1-1/2 inches deep and 24 or 22 gauge thick and can be designed to resist diaphragm forces. However, the majority of standing seam panels do not transfer diaphragm design forces, and supplemental in-plane roof bracing is required.

Floor Deck. Steel floor deck can either be composite or non-composite. Composite deck bonds with the concrete such that the deck provides the tensile reinforcement for the composite deck- slab without the need for reinforcing steel; whereas non-composite deck (form deck) provides a stay-in-place concrete form with the concrete slab designed independently. In either instance, the diaphragm design of the deck is similar when considering the contributions of the fasteners and the concrete slab to the in-plane shear resistance. The concrete slab shear strength and stiffness dominates the behavior of topped steel deck diaphragms. Interestingly, composite deck design does not significantly improve the stiffness of the diaphragm when compared to non-composite design, but both are substantially stiffer than untopped steel deck diaphragms.

Composite steel floor deck typically varies from 16 to 22 gauge in thickness, and is available in 1- 1/2, 2 and 3 inch depths; however, long-span profiles with thicknesses ranging up to 10 gauge and depths up to 12 inches are occasionally used. Non-composite steel floor deck typically ranges from 16 to 24 gauge and 1/2 to 3 inches in depth, however deeper and thicker sections are available. The governing standards for the design of steel floor deck are the ANSI/SDI NC-2010, *Standard for Non-composite Steel Floor Deck* (SDI, 2010b), and ANSI/SDI C-2011, *Standard for Composite Steel Floor Deck-Slabs* (SDI, 2011).

RP3-5.1.2.2 Steel Deck Diaphragm Supports

Roof and floor framing member supports for steel deck diaphragms are typically hot-rolled structural steel sections or open web steel joists (bar joists). Cold-formed steel purlins are also used for lighter or smaller structures, as in the case for metal building systems. Depending upon the chosen structural system, the governing standards include: AISC 360-10, Specification for Structural Steel Buildings (AISC, 2010), the Steel Joist Institute's (SJI) Standard Specifications and Load and Weight Tables for Steel Joists and Joist Girders (SJI, 2010), and AISI S100, North American Specification for the Design of Cold-Formed Steel Structural Members (AISI, 2010).

RP3-5.1.2.3 Connections

Two different connections occur with the steel deck diaphragm – structural connections and sidelap connections. Structural connections attach the steel deck panels to the supporting structure perpendicular to the deck span in the lower flutes of the deck profile. Structural connections can be welded or mechanically fastened with power-actuated fasteners or screws.

Sidelap connections attach the steel deck panels to adjacent steel deck panels parallel with direction of the deck span. Sidelap connections can be welded with seam welds, screw fastened, button punched, or mechanically formed. Two different types of steel deck panel sidelap conditions exist, either nestable or interlocking. The type of sidelap connection method used depends on the type of steel deck sidelap condition. Nestable deck sidelaps are typically screw fastened or welded. Interlocking deck sidelaps are typically button punched, mechanically formed or screw fastened (with a special interlocking screwable sidelap type deck).

Welding. Welding of steel deck diaphragms is performed in accordance with AISI S100, and AWS D1.3, *Structural Welding Code – Sheet Steel* (AWS, 2008). Both standards provide design equations and requirements for arc spot welding and seam welding. The *SDI Diaphragm Design Manual* (SDI, 2004) also includes separate design equations for calculating the strength and stiffness of welded structural and sidelap connections.

Power-Actuated Fasteners. Power-actuated fasteners are commonly used for attachment of steel deck diaphragms to supporting structural steel beams and open web steel joists. These mechanical fastening systems consist of proprietary powder cartridge or pneumatically driven steel fasteners.

Fastener designs include knurled shank fasteners for improved anchorage in the base steel, pre-mounted steel washers for clamping the steel deck panels to the base steel and proprietary hardened, yet ductile steel formulations. These features contribute to the shear and tensile resistance of the steel deck connections that, in certain cases, can approach the strength of properly installed arc spot welds. The most common power-actuated fastener types are listed in the *SDI Diaphragm Design Manual* (DDM) and ICC-ES Evaluation Service Reports (ESRs) through AC43 (ICC, 2013) product evaluations (www.icc-es.org).

Screw Fasteners. Screw-fastened connections of cold-formed steel decks are designed in accordance with the requirements of AISI S100. Alternatively, the *SDI Diaphragm Design Manual* provides separate equations for calculating the strength and stiffness of both screw- fastened structural and sidelap connections. It is anticipated that the next version of the *SDI Diaphragm Design Manual* (DDM04) (SDI, anticipated 2014) will align screw fastener design equations with the newly completed AISI S310, *North American Standard for the Design of Profiled Steel Diaphragm Panels* (AISI, 2013), and AISI S100. Data developed in accordance with AISI S904, *Standard Test Methods for Determining the Tensile and Shear Strength of Screws* (AISI, 2008), and AISI S905, *Standard Test Methods for Mechanically Fastened Cold-Formed Steel Connections* (AISI, 2008), test standards may also be used for design purposes.

RP3-5.1.3 Analysis

Most steel deck diaphragm designs use the Allowable Stress Design (ASD) method, however Load and Resistance Factor Design (LRFD) and Limit States Design (LSD) methods have been available for over 20 years. LSD is the primary design method in Canada and uses slightly different diaphragm resistance factors than LRFD. Typically, ASD results in a slightly more conservative seismic design, so future design practices may favor LRFD or LSD design.

RP3-5.1.4 Determination of Steel Deck Diaphragm Design Capacities

Steel deck diaphragm shear strength and stiffness capacities are based on analytical, semi-analytical, or test-based methods. All analytical and semi-analytical based methods have been extensively validated by testing. Diaphragm shear strength and stiffness capacities are published in tabular form in various sources, showing the necessary construction parameters (deck gauge, span, fastener types, patterns and spacings) for simplified design. Separate checks are made for the deck buckling limit state and are either incorporated in the tables or calculated separately. Manufacturers also publish diaphragm capacities based upon testing with specific fastening systems.

The general design approach for steel deck diaphragms assumes that the stressed skin is the web of a deep steel beam. The supporting members, upon which the steel deck bears and is fastened, act as web stiffeners. Diaphragm chords and collectors function as the beam flanges. However, the existing steel deck diaphragm methods address only the design of the stressed skin and not the supporting structural steel beams, open web steel joists, or cold-formed steel purlins. Rather, the AISC, SJI and AISI design standards mentioned in Section 5.1.2.2 are used for the design of these components of the building roof and floor system.

Diaphragm shear strength and stiffness tables are generally used by the engineer in order to proportion the stressed skin, using different gauge thickness steel deck panels and varying the fastening patterns for the structural and sidelap connections. In this sense, the design of the stressed skin can be somewhat iterative. Various design selector charts and tables by manufacturers are also available.

Historically, a number of different design references have been used to establish the shear strength and stiffness capacities of steel deck diaphragms, including the SDI Diaphragm Design Manual, ICC-ES Evaluation Reports, IAPMO-ES Evaluation Reports and the Tri-Services Method. Once it is available in 2014, AISI S310-13 will be the go-to standard for the industry; it is based primarily on the SDI Diaphragm Design Method with some modifications and updates.

RP3-5.1.4.1 SDI Diaphragm Design Manual, 3rd Edition (SDI-DDM03, SDI, 2004)

The SDI-DDM Method is the most fully developed and accepted design method currently available. It is applicable to both bare deck and filled diaphragms and presents a rigorous analytical method based on engineering mechanics principles that has been verified by large scale diaphragm testing.

Because of the proven analytical basis of this method, varied designs are possible, if the properties of the steel deck and the fasteners are known.

This analytical model allows for accurate prediction of both diaphragm strength and stiffness of the steel deck as a membrane. Panel buckling and fastener limit states are also considered. However, this method does not include the contributions of diaphragm chords and collectors, which must be designed using methods found in the applicable design standards published by AISC, AISI, and SJI.

Zoning of the steel deck diaphragm is often done in high seismic and wind regions in order to produce a more economical and efficient design. Since the shear demand on the diaphragm varies across the width of the structure, a more efficient selection of steel deck panel thickness and variation in the fastening type and patterns at the perimeter and interior portions of the stressed skin can provide major savings in materials, weight, cost and deck installation time, as long as the wind uplift demands at the perimeter and corner regions are met. Zoning the diaphragm to accommodate the varying shear demands is efficient design practice, but this has to be balanced with the constructability of the steel deck diaphragm.

RP3-5.1.4.2 Tri-Services Method

(TSM) 1982 Edition - TM 5-809-10 Seismic Design For Buildings Method – “Tri-Services Method” (NAVFAC, 1982, 1992)

In the 1982 edition of TM 5-809-10, seismic design is based on the 1975-1978 SEAOC Seismology Committee’s “Recommended Lateral Force Requirements and Commentary.” The TM covers diaphragms constructed of wood, concrete, or steel used as horizontal bracing and includes the following assumptions.

I. Distribution of Seismic Forces:

1. Rigid diaphragms distribute the horizontal forces to the vertical force resisting elements based on relative rigidity.
2. Flexible and very flexible diaphragms distribute the horizontal forces to the vertical force resisting elements based on tributary load analysis (tributary area).
3. Semi-rigid and semi-flexible diaphragms have sufficient stiffness to distribute a portion of the horizontal load in proportion to their rigidities. The TM suggests this analysis is difficult and the outcome is not better than the assumptions made to do the analysis.
4. Torsion is to be considered where the diaphragm is sufficiently rigid to transfer torsion. Torsion is transferred through rigid diaphragms, and 5% accidental torsion is required for all buildings with rigid diaphragms.

II. Diaphragm Deflections:

1. Deflections are the sum of two components – shear and flexure.
2. Deflections are limited by the allowable deflection (drift between the walls and the horizontal diaphragm). The TM contains information regarding seismic out-of-plane loads for walls due to seismic loads, but lists no specific displacement limitation for the diaphragm.

III. Flexibility Limitations:

Diaphragm deflections can be calculated with reasonable accuracy for some diaphragms. but for others accurate calculations may not be possible. Therefore an empirical table (Table 5-1) is used to proportion diaphragm in lieu of rational calculations. This table contains formulas to predict the flexibility factor, F, for steel deck diaphragms.

RP3-5.1.4.3 1992 Edition - TM 5-809-10 Seismic Design for Buildings Method

“Tri-Services Method” NAVFAC, (NAVFAC, 1982, 1992)

In the 1992 edition of TM 5-809-10, seismic design is based on the 1990 SEAOC Seismology Committee "Recommended Lateral Force Requirements and Commentary" and includes the following assumptions:

I. Distribution of Seismic Forces:

1. Relative flexibility
   a. Whether a diaphragm responds as rigid or flexible is dependent on the relative stiffness of the diaphragm compared to the vertical lateral force-resisting elements.
   b. It does not provide a prescriptive method of how to address this concept.

Table 5-1 Flexibility Limitations on Diaphragms Per TM5-809-10, 1982 Edition

			Span/Depth Limitations			
Flexibility Category	F (micro- in/lbs)	Maximum Span (feet)	No torsion considered in diaphragm[3]		Torsion considered in diaphragm[2]	
			Brittle Walls[1]	Flexible Walls[2]	Brittle Walls[1]	Flexible Walls[2]
Very flexible	Over 150	50	Not to be used	2:1	Not to be used	1-1/2:1
Flexible	70-150	100	2:1	3:1	Not to be used	2:1
Semi- flexible	10-70	200	2-1/2:1	4:1	Not to be used	2-1/2:1
Semi-rigid	1-10	300	3:1	5:1	2:1	3:1
Rigid	Less than 1	400	Deflection requirement only	No limitation	Deflection requirement only	3-1/2:1

Note: [1]Walls in concrete and unit-masonry are classified as brittle; in all cases, check allowable drift before selecting the type of diaphragm.
Note: [2]When applying these limitations to cantilever diaphragms, the span/depth ratio shall be limited to one-half that shown.
Note: [3]No torsion in diaphragm other than 5% "accidental" torsion required by chapter 3, paragraph 3-3(E)5.

2. Flexible diaphragms distribute the horizontal forces to the vertical force resisting elements based on tributary load analysis (tributary area). Generally, most bare steel decks are assumed to be flexible.
3. Rigid diaphragms distribute the horizontal forces to the vertical force resisting elements based on relative rigidity. Generally, concrete and concrete filled steel deck diaphragms are considered rigid.
4. Diaphragm of intermediate flexibility are diaphragms that are in-between flexible and rigid. These diaphragms have sufficient stiffness to develop some distribution of load based on relative rigidity of the vertical lateral force-resisting elements, and are dependent of relative stiffness of the diaphragm and vertical elements.
5. Torsion is to be considered where the diaphragm is sufficiently rigid to transfer torsion. Torsion is transferred through rigid diaphragms and 5% accidental torsion is required for all buildings with rigid diaphragms.

II. Diaphragm Deflections

1. Deflections are the sum of two components – shear and flexure.
2. Deflections are limited by the allowable deflection (drift between the walls and the horizontal diaphragm). The TM contains information regarding seismic out-of-plane loads for walls due to seismic loads, but lists no specific displacement limitation for the diaphragm.

III. Flexibility Limitations

1. Diaphragm deflections can be calculated with reasonable accuracy for some diaphragms, but for others accurate calculations may not be possible. Therefore an empirical table (Table 5-2) is used to proportion the diaphragm in lieu of rational calculations. This table contains formulas to predict the flexibility factor, F, for steel deck diaphragms.

2. Empirical rules are used because direct design is not feasible. (In 1992 FEA analysis software to analyze deflection and stiffness effects of a diaphragm was very limited in availability and not practical for most projects.)
3. Design requirements are considered to be met if the diaphragm conforms to Table 5-2 for ordinary buildings.
4. The limitations of Table 5-2 may be exceeded, but only when justified reliable evaluation of the strength and stiffness characteristics of the diaphragm are performed.
5. The following list was provided to help categorize diaphragms:
   a. Concrete diaphragms are rigid; gypsum concrete and lightweight insulating concrete (LWIC) diaphragms are semi-rigid
   b. Steel deck diaphragms can be semi-rigid, semi-flexible, or flexible
   c. Plywood diaphragms can be very flexible, flexible, or semi-flexible
   d. Special diaphragms of diagonal wood sheathing are flexible
   e. Conventional diaphragms of straight wood sheathing are very flexible
6. Table 5-2 contains formulas to predict the flexibility factor, F, for steel deck diaphragms.

Table 5-2 Flexibility Limitations on Diaphragms Per TM5-809-10, 1992 Edition

Flexibility Category	F (micro-in/lbs)	Maximum Span (feet)	Diaphragm Span/Depth Limitations	
			Concrete or Masonry Walls[1]	Other Walls
Very flexible	Over 150	50	Not to be used	2:1
Flexible	70-150	100	2:1	3:1
Semi-flexible	10-70	200	2-1/2:1	4:1
Semi-rigid	1-10	300	3:1	4:1
Rigid	Less than 1	400	4:1	4:1

Note: [1]Walls in concrete and unit-masonry are classified as brittle; in all cases, check allowable drift before selecting the type of diaphragm.

RP3-5.1.4.4 Evaluation Reports

Because of an historical lack of a consensus mandatory standard for steel diaphragm design, building code recognition of steel deck and fastening system product test data has been commonly based upon evaluation standards published by the particular evaluation service agency (International Code Council, ICC; International Association of Plumbing and Mechanical Officials, IAPMO, etc.). The most commonly cited evaluation standard for steel deck diaphragms is ICC AC43, Acceptance Criteria for Steel Deck Roof and Floor Systems, as published by ICCES

(ICC, 2013). AC43 was first published in 1996. It was subsequently revised administratively and technically in 2002, 2004, 2006, 2007, 2010, and 2011. Future revisions will most likely be tied directly to AISI S310 to the point that ICC-ES could eventually reference the standard in part or whole in AC43.

Over the years, ICC-ES AC43 has reflected the general practices in use by report holders at the time and has generally followed the requirements of ASTM E455, *Standard Method for Static Load Testing of Framed Floor or Roof Diaphragm Constructions for Buildings* (ASTM, various), and AISI TS7, *Cantilever Test Method for Cold-Formed Steel Diaphragms* (now AISI S907). Diaphragm system tests have been analyzed based on the general principles presented in either the Tri-Services Manual (TSM) TM5-809-10 or the SDI Diaphragm Design Manual. In fact, the TSM method has been used to evaluate proprietary fastening systems, including power-actuated fasteners, screw fasteners, and side seam clinch systems. The published evaluation reports have been readily approved by authorities having jurisdiction and engineers for use of these products on projects in the U.S. and world-wide.

ICCES AC43 provides test procedures for conducting the cantilever diaphragm system test in accordance with AISI S907, *Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragm*, and diaphragm connection tests in accordance with AISI S905, *Test Methods for Mechanically Fastened Cold-Formed Steel Connections*. In addition, AC43 covers how the diaphragm shear strength and shear stiffness are calculated based on the test measurements and provides Condition of Use statements that are included in the evaluation reports. At this time, AC43 does not require cyclic seismic load testing.

Flexibility limitations are placed on steel deck diaphragms in AC43 as follows:

Table 5-3 Flexibility Limitations on Diaphragms Per ICC AC43[1,2,3,4,5]

see note*			Span/Depth Limitations			
		Maximum diaphragm span for masonry or concrete walls (feet)	Rotation not considered in diaphragm[3]		Torsion considered in diaphragm[2]	
Flexibility Category	F (micro- in/lbs)		Masonry or concrete walls	Flexible Walls	Masonry or concrete walls	Flexible Walls[2]
Very flexible[4]	Over 150	50	Not used	2:1	Not used	1-1/2:1
Flexible	70-150	100	2:1 or as required for deflection	3:1	Not used	2:1
Semi- flexible	10-70	200	2-1/2:1 or as required for deflection	4:1	As required for deflection	2-1/2:1
Semi-rigid	1-10	300	3:1 or as required for deflection	5:1	As required for deflection	3:1
Rigid	Less than 1	400	As required for deflection	No limitation	As required for deflection	3-1/2:1

Note: [1]Diaphragms are to be investigated regarding their flexibility and recommended span-depth limitations.

Note: [2]Diaphragms supporting masonry or concrete walls are to have their deflections limited to the following amount:

$\Delta wall = H^2 fc/(0.1Et)$** where:

H = Unsupported height of wall in feet.

t = Thickness of wall in inches.

E = Modulus of elasticity of wall material for deflection determination in pounds per square inch.

fc = Allowable compression strength of wall material in flexure in pounds per square inch.

For concrete, fc = 0.45 f'c. For masonry, fc = Fb = 0.33 f'm.

Note: [3]The total deflection Δ of the diaphragm may be computed from the equation:

$\Delta = \Delta f + \Delta w$

where:

Δf = Flexural deflection of the diaphragm determined in the same manner as the deflection of beams

Δw = The web deflection may be determined by the equation:

Δw = qaveLF/106

where:

L = Distance in feet between vertical resisting element (such as shear wall) and the point to which the deflection is to be determined.

qave = Average shear in diaphragm in pounds per foot over length L.

F = Flexibility factor: The average micro inches (μm) a diaphragm web will deflect in a span of 1 foot (m) under a shear of 1 pound per foot (N/m).

Note: [4]When applying these limitations to cantilevered diaphragms, the allowable span-depth ratio will be half that shown.

Note: [5]Diaphragm classification (flexible or rigid) and deflection limits shall comply with the Diaphragm Design Considerations section of the evaluation report.

*Additional Note: This column was removed in the early 2000's because the IBC and UBC did not provide definitions for diaphragm other than flexible and rigid.

**Additional Note: The wall deflection equation comes from SEAOC Seismology Committee Bulletin No. 1, 1949 and is based on allowable bending stress of an unreinforced masonry wall.

RP3-5.1.4.5 IAPMO-ES Evaluation Reports

Requirements for steel deck diaphragm design are found in IAPMO Evaluation Services Evaluation Criteria EC-007, Evaluation Criteria for Steel Composite, Non-Composite and Roof Deck

Construction (IAPMO, 2013). This criterion may be used for design of buildings in accordance with the 2012 edition of the IBC, International Building Code, and provides similar evaluations of the diaphragm shear strength and stiffness using the AISI S907 test method. At this time, EC-007 does not require cyclic seismic load testing. Flexibility limitations are based on the requirements of the IBC and the 2005 edition of ASCE 7, Minimum Design Loads for Buildings and Other Structures, which constitutes the major difference between the IAPMO-ES EC-007 and ICC-ES AC43.

RP3-5.1.4.6 Full Scale Testing

Full-scale (or large scale) diaphragm system tests have been more common for diaphragm system evaluations in the western U.S. based on many tests by steel deck and fastener manufacturers. These tests have historically followed ASTM E455 Standard Method for Static Load Testing of Framed Floor or Roof Diaphragm Constructions for Buildings or AISI S907 Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms.

RP3-5.1.4.7 AISI S310-13 "North American Standard for the Design of Profiled Steel Diaphragm Panels"

Up until this point in time, there has been no consensus mandatory standard for establishing the shear strength and stiffness capacities of steel deck diaphragms within the IBC. AISI, with the cooperation of the SDI, sought to remedy this shortcoming by developing AISI S310, North American Standard for the Design of Profiled Steel Diaphragm Panels. This Standard was developed under ANSI consensus guidelines over a six year period and is planned for publication in 2014.

AISI S310 is based on the analytical SDI Method of determining diaphragm panel strength, while incorporating the AISI S100 strength formulations for welds and screws. Connector flexibilities from the SDI DDM03 were retained.

AISI S310 is flexible and allows for growth and development of alternate diaphragm connectors. Instead of using current formulations for fastener strength and flexibility, it is a permitted option to instead develop these values by small scale fastener testing (using AISI S905) for any diaphragm. Additionally, full scale testing (using AISI S907) is always an option.

Finally, AISI S310 only considers the steel deck or metal roofing panel as a stressed skin diaphragm and does not incorporate the design of chords and collectors.

RP3-5.1.5 Factors of Safety for Steel Deck Diaphragms

Factors of Safety and Resistance Factors for steel deck diaphragms are obtained from several sources, depending upon the design method used.

RP3-5.1.5.1 SDI Diaphragm Design Manual, 3rd Edition (SDI, 2004)

Bare Deck. Within the SDI-DDM, design of connections for bare deck diaphragms is in accordance with AISI S100. Table D5 of AISI S100 provides the following factors of safety and resistance factors for diaphragms designed using the Allowable Stress Design (ASD), Load Resistance Factor Design (LRFD) and Limit States Design (LSD) methods:

Factor of Safety for Diaphragms (Seismic Load) – ASD Design

Screw Connectors	$\Omega = 2.50$
Welded Connectors	$\Omega = 3.00$
Buckling	$\Omega = 2.00$

Resistance Factor for Diaphragms (Seismic Load) – LRFD Design

Screw Connectors	$\phi = 0.65$
Welded Connectors	$\phi = 0.55$
Buckling	$\phi = 0.80$

Resistance Factors for Diaphragms (Seismic Load) – LSD Design

Screw Connectors	$\phi = 0.60$
Welded Connectors	$\phi = 0.50$
Buckling	$\phi = 0.75$

For mechanical fasteners other than screws, the factor of safety and resistance factor is determined by statistical analysis of test data conducted in accordance with AISI S100; however, the factor of safety cannot be less than, and the resistance factor cannot be greater than, the applicable factors for screws. If a diaphragm consists of both mechanical and welded connectors (either support or sidelap), then the most stringent factors apply.

The factors of safety and resistance factors listed in AISI S100 are based on a recalibration of the full-scale test data summarized in the SDI Diaphragm Design Manual, First Edition (SDI, 1981). The recalibration used the load factors in the 1998 edition of ASCE 7. The dominant diaphragm limit state is connection related. The calibration used $\beta_O = 3.5$ for all load effects except wind load. The LRFD method allows $\beta_O = 2.5$ for connections subjected to wind loads. For both welds and screws, calibration for wind using 2.5 suggests factors less than $\phi = 0.8$ and $\Omega = 2.0$, less severe than the values provided above for seismic loading.

Consistent with the level of confidence in construction quality control and the test data, the recalibration provides a distinction between screw fasteners and welded connections for load combinations not involving wind loading. The calibration of resistance to seismic loads is based on a load factor of 1.6 and is consistent with AISC design standards. The safety factor for welded diaphragms subjected to earthquake loading, as noted above, is slightly larger than for other loading types. That factor is also slightly larger than the recalibration suggested. The increase is due to the greater toughness demands required by seismic loading, uncertainty over load magnitudes, and concern over weld quality control. When the load factor for earthquake loading is one, the 0.7 multiplier of ASCE 7 -98 is allowed in ASD and the safety factors of AISI S100, Table D5 apply. If a local code requires a seismic load factor of 1.6, the factors of Table D5 still apply.

SDI and AISI have consistently recommended a safety factor of two to limit out-of-plane buckling of diaphragms. Out-of-plane buckling is related to the steel deck panel profile, while the other diaphragm limit states are connection related. When published, the formulation for out-of-plane panel buckling in AISI S310 will also be revised based on an industry sponsored research project by S.B. Barnes Associates (Rogers and Tremblay, 2003b).

AISI S100 also allows mechanical fasteners other than screws. The diaphragm shear value using any fastener must not be based on a safety factor less than the individual fastener shear strength safety factor unless: 1) sufficient data exists to establish a system effect, 2) an analytical method is established from the tests, and 3) test limits are stated.

Concrete Filled Diaphragms. AISI S100 does not contain information for concrete filled steel deck diaphragms. For concrete filled diaphragms, and diaphragms filled with insulating concrete, the following factors are recommended by the SDI DDM.:

Factor of Safety for Diaphragms (Seismic Load) – ASD Design Any Connector	$\Omega = 3.25$
Resistance Factor for Diaphragms (Seismic Load) – LRFD Design Any Connector	$\phi = 0.50$

RP3-5.1.5.2 TM 5-809 Method – "Tri-Services Method" NAVFAC, 1992

Slightly different factors of safety for diaphragms subjected to seismic loads have been provided for ASD design using the Tri-Services Method (TSM).

Bare deck or concrete filled diaphragms with welded connectors, developed from TM 5- 809 equations and testing, where buckling does not control:

Welded Connectors $\Omega = 3.00$

Diaphragms with mechanical fasteners, developed from testing $\Omega = 2.50$

Buckling $\Omega = 2.00$

RP3-5.1.5.3 AISI S310-13

Factors of safety and resistance factors are the same as used for the SDI Diaphragm Design Manual.

RP3-5.2 Past Performance

There are few reported failures of bare steel deck diaphragms during earthquakes. The most common potential failure modes, however, are well understood and confirmed through many laboratory research investigations, as addressed in the following discussion. These potential failure modes have been taken into consideration in the design methods discussed in this resource paper. It is of value, however, for the designer to have an understanding of these behavior modes when undertaking design and detailing of steel deck diaphragms..

Bare steel deck diaphragms are typically connection dependent. A balanced design should be implemented with respect to the diaphragm strength and stiffness contributions of structural connections and sidelap connections. If structural connections of the steel deck to the supporting steel members are not sufficiently strong and stiff in comparison to the sidelap connections, an unbalanced design condition could lead to failure of the structural connections and loss of the diaphragm from the supporting steel members.

It is desirable to control steel deck diaphragm behavior through the redundant connections to the supporting steel members and between steel deck panels at the sidelaps. It would be undesirable to develop zipper type failure mechanisms at the structural connections to the supporting steel members, as this would compromise the integrity of the diaphragm.

A connection related behavior of bearing, tearing and piling up of the steel deck around the fastener shank or weld contour is the most common failure mode (identified as a Type II failure mode in AISI S905). This slotting behavior of the thin steel deck around the connections is still classified as a fastener controlled limit state, although it's the steel deck component that is deforming around the fastener. Tilting of sidelap connections consisting of mechanical fasteners such as screw fasteners is also common (identified as a Type V failure mode in AISI S905), with shear fracture of structural connections to the supporting steel beams or open web steel joists or sidelap connections (identified as a Type IV failure mode in AISI S905), less common except in thicker deck panels (of 16 gauge or thicker). Finally, out-of-plane buckling of the steel deck diaphragm can occur, especially in longer span configurations with thinner deck panels of 22 or 20 gauge. Heavier fastening patterns can contribute to this behavior when the diaphragm is subjected to higher shear loads.

RP3-5.3 Areas of Potential Improvement

While evaluations of steel deck diaphragms after a seismic event have found minimal evidence of structural failures, there are some potential areas for improvement in the seismic resistance of steel deck diaphragms.

RP3-5.3.1 Standards

Test standards and acceptance criteria for conducting cyclic tests of diaphragm connections and diaphragm systems should be developed. Placeholders have been created in the most current versions of AISI S905 and AISI S907 specifically for this purpose. Comparisons of the cyclic backbone curve to the reference static load-displacement curve (similar to how cold-formed steel shear walls are evaluated) can yield greater insight into how the static designed diaphragm will perform in a seismic event.

The AISI S310, AISI S905 and AISI S907 standards should be updated with expanded commentary on seismic design and evaluation, including testing and evaluation of diaphragm connections under cyclic seismic forces. Some manufacturers have already taken the lead on independently exploring this. Continued research should be encouraged and supported.

RP3-5.3.2 Research

RP3-5.3.2.1 Implementation of Past Research Findings

The overall goal of Tremblay's and Rogers's research (Engleder and Gould, 2010; Essa et al., 2002, Rogers & Tremblay, 2003a; Rogers and Tremblay, 2003b) was to investigate whether single story steel buildings could be constructed more cost efficiently. If a ductile response of steel deck diaphragms was demonstrated and proven, lighter designs of steel structures could be possible. Corresponding building code adoptions are necessary in order to enable utilization of this optimization of steel deck diaphragms. Such building code changes were partially implemented in the 2010 edition of the Canadian National Building Code and its reference standards. However, in the U.S. the verification of steel deck diaphragms is still done using a quasi-static design approach with application of a global safety factor as described in AISI S100 Section D5, Floor, Roof or Wall Steel Diaphragm Construction. Therefore, to date, the experimental findings have not been utilized to change U.S. diaphragm design requirements.

Experimental research conducted at Ecole Polytechnique in Montreal, Canada by Tremblay and Rogers (Guenfoud et al., 2010; Rogers and Tremblay, 2003a) and by Hilti, Inc. by Engleder and Gould (2010) have shown that power-actuated fasteners could potentially offer additional benefits with respect to steel deck diaphragm performance when subjected to cyclic, inelastic loading. Specifically, Tremblay's and Rogers's research findings compare the advantages of mechanical fastening technologies, including powder-actuated fasteners and screwed connections, over arc spot welding under seismic loading. Experimental testing of traditionally welded steel deck diaphragms showed limited energy dissipation under cyclic loading. Ultimately, the arc spot welds may not allow for load redistribution and may not be able to sustain excessive deformations without fracture. The cyclic performance of arc spot welds could potentially be improved by using weld washers. However, assurance of a proper welding protocol is required in order to achieve sufficient weld penetration to avoid premature weld fracture. Experimental investigations further showed that, from a local ductility standpoint, mechanically fastened steel deck diaphragms demonstrated additional energy dissipation and the ability to maintain load in case of excessive deformations. The use of mechanical and welded-with-washer deck-to-frame fasteners could possibly enhance the strength, ductility and energy dissipation characteristics of steel deck diaphragms.

Ultimately, the development of provisions for explicit recognition of inelastic response of mechanically connected diaphragms would be beneficial. This would build upon the research by Tremblay and Rogers and bring their findings into the building code through the applicable reference standards. This additional work could include developing methods of categorizing the magnitude of different types and configurations of steel diaphragms. These new reference standards would then need to be adopted into ICC-ES AC43 and IAPMO-UES EC-007 criteria, so that manufacturers could qualify their steel deck and fastener products through suitable testing programs.

RP3-5.3.2.2 Future Research

The current state of design practice for steel diaphragms does not consider global ductility demands, especially in diaphragm-critical seismic systems like rigid wall/flexible diaphragm systems; that is, the stressed skin panel and the chords and collectors behaving as a cohesive whole. Additional research, similar to that has occurred for precast diaphragms under seismic loading, must be performed before a true understanding of global ductility for steel diaphragms can exist.

Recent research by Essa et al. (2002) and Engleder and Gould (2010) has focused on the performance of various fastening mechanisms. However, more research is needed on all current fastening methods, because expected demands for diaphragm-critical systems like rigid wall/flexible diaphragm systems exceed current performance capabilities in all fastening methods.

RP3-6 CHAPTER 6 - WOOD-FRAME DIAPHRAGMS

This chapter discusses wood light-frame (wood-frame) diaphragms, including current practice, past performance, and areas suggested for evaluation and improvement. The primary focus of this chapter is wood structural panel (plywood and oriented strand board (OSB)) diaphragms installed on solid sawn and engineered wood framing systems. Diaphragms with cold-formed steel framing systems combined with plywood or OSB sheathing are similar in some aspects, but differ in others; this chapter does not set out to discuss these systems in depth; however, there are similarities in a number of the design aspects discussed. Sawn lumber and gypsum board sheathed diaphragms are beyond the scope of this discussion.

RP3-6.1 Current Practice

RP3-6.1.1 Common Building Types

Wood light-frame diaphragms are used in a wide range of building types and sizes. The two predominant groups include buildings constructed entirely of wood light-frame construction, and buildings with wood light-frame diaphragms combined with concrete or masonry walls.

Of the buildings constructed entirely of wood light-frame construction, a large portion is small buildings, with single-family homes of one to three stories being a majority (Figure 6-1). Medium size buildings constructed entirely of wood light-frame construction include multi-family residential buildings (Figure 6-2) and hotels, institutional buildings such as schools, and small commercial buildings (Figure 6-3). These buildings are of varying size and range from one to six stories. In addition, a number of commercial and light-industrial buildings are also constructed entirely of wood light-frame construction. These buildings often have a large plan area, but are primarily of single-story construction.

FIGURE 6-1 Single-family residential wood-frame construction

FIGURE 6-2 Multi-family residential wood-frame construction

Buildings constructed using wood light-frame diaphragms in combination with concrete and masonry walls include commercial, institutional and light-industrial buildings, predominantly of one to two stories, and "big-box" buildings with a large plan area and predominantly of single-story construction. Concrete and masonry wall buildings in some circumstances extend to several stories; when this occurs, the wood light- frame diaphragm most often is limited to the roof level, with floor diaphragms being constructed of other materials. Concrete tilt-up wall buildings (Figure 6-4) represent a significant portion of wood light-frame diaphragm building inventory, however masonry wall buildings are also common.

FIGURE 6-3 Commercial wood-frame building

FIGURE 6-4 Concrete tilt-up wall building with wood-frame roof diaphragm

Wood light-frame diaphragms have, in the past, been constructed primarily using solid-sawn wood framing members, often in combination with glued laminated timber (glulam) beams; however, engineered framing members have become more prevalent in recent construction, including prefabricated wood I-joists (Figure 6-5), prefabricated wood trusses (Figure 6-6), and a variety of structural composite lumber framing members such as laminated veneer lumber (LVL), laminated strand lumber (LSL) and parallel strand lumber (PSL). Also common is the use of steel open-web trusses, with wood chords or with steel chords to which wood nailers are fastened. The design and construction of diaphragms using engineered framing members is very much the same as for solid-sawn, with the added consideration of fastening into the wood member. Manufacturers of engineered framing often limit the size and spacing of fasteners into these members to reduce the potential for member splitting during fastener installation.

FIGURE 6-5 I-joist framing members

FIGURE 6-6 OSB floor diaphragm on engineered wood trusses

RP3-6.1.2 System Description

Wood-frame diaphragms are commonly constructed with plywood or oriented strand board (OSB) sheathing installed over a wood-frame floor or roof system, as shown in Figure 6-7. Sheathing thickness is most often chosen to meet gravity load requirements, while sheathing fastening is most often chosen based on in-plane shear demand due to wind and seismic forces.

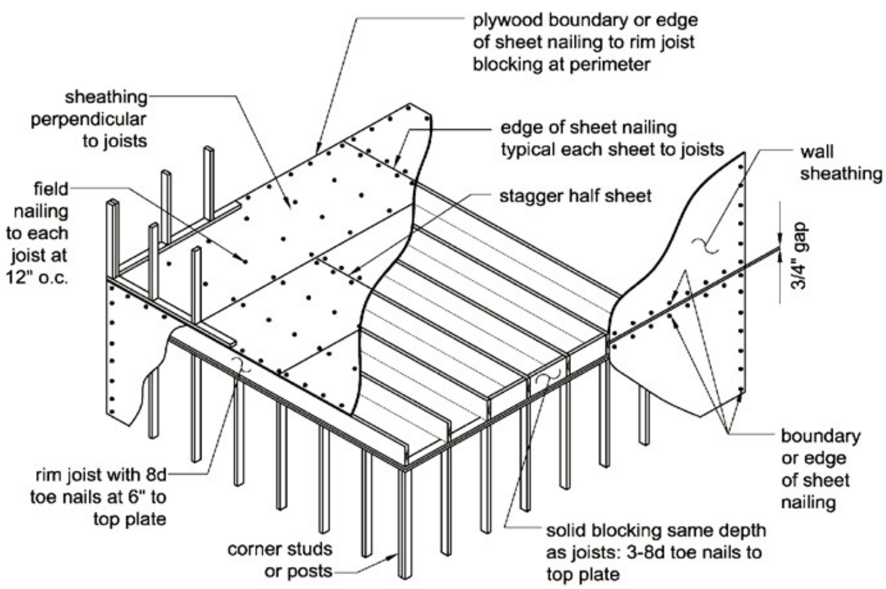


FIGURE 6-7 Unblocked wood diaphragm example (credit: FEMA 232)

When designing for in-plane diaphragm forces, the sheathing is generally modeled as resisting shear only, and chord members are provided to carry tension and compression forces analogous to beam flanges. In addition, framing members are provided to serve as collectors, transferring diaphragm forces from the sheathing to supporting vertical elements. Collectively the chord and collector elements are referred to as diaphragm boundary elements. Boundary elements are provided at nearly every edge of the diaphragm sheathing, whether at the building perimeter or interior openings, and interior to the diaphragm where required to transfer forces into or out of the diaphragm. Where shear can be transferred directly to a supporting shear wall, collectors may not be required.

It is understood that the behavior of the diaphragm is more complex than this simplistic modeling approach; this approach, however, is believed to generally result in diaphragm designs that perform adequately.

Diaphragms may be constructed using either blocked or unblocked construction. Figure 6-7 illustrates un- blocked diaphragm construction where the sheathing is fastened only at supporting joists or rafters and boundary elements. Figure 6-8 illustrates blocked diaphragm construction. In blocked diaphragm construction, wood blocking is provided at each edge of each sheathing panel, so that sheathing to framing fastening can be provided around the entire perimeter of each sheathing panel. Unblocked diaphragms are prevalent in lightly-loaded applications where the increased strength and stiffness of a blocked diaphragms are not required.

Proper specification of a diaphragm includes identifying the diaphragm as blocked or unblocked. The sheathing specification needs to include sheathing grade, thickness, span rating, span direction, and sheathing panel stagger configuration per Special Design Provisions for Wind and Seismic (SDPWS, AWC, 2008) Table 4.2A and 4.2B. The fastening specification for a diaphragm needs to include nail size and type. The nominal shear capacity tables in the SDPWS are based on use of common nails, with length to meet or exceed the tabulated minimum embedment into framing members. Nail size is specified by penny-weight (i.e. 8d = 8-penny). Diaphragm shear capacities are proportional to the nail type and size, so use of nails other than specified in SDPWS requires additional engineering justification. For unblocked diaphragms, nail spacing needs to be specified for 1) supported panel edges and diaphragm boundary elements and 2) for field nailing locations (nailing to supports not at panel edges). For blocked diaphragms nail spacing needs to be defined for 1) diaphragm boundaries and continuous panel edges, 2) other panel edges, and 3) field locations (supports not at panel edges).

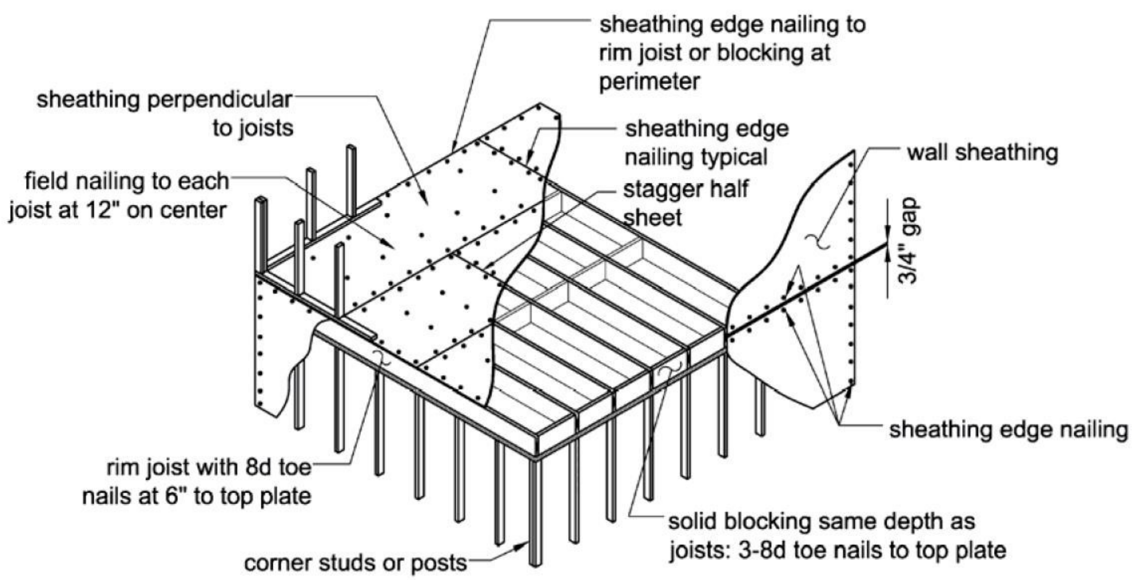


FIGURE 6-8 Blocked wood diaphragm example (credit: FEMA 232)

Diaphragm shear capacities are also a function of the specific gravity (generally a function of species) of the framing to which the sheathing is being fastened. The SDPWS tabulated values are applicable to Douglas-Fir-Larch or Southern Pine species, and adjustments to tabulated nominal capacities are required for framing members of other species based on specific gravity. Directions for adjustment are provided in footnotes to the nominal capacity tables.

In wood structural panel diaphragms with long diaphragm spans, it is not unusual for a series of nailing zones, each with differing nail spacing, to be defined. This avoids having to use close nail spacing, based on the highest diaphragm shear, for the entire diaphragm.

Detailing of the load path at all diaphragm perimeter and interior boundaries and boundary elements is also required. In smaller wood light-frame buildings with simple geometries, wall top plates often serve as the diaphragm chord and collector members, as illustrated in Figure 6-9. As geometries become more complex, provision for chords and collectors also becomes more complex, often requiring that members and connections be added to serve the function of chords and collectors. Figure 6-10 shows a chord and collector layout for a more complex diaphragm geometry. Once a complete layout of chord and collector members is developed, these members and their connections are designed for resulting tension and compression forces under applicable provisions of the National Design Specification for Wood Construction (NDS) (AWC, 2012). For buildings with concrete or masonry walls, either reinforcing steel within the walls (with proper continuity) or ledgers or other members on the face of the wall serve as the chord and collector members.

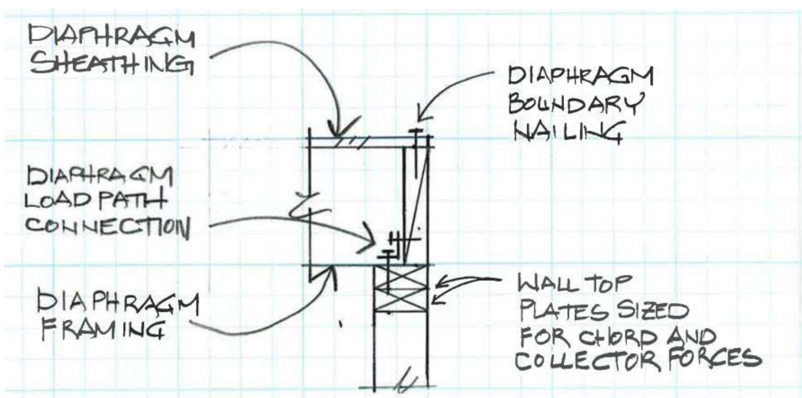


FIGURE 6-9 Diaphragm edge detail showing wall top plates serving as chord and collector

A load path from the diaphragm sheathing to all boundary elements, including the chord or collector members, is required. In some instances this is straight forward, while in others this load path takes more attention to detailing. Figure 6-9 shows load path connections from a roof diaphragm to a double top plate serving as a chord for load in one direction and a collector for load in the opposite direction. Figure 6-11 shows a similar load path for a two-story building where some of the load originates in walls above the diaphragm level shown, and is transmitted through the diaphragm to a shear wall below. Diaphragm boundary nailing is provided both at the wall above where load is transferred into the diaphragm and at the wall below where load is transferred out.

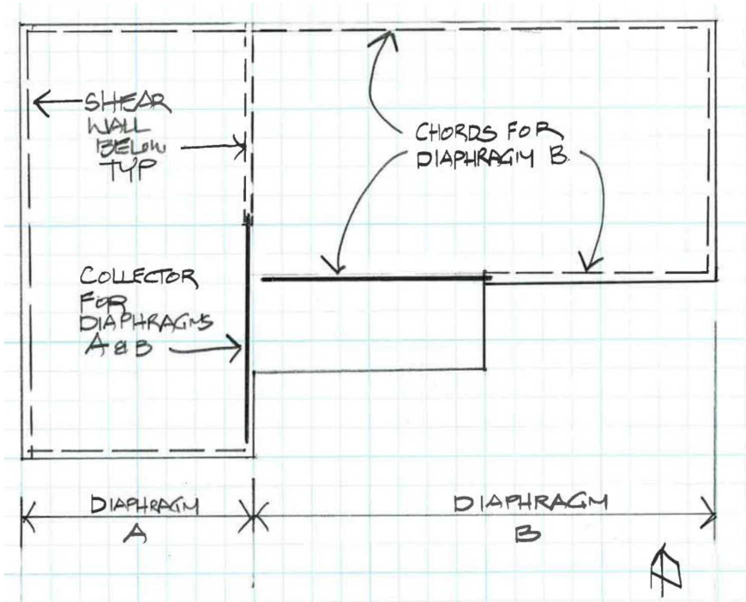


FIGURE 6-10 Diaphragm plan showing chord and collector for more complex diaphragm configuration

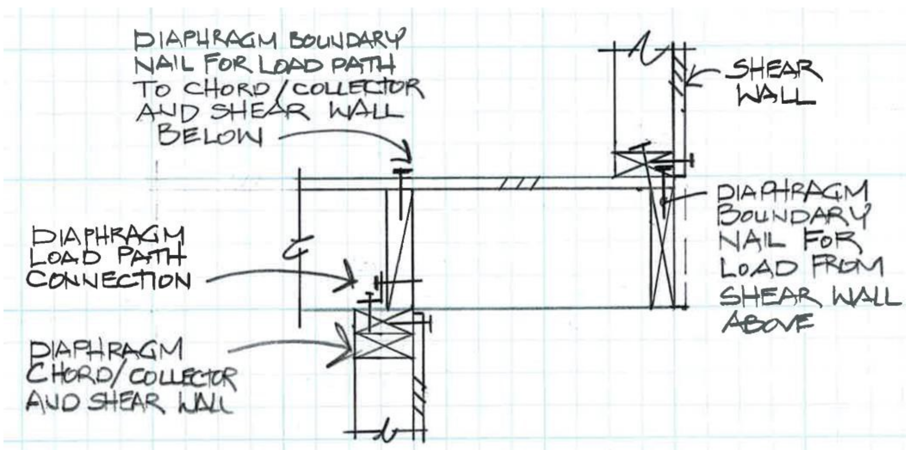


FIGURE 6-11 Diaphragm detail showing load path from shear wall above diaphragm to chord or collector

Where wood structural panel diaphragms provide out-of-plane support for concrete and masonry walls, ASCE 7 (ASCE, 2010) imposes additional requirements related to the wall anchorage to the diaphragm. Continuous ties are required from the wall anchor across the full width of the diaphragm to the far side, so that the entire diaphragm width is engaged in resisting wall anchorage forces. This requirement is based on past earthquake experience in which concrete tilt-up walls separated from the wood diaphragm, in some cases resulting in local collapse of the roof. Because wall anchorages are often spaced as close as four feet on center, and because it is inefficient to provide continuous ties across the full diaphragm width at this close spacing, the subdiaphragm has been developed as an analytical tool. The sub-diaphragm is a smaller diaphragm within the main diaphragm (Figure 6-12). Wall anchors are developed into the subdiaphragm, and continuous ties across the diaphragm are provided at the ends of each sub-diaphragm rather than at each wall anchor. Figure 6-12 illustrates subdiaphragms to anchor the east and west walls for seismic loading in the east-west direction. Similar subdiaphragms would be provided along the north and south walls for loading in the north-south direction. Figure 6-13 illustrates a typical wall anchor, anchored to a subdiaphragm roof purlin. Sheathing edge nailing is provided into the purlin as part of the load path.

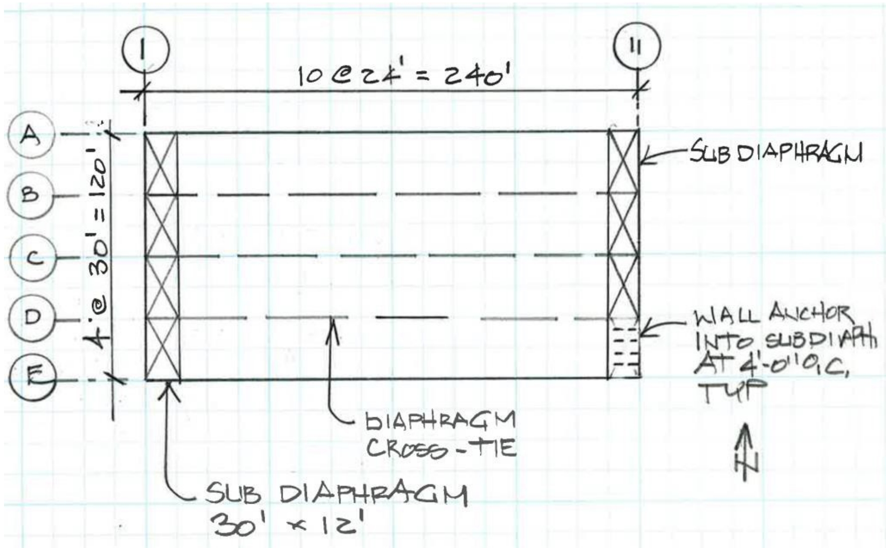


FIGURE 6-12 Plan of roof diaphragm with subdiaphragms for concrete or masonry wall seismic anchorage for loading in the east-west direction

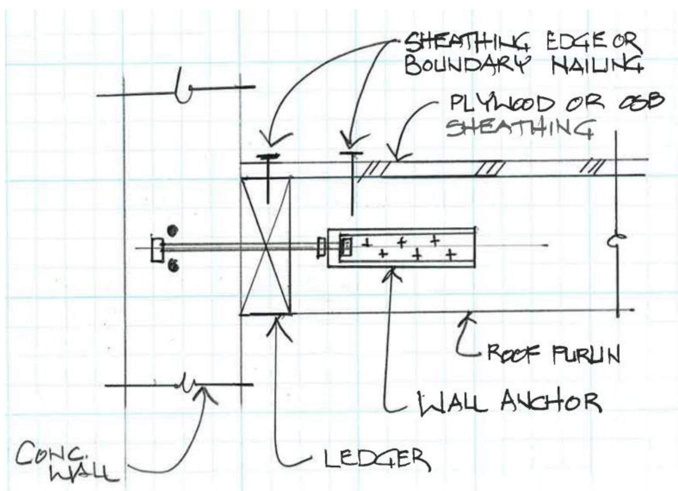


FIGURE 6-13 Concrete wall anchor to wood-frame diaphragm or subdiaphragm

RP3-6.1.3 Design Standards and Capacities

The primary design standards adopted by the International Building Code (IBC) (ICC, 2012) for design of wood structures are the NDS (AWC, 2012) and SDPWS (AWC, 2008). The SDPWS standard is the primary resource for diaphragm design requirements and diaphragm shear capacities. The NDS is used for design of boundary elements and load path connections other than the sheathing to framing fastening. Additional requirements imposed by ASCE 7 also affect diaphragm design, including specifying design force levels and additional requirements imposed on collectors and the previously described requirements for anchorage of concrete and masonry walls.

Section 4 of the SDPWS standard addresses requirements for diaphragms including requirements for a complete load path, deformation compatibility of elements and connections, requirements for boundary elements, specific limitations when resisting forces from concrete or masonry, and limits on seismic forces permitted to be carried by toe nail connections.

The SDPWS also addresses deflection limits, provides deflection calculation methods, unit shears, diaphragm aspect ratios, horizontal distribution of forces and construction requirements. Nominal design capacities for plywood and OSB sheathed diaphragms are provided for varying diaphragm types in SDPWS as follows:

- Blocked diaphragms
- High-load diaphragms
- Unblocked diaphragms

Capacities for other diaphragm sheathing materials, such as lumber and gypsum board, are provided in the SDPWS and IBC, but are beyond the scope of this discussion. Capacities provided in SDPWS are nominal values, with phi (ϕ) and omega (Ω) reduction factors identified to adjust to LRFD or ASD capacities, respectively, for use in design. Commentary to SDPWS provides background on the forgoing provisions.

Included in the International Building Code but not in the SDPWS are values for stapled diaphragms. Stapled diaphragms have been tested by APA, and stapled shear walls have been tested by a number of researchers. Because the staple leg diameter is much smaller than typical nails used for diaphragm construction, the staples greatly reduce splitting of framing members during installation of fasteners, allowing the staples to be installed at very close spacing. Staples are particularly advantageous for high- load diaphragms with close fastener spacing and for retrofit of existing diaphragms where older, drier lumber framing is more susceptible to splitting.

RP3-6.1.4 Analysis

Analysis of wood-frame diaphragms serves two primary purposes: 1) determination of forces for diaphragm design and 2) determination of the distribution of seismic and wind forces to the vertical resisting elements of the lateral force-resisting system. Analysis typically involves use of linear-elastic seismic forces and either hand calculations or simple spreadsheet type analysis tools.

The distribution of shear from the diaphragm to vertical resisting elements is based on an analysis where the diaphragm is idealized as flexible, idealized as rigid, or modeled as semi-rigid. For buildings with wood sheathed diaphragms combined with concrete or masonry walls, ASCE 7 Section 12.3.1 permits the diaphragm to be idealized as flexible, and analysis is permitted to assume the diaphragm behaves as a simple beam spanning between supporting walls as shown in Figure 6-14. ASCE 7 Section 12.3.1 also includes provisions allowing the diaphragm to be idealized as flexible for one and two-family dwellings. Further, Section 12.3.1 permits the diaphragm to be idealized as flexible in structures of light-frame construction with wood structural panel diaphragms, provided the diaphragms are untopped or have not more than 1-1/2 inch nonstructural topping and meet code required drift limits at each line of vertical elements of the lateral force-resisting system. This last item will permit use of a flexible diaphragm assumption for many wood light-frame structures; in fact, this is so broadly permissive that the 2015 SDPWS has implemented limits related to cantilevered diaphragms and diaphragms acting in rotation.

Where criteria for idealization of the diaphragm as flexible are not met, ASCE 7 Section 12.3.1 criteria will generally result in required use of semi-rigid modeling assumptions, and require the relative stiffness of both diaphragms and supporting walls to be considered in analysis. This type of analysis, however, is beyond software capabilities and design budgets of most engineering offices. Analysis using a flexible diaphragm assumption is most prevalent in engineering practice, as has long been engineering design practice. While an envelope of worst case demands using both rigid and flexible diaphragm assumptions has occasionally been required by building departments, this is generally viewed as overly conservative and not commonly used. See additional discussion of this topic in Chapter 2 of this resource paper.

Variations in flexible diaphragm modeling assumptions occur as diaphragms span between more than two shear walls, and include more complex geometries. Analytical modeling assumptions include consideration of moment continuity where diaphragms are supported at interior shear walls. Figure 6-15 illustrates one common assumption, where each diaphragm is treated as a simple span without continuity.

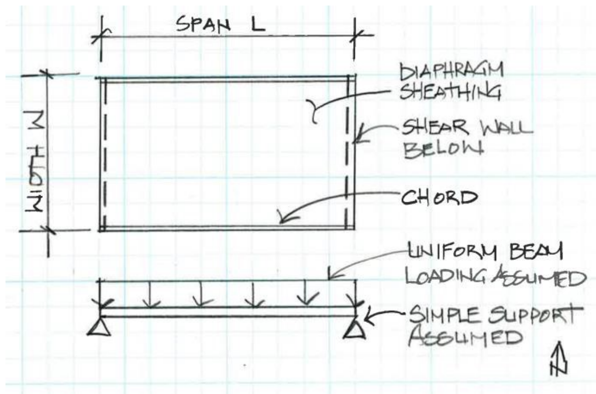


FIGURE 6-14 Common Analysis Assumption for Light-Frame Diaphragm Showing Loading in the North-South Direction

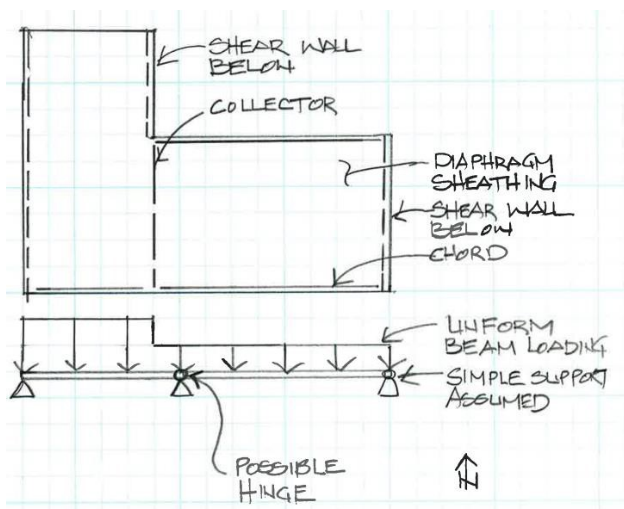


FIGURE 6-15 Common Analysis Assumptions for Multi-Span Light-Frame Diaphragm

On the occasions where computer analysis programs are used for buildings with wood light-frame diaphragms, it is common to use plate or shell elements, with the section properties derived based on the deflection equations provided in the SDPWS. More sophisticated diaphragm analysis models have been used for research efforts, with the diaphragm modeled using a series of non-linear springs for response-history analysis. The models are similar to those derived by Folz and Filiatrault for the CUREE SAWS analysis program (Folz and Filiatrault, 2002). For these studies, both pinching and degrading of the hysteresis loops are included in modeling. Figure 6-16 shows the type of hysteretic behavior used.

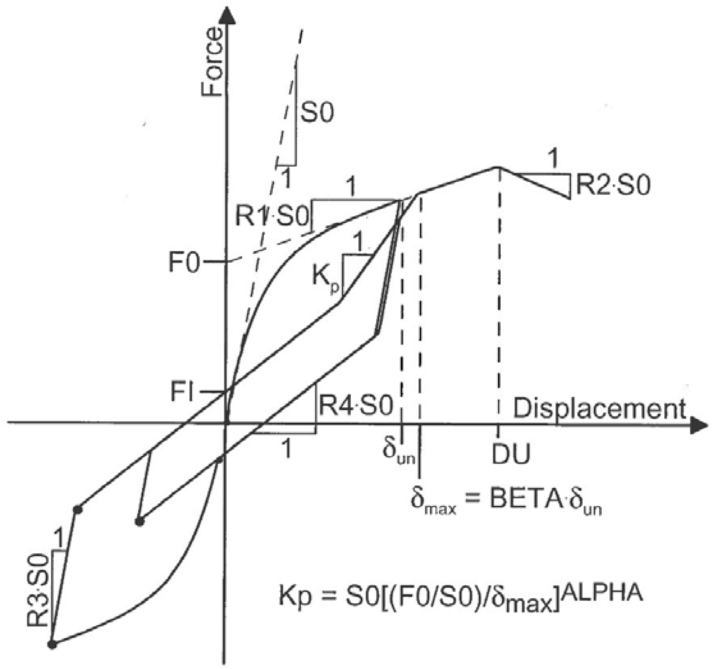


FIGURE 6-16 Wood Hysteresis Loop Showing Pinching and Degrading Behavior (Folz and Filiatrault, 2002)

RP3-6.1.5 Intended mechanism, ductility and overstrength

Like plywood and OSB sheathed shear walls, the primary mechanism of energy dissipation in the diaphragm is intended to occur in the nails fastening the sheathing to supporting framing. Significant deflection of the diaphragm can occur at peak loading. The primary sources of this deflection are yielding of the nail, nail withdrawal, and local crushing of the wood under the nail head and shank. These behaviors serve as energy dissipation mechanisms, along with energy dissipation through friction between sheathing and framing. Like plywood and OSB shear walls the diaphragms are found to have good hysteretic behavior, significant overstrength and significant deformation capacity.

As part of the PUC IT6 proposal for updated diaphragm design provisions, the ductility, drift capacity, and overstrength of diaphragms of varying construction were considered in detail. For wood structural panel diaphragms, the primary available resources for test data are four APA test reports (APA, 1966; APA, 2000; DFPA, 1954; DFPA, 1963). The available testing includes diaphragm spans (loaded as simple-span beams) ranging from 24 to 48 feet, aspect ratios ranging between 1 and 3.3, and diaphragm construction covering a range of construction types including blocked and unblocked construction, and regular and high-load diaphragms. The loading was applied with a series of point loads at varying spacing; however, this was reasonably representative of uniform loading. All available testing was monotonic, sometimes with limited load cycling. Based on shear wall loading protocol studies (Gatto and Uang, 2002), it is believed that the monotonic load-deflection behavior is reasonably representative of the cyclic load-deflection envelope, suggesting that it is appropriate to use monotonic load-deflection behavior in the estimation of diaphragm load deformation response. The diaphragms were loaded either to peak capacity or to the maximum load or displacement capability of the testing equipment.

For these diaphragms, ductility ratios (defined as the displacement at peak recorded capacity divided by an approximation of the displacement at nominal strength multiplied by the resistance factor) exhibited a wide range with a ratio of approximately 5 for many common types of diaphragm construction. The ratio of peak recorded strength to LRFD design strength ranged from 1.7 to 4 (2.4 to 5.7 for ASD). Collectively these diaphragms showed significant overstrength, ductility, and deformation capacity. A summary of the test data is provided in Appendix A to this resource paper. While parameters of overstrength, ductility, and deformation capacity are quantified based on testing it is not the intent to suggest that specific levels of each parameter are necessary for successful performance of a wood frame diaphragm. For example, level of overstrength may be the dominant factor in favorable performance of wood diaphragms as opposed to a combination of overstrength, ductility and displacement capacity.

The boundary elements and load path connections for the diaphragm must be adequate to allow development of the diaphragm demand, and if possible the peak capacity. Earthquake and testing performance to date suggests that current detailing practice accomplishes adequate performance without imposing specific capacity-based design and detailing requirements. A number of factors are thought to contribute to this performance including the high inherent overstrength present in tension and compression members and connections designed in accordance with the NDS.

RP3-6.1.6 Deflections

Deflection equations for wood structural panel diaphragms are provided in SDPWS, accounting for four primary sources of deflection of the overall diaphragm: 1) chord bending, 2) panel shear deformation, 3) panel nail slip, and 4) chord splice slip. These are provided in three terms, with the second and third sources combined into one term. Material properties used in calculation of these components of deflection are provided in SDPWS. Equations in SDPWS are applicable for the calculation of the mid-span deflection of a single span uniformly loaded diaphragm. Adjustment of the design equations is necessary for other diaphragm support and loading conditions such as cantilevered diaphragms and where loading departs from a uniform load condition.

Deflection equations provide a standard method for estimating diaphragm stiffness and are used to determine relative stiffness of the diaphragms and shear walls when needed to classify diaphragms as flexible, semi-rigid or rigid. Diaphragm deflection is a also a factor where out of plane wall deflection is limited based on permissible drift for wall systems such as masonry veneers, to provide adequate seismic building separation, and for evaluating the building configuration for presence of torsional irregularity.

RP3-6.1.7 Configuration Limits

Blocked and unblocked wood structural panel diaphragms have aspect ratio limits beyond which their use is not recognized in SDPWS provisions. For blocked construction, the maximum ratio of length to width, L/W, is 4, where W is measured parallel to the load direction under consideration, and L is measured perpendicular. For unblocked construction the maximum aspect ratio, L/W, is 3. More restrictive limitations on diaphragm aspect ratio are applied for cases involving an open front and cantilevered diaphragm where distribution of story shears occurs through diaphragm rotation (Figure 6-17).

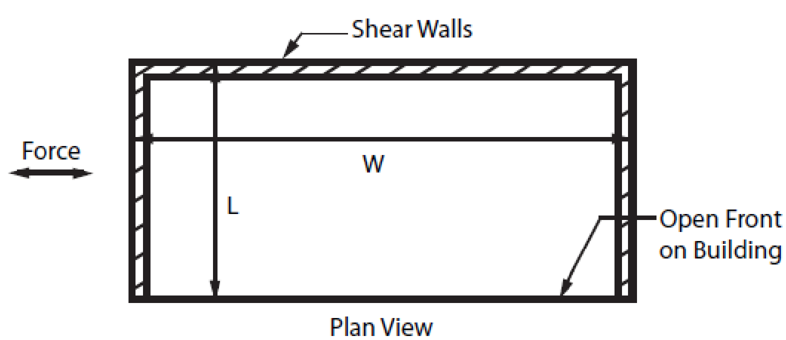


FIGURE 6-17 Diaphragm in Open Front Structure

RP3-6.1.8 Alternate Diaphragm Fastening, Sheathing and Framing

While code tables are based on fastening using standard nails or staples, a number of alternate methods of fastening are available. Use of proprietary fasteners, sheathing and/or framing in diaphragm construction is recognized in evaluation reports published by product evaluation organizations such as ICC-ES. These reports are intended to provide guidance to designers and building officials who will be considering use of alternative materials to those specified in the building code.

Use of proprietary sheathing nails and screws is most often through direct substitution with the code prescribed nail. Under direct substitution, the proprietary sheathing nails and screws are assumed to directly substitute for the code-prescribed nail assuming all other construction features remain unchanged (i.e. same framing requirements, sheathing requirements, and fastener spacing) and assuming the recognized diaphragm shear values for the code-prescribed nails are also applicable. Equivalent performance of the proprietary fasteners to the code-prescribed nail is determined by different methods to address various failure modes but generally employ either single fastener testing or small or large scale diaphragm assembly testing.

Construction adhesives have long been used in combination with nails in the construction of light-frame wood floor systems. This very common practice is recommended by APA and others as a method to mitigate floor vibration, increase floor stiffness for gravity loading, and reducing the potential for squeaking. While use of adhesives is generally recognized as increasing shear strength and stiffness of the diaphragms above that provided by specified nailing alone, the design strength and stiffness of the diaphragm determined in accordance with SDPWS is based on the specified nailing alone, neglecting the adhesive contribution. The strengthening and stiffening effects of such adhesives are not considered to be detrimental to overall diaphragm performance. While potential for more sudden loss in strength due to a combination of adhesive and wood failure is increased where adhesives are used, such a mechanism is associated with overstrength levels well beyond that provided by nailing alone.

RP3-6.1.9 Construction Issues

Adequate construction of wood-framed diaphragms requires that all of the items previously noted in Section 6.1.2 be addressed. Included are sheathing specification, nailing specification, panel layout, chords and collectors and their splices and complete load paths between the sheathing and the members serving as chords and collectors.

In addition, there are several additional aspects of construction important to performance, which deserve specific mention. First, adequate distance between the edge of the sheathing and center of the sheathing edge nailing needs to be maintained. Like shear walls, if the edge distance is less than the 3/8-inch minimum specified by SDPWS, premature tear-out of the nailing will occur, with reduced strength and reduced deformation capacity. Secondly, overdriving of the nails such that the face ply is broken under the nail head will reduce strength and deformation capacity. Finally, splitting of the framing during fastener installation can not only reduce capacity, but in some cases lead to significantly premature failure of the diaphragm. NDS provides a performance criteria to address splitting of framing. If splitting is occurring during installation, then measures need to be taken to eliminate splitting; this will usually involve predrilling holes for the nails.

The IBC requires special inspection be provided for diaphragms with moderate to high capacities to help ensure construction in accordance with the design intent and ensure that adequate performance can be anticipated. The items just discussed in this section should be verified as part of the special inspection.

RP3-6.1.10 Design Resources

The following are suggested as resources for design of wood diaphragms:

- SDPWS Commentary (AWC, 2008)
- Diaphragms and Shear Walls, Design/Construction Guide (APA, 2007)
- Design of Wood Structures (Breyer et al., 2006)
- Wood Structures chapter in Earthquake Engineering Handbook (Dolan, 2003)
- Guidelines for Seismic Evaluation and Rehabilitation of Tilt-up buildings and Other Rigid Wall/ Flexible Diaphragm Structures (SEAONC, 2001)
- SEAOC 2012 IBC Structural/ Seismic Design Manual, Volume I and Volume II (SEAOC, 2013)

RP3-6.2 Past Performance

RP3-6.2.1 Observed Earthquake Performance

Available references documenting observed earthquake damage to wood-frame buildings in general lack mention of damage to wood-frame diaphragms. In Earthquake Engineering (Bozorgnia and Bertero, 2004), the discussed survey of literature on California Earthquakes from the 1971 San Fernando to the 1994 Northridge Earthquakes identified a number of sources of damage to wood-frame buildings, primarily associated with irregular building configurations such as unbraced cripple walls, soft stories, hillside buildings, and open front building torsional irregularities. Diaphragm damage is not discussed as an observed performance issue; references include ATC, 1976; EERI, 1989; SEAOC, 1991; EERI 1994; EERI, 1996; LATF, 1994a, 1994b, 1994c; Hamburger, 1994; and Seismic Safety Commission, 1994. While this should not be taken as an indication that there was no diaphragm damage, it can be said that diaphragm damage did not shown up as a significant item of concern meriting modification of design practice.

One type of wood diaphragm building that does have a history of earthquake damage is the large plan area building with the combination of heavy concrete or masonry walls and a wood diaphragm. Although damage has been most commonly observed in buildings with concrete tilt-up walls, the damage has extended to similar buildings with masonry and cast-in-place concrete walls. The damage has, however, been almost exclusively associated with the anchorage of the walls to the diaphragm to resist wall out-of- plane seismic forces. In fact, damage from such wall anchorages has been observed in every earthquake where this type of construction experienced moderate to major ground shaking, going back to the 1964 Anchorage, Alaska Earthquake. A comprehensive resource discussing the past earthquake performance and recommended rehabilitation of this building type is Guidelines for Seismic Evaluation and Rehabilitation of Tilt-up buildings and Other Rigid Wall/ Flexible Diaphragm Structures (SEAONC, 2001). This publication provides a comprehensive discussion of past earthquake performance and the development of code requirements for anchorage design. Also included is a chapter specifically discussing diaphragms, in which a brief mention of past diaphragm performance is included. The following is said:

"To date, high shears or excessive deflections in diaphragms have not been major contributors to damage observed in earthquakes. However, localized damage at re-entrant corners and at corners of large openings has been observed. Tearing or debonding of roofing from the diaphragm in areas of high shear have also been observed.

Cases where diaphragm boundary nailing has failed were observed in the Northridge Earthquake, but typically the failures were attributed to inadequate wall anchorage, which allowed out-of-plane forces to be transferred to the nails, causing failures under the combined action of shear and tension. Similarly, some damage to struts and/or collectors can be linked to diaphragm nailing…"

It is of significance that even in these buildings that put significant demand on diaphragms, damage and poor performance of the diaphragms has not shown up as an issue of significance. Additional discussion of damage to this type of building was addressed in a workshop held by the Applied Technology Council (ATC, 1979).

RP3-6.2.2 Observed Testing Performance

In addition to observations of past earthquake performance, testing of wood-frame buildings and components has contributed to understanding of wood-frame building earthquake performance. Over the past 20 years a number of full-building shake table tests have been conducted on wood-frame buildings, including CUREE-Caltech Woodframe Project testing (Fischer et al., 2001; Mosalam et al., 2002) NEESWood testing (Christovasilis et al, 2009; Pei et al., 2010) and most recently NEESSoft testing (just completed, unpublished at this time). In these tests, observed indications of inelastic behavior of note have been limited to the vertical elements of the lateral force-resisting system, with no observations made of noticeable deterioration in the horizontal diaphragms. The diaphragms in tested buildings have been limited to short to moderate diaphragm spans, and in general, diaphragms with overstrength capacity.

Much of the component testing that has been conducted on diaphragms has been conducted by APA-The Engineered Wood Association (including under previous names including the American Plywood Association and the Douglas Fir Plywood Association). This testing was introduced in Section 6.1.5 of this chapter as the source for load-deflection data for quantification of diaphragm behavior. This testing has been the basis for derivation of allowable diaphragm capacities provided in building codes and standards, as well as derivation of deflection equations. Available in the APA test reports are descriptions of behavior of the tested diaphragms at either peak capacity or peak test capacity. As discussed in Section 6.1.5, plywood and OSB diaphragms are found to provide stable hysteretic behavior and good strength and deformation capacity, with the ductility of diaphragms understood to be approximately on par with that of plywood and OSB shear walls.

Diaphragm component testing was conducted by Dolan et. al. (2003) to determine the stiffness of diaphragms in the small-deformation range in order to provide load-deflection relationships for use by designers. Included in the testing were diaphragms with and without blocking, with and without chords, with and without adhesive, and with several different opening locations in the diaphragms. Formulas for calculating diaphragm deflections for these variations in construction were derived. Additional diaphragm-related testing includes Ficcadenti et al. (2003), investigating load path connections between shear walls and diaphragms.

RP3-6.3 Areas of Potential Evaluation/ Improvements

RP3-6.3.1 Future Directions

Evolving analysis tools combined with testing provide opportunities to better understand wood diaphragm performance, evaluate the adequacy of performance resulting from current design methods, and explore improvement of those methods. In considering this, four primary inter-related areas of interest for future directions are discussed below:

Seismic force demands on wood-frame diaphragms and adequacy of design methods need to be better understood. Two ongoing efforts have been investigating this issue: 1) BSSC IT6 work on development of design methodologies assigning diaphragm force demands considering the behavior of the diaphragm through a diaphragm design force reduction factor, as discussed in Section 2.2 of this resource paper, and 2) the ongoing investigation on design approach for diaphragms in big-box structures with concrete or masonry walls and light-frame diaphragms (NEHRP, 2015). IT6 investigated anticipated seismic demands for diaphragms ranging from near elastic to inelastic behavior. As a part of this effort, a limited analytical study was conducted using wood diaphragms (Zhang, 2013) in which it was found that the significant displacement capacity of wood diaphragms, along with associated overstrength, tended to greatly reduce forces from those anticipated with near elastic diaphragm response; however, more rigorous studies are needed. In order to support more rigorous studies, additional hysteretic test data would be of benefit to verify hysteretic behavior and validate analysis models, since most test data currently available is monotonic and does not necessarily capture peak strength and deformation capacity. This further information would allow verification of the diaphragm methodology and diaphragm response factor recommended in the IT6 Provisions Part I recommendations. The NIST effort is focused on developing design guidance for buildings in which the seismic response is primarily driven by the diaphragm behavior rather than the vertical elements.

Seismic deformation demands need to be better understood. It is recognized that wood diaphragms are neither flexible nor rigid, but rather semi-rigid and highly non-linear in behavior. Understanding of the load-deflection behavior is key to: 1) understanding deformation demands for purposes of modeling for analytical studies to explore diaphragm demands, 2) understanding deformation demands in order to explore design limits necessary for performance of the diaphragm and attached building components, and 3) understanding load-deflection behavior for analytical modeling to identify force distribution methods that will provide adequate structure performance.

Diaphragm seismic design that will result in adequate building performance needs to be understood. If the performance resulting from current design practice can be understood, then the need to modify this performance can be judged. The goal of our seismic analysis and design is obtaining acceptable behavior with a reasonably economic solution and design effort. Are capacities and ductilities currently adequate? Is expected inelastic behavior acceptable? Design approaches need to provide for performance, not exact solutions, which are by definition unobtainable.

If diaphragm force demand, deformation demand and design approach can be better understood, a rationalization of detailing practice for diaphragms can follow. At this time there are a series of prescriptive rules that have evolved out of past practice without a rigorous development of justification; at this time we do not know whether the rules will systematically provide acceptable performance and support desired diaphragm behavior. In addition, performance criteria to replace the prescriptive design and detailing criteria are desired to allow the designer more flexibility in design and detailing. Prescriptive limits of interest include: diaphragm aspect ratio, geometric limitations on cantilevered diaphragms and diaphragms in open-front structures, and continuous wall ties required in structures with concrete or masonry walls. Also requiring rationalization are the current requirements for design of diaphragm chords and collectors. Current design practice does not include an explicit requirement for capacity design of these boundary members in order to force inelastic behavior into the sheathing nailing; it is generally felt, however, that current design practice and design standards do support this behavior. Rigorous evaluation of this has not occurred to date.

In summary, if a better understanding of diaphragm force and deformation demand can be obtained through testing and analytical studies, a systematic evaluation of design practice including proportioning of diaphragms, force distribution in structures, prescriptive limits and detailing practice should follow. The described areas of interest are interrelated and must be addressed in combination.

RP3-6.3.2 Sub Diaphragm Clarification

During development of this chapter it was discovered that ASCE 7 Section 12.11.2.2.1 aspect ratio limits for subdiaphragms, initially developed for wood-frame diaphragms supporting concrete and masonry walls (and particularly tilt-up walls), had been extended to be applicable to all subdiaphragms materials. IT6 provided a clarification that this requirement was intended for wood and cold-formed steel framed diaphragms with wood structural panel sheathing that serve as part of the anchorage for concrete and masonry walls. In 1997 UBC, this requirement only applied to wood diaphragms, in 2000 IBC, 1997 to 2003 NEHRP, and ASCE 7-98, 7-05 and 7-10, it applied to all diaphragm types. An ICBO code change submittal by Kariotis (1631.2.8-95-1-K.A.S.E.- [Lat.]), supplemented by SEAOC Bluebook discussion (SEAOC, 1990), gives the available commentary. This IT6 proposed clarification has been incorporated into the 2015 IBC and provided to ASCE 7 for their consideration.

RP3-6.3.3 Chord Design for Multi-Span

Designers currently make a variety of assumptions regarding chord forces in wood diaphragms at interior supports, including full force continuity as in a continuous beam, hinging to allow single-span diaphragms to each side, and partial continuity solutions. At this time the impact of these modeling decisions on diaphragm performance is not known. As force and deformation behavior of diaphragms is better understood, the influence of the modeling decisions on diaphragm performance and recommendations for practice should be developed and provided to the design community.

RP3-6.3.4 Design for Openings

Available guidance for design of diaphragms with openings has evolved primarily from testing by APA (APA, 2007). Dolan et al. (2003) provided additional test information to evaluate stiffness effects of openings, however the impact of modeling and detailing decisions on performance of diaphragms with openings has not been studied in detail. As force and deformation behavior of diaphragms is better understood, the influence of the modeling decisions on diaphragm performance and recommendations for practice should be developed and provided to the design community.

RP3-7 CHAPTER 7 - CONCLUDING REMARKS

In the preparation of this resource paper, a number of future needs or future improvements concerning diaphragm design methodology were identified. Each of the material-specific chapters provides a summary of the needs identified for that material. A primary future need for global issues is the further development of the alternate diaphragm design force methodology introduced in Section 2.2 of this resource paper and addressed in the 2015 NEHRP Provisions Part 1 recommended provisions and Part 2 commentary. In order for the alternate method to be used beyond the limited systems currently described, the development of a methodology for determination of diaphragm force reduction factors, R_S, is required. This needs to be a large-scale effort along the lines of development of the FEMA P-695 and P-795 methodologies. The development of testing and analysis procedures are required.

There is currently an ongoing effort considering design direction for big-box type buildings, having long- span flexible diaphragms and concrete or masonry walls. The seismic response of these buildings is unique in that it is greatly influenced by the diaphragm flexibility, rather than being primarily dependent on the stiffness of the vertical elements of the lateral force-resisting system. Recommendations for design and detailing of this building type are anticipated in the near future.

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RP3-APPENDIX A

Plywood Diaphragm Strength and Deformation Capacity Data from Past Testing.

Appendix A
Plywood Diaphragm Strength and Deformation Capacity Data from Past Testing

	1: Rated sheathing, blocked, nailed												2: STR I, blocked, nailed			3: Rated sheathing, unblocked, nailed						
Group	1	1	1	1	1	1	1	1	1	1	1	1	2	2	2	3	3	3	3	3	3	3
Entry Number	1	2	3	5	6	7	16	18	19	20	21	31	12	15	27	4	8	9	10	11	23	24
Report1 - Specimen	55-1	55-2	55-3	63-A	63-B	63-C	106-5	106-7(a)	106-8 (a)	106-9 (b)	106-10	138-1	106-1	106-4	106-15	55-4	63-D	63-E	63-H	63-J	106-12	106-12A
Span (ft)	40	40	40	24	24	24	48	48	48	48	48	48	48	48	48	40	24	24	24	24	48	48
Depth (ft)	20	20	12	24	24	24	16	16	16	16	16	16	16	16	16	12	24	24	24	24	16	16
Aspect Ratio	2.0	2.0	3.3	1.0	1.0	1.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.3	1.0	1.0	1.0	1.0	3.0	3.0
Loading points (ft o.c.)	8	8	8	8	8	8	3	3	3	3	3	2	3	3	3	8	8	8	8	8	3	3
Sheathing (in)	1/2	1/2	1/2	3/8	3/8	3/8	3/8	3/8	3/8	1/2	1/2	1/2	3/8	3/8	3/8	1/2	3/8	3/8	3/8	3/8	1-1/8	1-1/8
Rated or STR I	Rated	Rated	Rated	Rated	Rated	Rated	Rated	Rated	Rated	Rated	Rated	Rated	STR I	STR I	STR I	Rated	Rated	Rated	Rated	Rated	Rated	Rated
Blocked	yes	yes	yes	yes	yes	yes	yes	yes	yes	yes	yes	yes	yes	yes	yes	no	no	no	no	no	no	no
Unblocked Case	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	1	1	3	4	2	1	1
Opening (ft)																						
High Load	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no	no
Fastening Type	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	common nail	ring shank	ring shank
Nail Size	8	10	8	8	8	8	8	8	8	8	10	10	8	8	8	8	8	8	8	8	8	8
Nail Spacing	6/6/12	4/6/12	6/6/12	6/6/12	6/6/12	6/6/12	4/6/12	4/6/12	4/6/12	4/6/12	4/6/12	4/6/12	4/6/12	4/6/12	4/6/12	6/6/12	6/6/12	6/6/12	6/6/12	6/6/12	6/6/6	6/6/6
vn (plf)	540	720	540	480	480	480	640	640	640	720	770	850	720	720	720	480	430	320	320	320	570 (f)	570 (f)
Phi*vn (plf)	432	576	432	384	384	384	512	512	512	576	616	680	576	576	576	384	344	256	256	256	456	456
Peak test shear (plf)	1388	1920	1750	1392	1490	1489	1115	1120	1125	1380	1435	1788 (n)	1350	1160	1728	1400	1042	733	822	814	1135	1220
0.4*peak test shear (in)	555	768	700	557	596	596	446	448	450	552	574	711	540	464	691	560	417	293	329	326	454	488
Defl at 0.4*peak test (in)	0.43	0.36	0.50	0.25	0.21	0.37	0.35	0.34	0.44	0.44	0.43	0.53	0.37	0.35	0.42	0.70	0.33	0.32	0.22	0.22	0.25	0.25
Defl at phi*vn (in)	0.33	0.27	0.31	0.17	0.14	0.24	0.40	0.39	0.50	0.46	0.46	0.51	0.39	0.43	0.35	0.48	0.27	0.28	0.17	0.17	0.25	0.23
Defl at peak test (in)	2.74	2.71	3.85	1.70	1.20	2.50	2.36	2.64	2.65	3.49	3.39	3.90	2.54	2.40	2.83	4.57	2.20	1.70	1.90	1.40	3.61	4.21
Defl at post peak (in)																						
Ductility	8.19	10.04	12.48	9.86	8.87	10.48	5.87	6.79	5.29	7.60	7.35	7.70	6.44	5.52	8.09	9.52	8.08	6.08	11.09	8.09	14.38	18.02

peak strength/ phi*vn	3.21	3.33	4.05	3.63	3.88	3.88	2.18	2.19	2.20	2.40	2.33	2.61	2.34	2.01	3.00	3.65	3.03	2.86	3.21	3.18	2.49	2.68
peak strength/ vn	2.57	2.67	3.24	2.90	3.10	3.10	1.74	1.75	1.76	1.92	1.86	2.10	1.88	1.61	2.40	2.92	2.42	2.29	2.57	2.54	1.99	2.14
	Group 1												Group 2			Group 3						

Ductility		Ductility		Ductility	
Average	8.38	Average	6.68	Average	10.75
Max	12.48	Max	8.09	Max	18.02
Min	5.29	Min	5.52	Min	6.08
Std Dev	2.06	Std Dev	1.30	Std Dev	4.16
Avg-StDv	6.31	Avg-StDv	5.38	Avg-StDv	6.60
Min	5.29	Min	5.52	Min	6.08

Footnote 1: 55 = DFPA, 1963
63 = DFPA, 1954
106 = APA, 1966
138 = APA, 2007

December 15, 2014

Appendix A
Plywood Diaphragm Strength and Deformation Capacity Data From Past Testing

4: STR I, short nails		5: Close nail spacing			6: Stapled blkg		7: Teks to steel		8: Openings		9: Glued		10: High load nailed		11: high load stapled	
4	4	5	5	5	6	6	7	7	8	8	9	9	10	10	11	11
13	14	17	22	28	25	26	29	30	33	34	35	36	39	39	37	38
106-2	106-3	106-6	106-11	106-16 (d)	106-13	106-14	106-17 (e)	106-18 (e)	138-3	138-4	138-5	138-6	138-9	138-10	138-7	138-8
48	48	48	48	48	48	48	48	48	48	48	48	48	48	48	48	48
16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16
3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2	2
3/8	3/8	3/8	1/2	3/4	1-1/8	1-1/8	3/4	3/4	1/2	1/2	1/2	1/2	3/4	3/4	5/8	3/4
STR I	STR I	Rated	Rated	STR I	Rated	Rated	STR I	STR I	STR 1	STR 1	STR 1	STR 1	STR 1	STR 1	STR 1	STR 1
yes	yes	yes	yes	yes	stapled	stapled	no	no	yes	yes	yes	yes	yes	yes	yes	yes
NA	NA	NA	NA	NA	NA	NA	3	3	NA	NA	NA	NA	NA	NA	NA	NA
									2 - 4'x4'	2 - 8'x8'						
no	no	no	no	no	no	no	no	no	no	no	no	no	yes/ zoned	yes/ zoned	yes	yes/ zoned
short common	short common	common nail	common nail	common nail	ring shank	ring shank	Teks Screw	Teks Screw	common nail	common nail	structural adhesive	structural adhesive	common nail	common nail	staples	staples
8	8	8	10	8	8	8	10	10	10	10	NA	NA	10	10	14 Ga	14 Ga
4/6/12	4/6/12	2/3/12	2/3/12	2.5/4/12	4/6/6	2/3/6	16/16/16	6.5/6.5/16	4/6/12 (m)	4/6/12 (m)	NA	NA	3@3/2@3/6	3@3/2@3/6	3@2/3@3/6	3@1.5/2@1.5/3
720	720	1090	1310	1060 (g)	NA	NA	NA	430 (h)	850	850	980 (o)	980 (o)	3060 (p)	3060 (p)	2950 (h)	2950 (h)
576	576	872	1048	848	NA	NA	NA	344	680	680	784	784	2448	2448	2360	2360
1155	1120	1660	1860	2960	2050	2910	600	720	1314 (n)	1482 (n)	2359	2624	4668 (l)	4946	3925	5234
462	448	664	744	1184	820	1164	240	288	526	593	944	1050	1867	1978	1570	2094
0.30	0.41	0.50	0.57	0.56	NA	NA	NA	0.17	0.49	0.57	0.37	0.65	0.91	0.88	0.65	0.91
0.37	0.53	0.66	0.80	0.40	NA	NA	NA	0.20	0.63	0.65	0.31	0.49	1.19	1.09	0.98	1.03
1.97	2.63	2.66	3.08	3.77	1.94 (c)	1.88 (c)	0.96	1.14	3.30	3.75	1.20	1.25	2.78	2.08	3.00	3.23
5.27	4.99	4.05	3.84	9.40	NA	NA	NA	5.61	5.21	5.74	3.90	2.57	2.33	1.91	3.07	3.15
2.01	1.94	1.90	1.77	3.49	NA	NA	NA	2.09	1.93	2.18	3.01	3.35	1.53	2.02	1.66	2.22
1.60	1.56	1.52	1.42	2.79	NA	NA	NA	1.67	1.55	1.74	2.41	2.68	1.53	1.62	1.33	1.77
Group 4		Group 5			Group 6		Group 7		Group 8		Group 9		Group 10		Group 11	

Ductility		Ductility		Ductility		Ductility		Ductility		Ductility		Ductility	
Average	5.13	Average	5.76	Average	5.61	Average	5.47	Average	3.24	Average	2.61	Average	3.11
Max	5.27	Max	9.40	Max	5.61	Max	5.74	Max	3.90	Max	3.15	Max	3.15
Min	4.99	Min	3.84	Min	5.61	Min	5.21	Min	2.57	Min	1.91	Min	3.07
Std Dev	0.20	Std Dev	3.15	Std Dev		Std Dev	0.37	Std Dev	0.94	Std Dev	0.60	Std Dev	0.06
Avg-StDv	4.93	Avg-StDv	2.61	Avg-StDv		Avg-StDv	5.10	Avg-StDv	2.30	Avg-StDv	2.02	Avg-StDv	3.05
Min	4.99	Min	3.84	Min	5.61	Min	5.21	Min	2.57	Min	1.91	Min	3.07

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RESOURCE PAPER 4
UPDATED MAXIMUM-RESPONSE SCALE FACTORS

RP4-1 UPDATED MAXIMUM-RESPONSE SCALE FACTORS

The proposed changes below update the "maximum-response scale factors" specified in the site-specific ground motion procedures (Chapter 21) of ASCE/SEI 7-10. These factors increase spectral response accelerations that represent the geometric mean (or a similar metric) of two horizontal ground motion components, such that they represent the maximum response in the horizontal plane. Recall that ASCE/SEI 7-10, via both Chapter 21 and the MCE_R ground motion maps, specifies maximum-response spectral response accelerations. Typical ground motion attenuation relations, including those applied by the USGS in preparing the MCE_R ground motion maps, provide geometric-mean spectral response accelerations.

The values of the updated maximum-response scaling factors proposed herein are illustrated in Figure 1, where they are compared with those in ASCE/SEI 7-10. Also illustrated in the figure are the ratios of the updated factors to the ASCE/SEI 7-10 factors. Note, in particular, that the updated factors at 0.2 and 1.0 s (i.e., 1.2 and 1.25, respectively) are approximately 10% larger and 5% smaller than the respective factors in ASCE/SEI 7-10 (i.e., 1.1 and 1.3).

This resource paper was originally a Provisions Part 1 proposal for change, prepared by the BSSC 2015 Issue Team (IT) 11 on Seismic Design Mapping. That proposal received enough affirmative votes from the 2015 Provisions Update Committee (PUC) to pass, but via responses to the PUC comments on the proposal, IT 11 recommended (and the PUC approved) interim placement of the proposed changes in Part 3 of the Provisions, for the reason that follows. In the course of developing a separate proposal to add longer (than 1 s) period points to the design response spectrum of Section 11.4.5, IT 11 found that, for Site Classes D and E, the longer-period part of the ASCE/SEI 7-10 spectrum is generally un-conservative at high-hazard locations, in some cases by a factor of two. The IT also found that adding longer periods to the response spectrum instigates other issues (e.g., a need to also add shorter periods) that require more research before additional (to 0.2 and 1.0 s) periods can confidently be proposed. In the interim, IT 11 recommended against changing to the updated maximum-response scale factors presented below, which would **decrease** spectral response accelerations at longer periods relative to ASCE/SEI 7-10 (e.g., by approximately 15% at 5 seconds). By placing the updated factors in Part 3 of the Provisions, they can be enacted concurrently (e.g. for the MCE_R ground motion maps) with a future change to the aforementioned un-conservative longer-period part of the ASCE/SEI 7-10 design response spectrum.

The impetus for updating the maximum-response scale factors is the recent research of the Pacific Earthquake Engineering Research Center (PEER) NGA-West2 Directionality Working Group, the results of which are published in (Shahi and Baker, 2013 and 2014). The factors in these twin publications, which are illustrated in Figure 1, include the improvements listed below relative to the factors in ASCE/SEI 7-10, as discussed by IT 11 via email and numerous web conferences. One of the web conferences was joined by both Jack Baker (the chair of the PEER NGA-West2 Directionality Working Group) and Andrew Whittaker, who was the task leader for the development of the maximum-response scale factors in ASCE/SEI 7-10.

1. The updated factors are consistent with the new spectral response acceleration parameter ("RotD50") provided by the NGA-West2 attenuation relations, the relations that have been applied for shallow crustal earthquakes in preparing updated MCE_R ground motion maps for the 2015 Provisions. At long periods (greater than 1 s), RotD50 is approximately 5% larger than the spectral response acceleration parameter that is implicit in the MCE_R ground motion maps of ASCE/SEI 7-10 ("GMRotI50" from NGA-West1). This is part of the reason why, at long periods, the updated factors are smaller than those of ASCE/SEI 7-10.

2. The updated factors are based on approximately 40% more near-fault strong ground motion data (with source-to-site distances of 15km or less and earthquake magnitudes of 6.5 and greater) compared to the factors in ASCE/SEI 7-10. The additional data, from the NGA-West2 database, include ground motions from recent earthquakes in New Zealand, Japan, and China.
3. The updated factors are insensitive to the inclusion or exclusion of the overwhelmingly-large number of ground motion data from the 1999 Chi-Chi earthquake, which generated short-period spectral response accelerations that, on average, are relatively small compared to other similar earthquakes. In contrast, the Huang et al. (2008) factors upon which those in ASCE/SEI 7-10 are based were reported to substantially increase at short periods (less than 1 s), and slightly decrease at long periods, with the exclusion of the Chi-Chi data. This is much of the reason why, at short periods, the updated factors are larger than those of ASCE/SEI 7-10. It is also part of the reason why, at long periods, the updated factors are smaller than those of ASCE/SEI 7-10.
4. The updated factors are based on ratios of maximum-response data to geometric-mean data ("data-to-data" ratios), rather than the ratios of maximum-response data to geometric-mean attenuation relation predictions ("data-to-predictions" ratios) that the ASCE/SEI 7-10 factors are based on. This is likely the reason that, as described in the preceding bullet, the updated factors are insensitive to the Chi-Chi data. Note that in addition to data-to-prediction ratios, Huang et al. (2008) also present data-to-data ratios that are similar to the updated factors proposed herein.
5. The updated factors have been shown by Shahi and Baker (2013) to be practically independent of earthquake magnitude, source-to-site distance, and near-fault directivity parameters. Hence, the maximum-response ground motions resulting from the updated factors (e.g., on the MCE_R ground motions maps) can be thought as those that would result from attenuation relations for maximum response. In fact, this is why the PEER NGA-West2 Directionality Working Group developed maximum-response scaling factors rather than directly developing attenuation relations for maximum response.

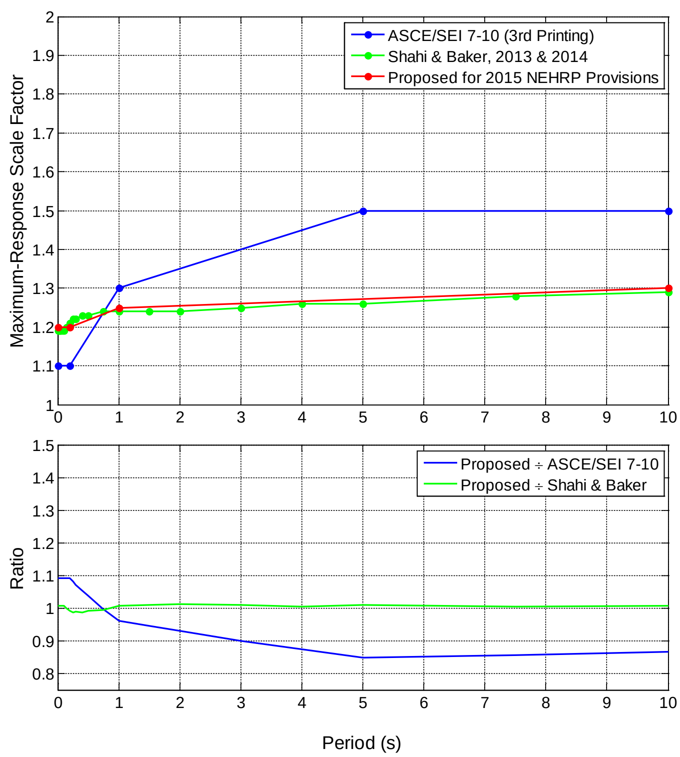

FIGURE 1 A Comparison of the Updated Maximum-Response Scale Factors Proposed herein with those in ASCE/SEI 7-10 (Chapter 21), and those Published in (Shahi and Baker, 2013 and 2014)

RP4-2 PROPOSED CHANGES:

RP4-2.1 Replace ASCE/SEI 7-10 Section 21.2 with the following:

21.2 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Hazard Analysis. The requirements of Section 21.2 shall be satisfied where a ground motion hazard analysis is performed or required by Section 11.4.7. The ground motion hazard analysis shall account for the regional tectonic setting, geology, and seismicity, the expected recurrence rates and maximum magnitudes of earthquakes on known faults and source zones, the characteristics of ground motion attenuation, near source effects, if any, on ground motions, and the effects of subsurface site conditions on ground motions. The characteristics of subsurface site conditions shall be considered either using attenuation relations that represent regional and local geology or in accordance with Section 21.1. The analysis shall incorporate current seismic interpretations, including uncertainties for models and parameter values for seismic sources and ground motions. If the spectral response accelerations predicted by the attenuation relations do not represent the maximum response in the horizontal plane, then the response spectral accelerations computed from the hazard analysis shall be scaled by factors to increase the motions to the maximum response. If the attenuation relations predict the geometric mean or similar metric of the two horizontal components, then the scale factors shall be: 1.2 for periods less than or equal to 0.2 sec; 1.25 for a period of 1.0 sec., and, 1.3 for periods greater than or equal to 10.0 sec., unless it can be shown that other scale factors more closely represent the maximum response, in the horizontal plane, to the geometric mean of the horizontal components. Scale factors between these periods shall be obtained by linear interpolation. The analysis shall be documented in a report.

RP4-2.2 Replace ASCE/SEI 7-10 Section C21.2 with the following:

C21.2 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Hazard Analysis. Site-specific risk-targeted maximum considered earthquake (MCE_R) ground motions are based on separate calculations of site-specific probabilistic and site-specific deterministic ground motions.

Both the probabilistic and deterministic ground motions are defined in terms of 5 percent damped spectral response in the maximum direction of horizontal response. The maximum direction in the horizontal plane is considered the appropriate ground motion intensity parameter for seismic design using the equivalent lateral force (ELF) procedure of Section 12.8 with the primary intent of avoiding collapse of the structural system.

Most ground motion relations are defined in terms of average (geometric mean) horizontal response. Maximum response in the horizontal plane is greater than average response by an amount that varies with period. Maximum response may be reasonably estimated by factoring average response by period-dependent factors, such as 1.2 at short-periods and 1.25 at a period of 1.0 seconds. These and the other period-dependent factors specified in Section 21.2 are updates of the factors in ASCE/SEI 7-10 and the 2009 *Provisions*. The updated factors are based on (Shahi and Baker, 2013 and 2014), whereas the previous factors were based on (Huang et al., 2008). The improvements included in the updated factors are i) consistency with the new spectral response acceleration parameter of updated ground motion relations for shallow crustal earthquakes (e.g., Boore et al. 2013); ii) additional strong ground motion data from recent earthquakes in New Zealand, Japan, and China (Ancheta et al, 2013); iii) insensitivity to the inclusion or exclusion of the overwhelmingly-large number of strong ground motion data from the 1999 Chi-Chi earthquake, which generated short-period spectral response accelerations that, on average, are relatively small compared to other similar earthquakes; iv) basis in ratios of maximum-response data to average-response data, as opposed to maximum-response data to average-response predictions from ground motion relations; and v) demonstrated independence with respect to earthquake magnitude, source-to-site distance, and near-fault directivity parameters. The maximum direction was adopted as the ground motion intensity parameter for use in seismic design in lieu of explicit consideration of directional effects.

RP4-2.3 Replace ASCE/SEI 7-10 References of Chapter 21 with the following:

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RESOURCE PAPER 5
ONE-STORY, FLEXIBLE DIAPHRAGM BUILDINGS WITH STIFF VERTICAL ELEMENTS

RP5-1 INTRODUCTION

The seismic-force-resisting systems (SFRS) of many one-story buildings are constructed with non-filled steel deck or structural wood panel roof diaphragms that transmit inertial seismic force to stiff vertical elements, such as walls or braced frames. Occupancies for these buildings vary widely; common uses include retail, warehouse, athletics, institutional, and industrial. Many of these buildings are large in plan with long horizontal diaphragm spans between the vertical elements of the SFRS. The roof-heights of these buildings typically are 20 to 40 ft above the floor slab to create a large volume of enclosed space. Occasionally, these buildings may have even higher roofs for uses such as airplane hangars or athletic facilities.

In the United States (U.S.), the SFRS of these buildings are designed to meet the provisions of the *Minimum Design Loads for Buildings and Other Structures* (ASCE/SEI 7-10) [ASCE, 2010]. Other than how to treat the diaphragm stiffness for the purposes of distributing loads to vertical elements and the out-of-plane wall anchorage design, ASCE/SEI 7-10 considers the design for earthquake resistance of these buildings similar to that of all other buildings, including multi-story buildings with rigid floor diaphragms. The focus of ASCE/SEI 7-10 is on the design of the vertical elements of the SFRS, which are the elements that are expected to yield and dissipate energy when the building is subjected to severe earthquake ground shaking. The buildings being considered for this white paper have stiff vertical SFRS elements that are commonly constructed of reinforced concrete or masonry walls or steel-braced frames. Reinforced-concrete walls are usually precast but can also be cast in place. The precast walls can be cast on site and tilted up into place or cast in a plant, shipped to the site and lifted into place. The precast concrete walls are usually individual panels that range in width from about 12 ft to 30 ft.

The wood structural panel and metal deck diaphragms for these buildings are idealized as flexible in accordance with ASCE/SEI 7-10. The wood structural panels and the steel deck roofs resist in-plane diaphragm shears. Chords that resist tension and compression from flexural demands are provided at the perimeter of the diaphragm.

For wood diaphragms, the wood structural panels are attached to either wood framing or wood nailers that are fastened to open web steel joists. The flexibility of these wood diaphragms is primarily a function of the deformations at the nailed connections, shear deformation of the panel, and flexural deformation due to elongation and shortening of the diaphragm chords [AF&PA, 2009].

For metal deck diaphragms, the steel deck panels are attached to the steel roof framing, commonly composed of open web steel joists, with puddle welds, powder-actuated fasteners, or self-drilling screws. The steel deck sheets normally are 2 to 3 ft in width and are supplied in lengths that are easily shipped and handled. The sides of adjacent deck sheets are interconnected with button punches, top seam welds, self-drilling screws, or a variety of proprietary connectors and punches. These inter-panel connections are referred to as sidelaps. The flexibility of steel deck diaphragms is dependent on the shear and warping stiffness of the deck between connectors, the movement or slip of framing connections, the movement or slip at sidelap connectors, and the flexural deformations due to elongation and shortening of the diaphragm chords [SDI, 2004].

One-story buildings with stiff vertical elements and flexible roof diaphragms respond differently to earthquake ground shaking than typical multi-story buildings envisioned by codes respond. The response of these buildings is often dominated by diaphragm deformation and yielding rather than deformation and yielding of the vertical elements of the SFRS (Lawson, et al. 2014). The main objective of this study was to develop simplified seismic design procedures for one-story Rigid Wall-Flexible Diaphragm (RWFD) buildings that better reflect this type of response. For the purposes of this white paper, the RWFD buildings considered are one story with exterior walls of reinforced concrete or masonry construction though it also could include steel braced frames used for interior lines of resistance for large footprint buildings. The proposed design approach presented in Section 5 is only applicable to buildings with wood diaphragms, though it has the potential to be used for steel deck diaphragms as well. The adequacy of current design requirements for this type of building with wood or steel deck diaphragms was also explored. The proposed design approach is not intended to apply to standing seam metal roofs or similar metal roofs that are attached to framing members with clips that allow for relative movement between the metal roofing and the framing. It is also not intended to apply to steel deck attached to wood framing.

RP5-2 BEHAVIOR OF FLEXIBLE DIAPHRAGM BUILDINGS SUBJECTED TO EARTHQUAKE GROUND SHAKING

Performance of one-story RWFD buildings has often been poor when subjected to earthquake ground motion (SEAOC 2008). This poor performance has been primarily due to lack of wall anchorage or failure of inadequate wall anchorage to the diaphragm and partial collapse of the roof, and particularly collapse of perimeter bays of framing that were supported by exterior walls. Damage at re-entrant corners, at pilasters, and in the interior of diaphragms without continuity ties was also observed. In response to this poor performance, U.S. building code detailing requirements have become more restrictive and design forces for support of the top of walls for out-of-plane seismic loads have been increased in multiple editions of model building codes.

In the U.S. the reported failures of this style building due to earthquake ground shaking have been primarily of buildings with wood diaphragms (SEAONC (2001), EERI (1996), SSC (1995), APA (1994), SEAOSC/COLA (1994), EERI (1988), NOAA (1973), and USDOC (1967)). An example of a steel deck diaphragm with a partial roof collapse is the K-Mart building in Yucca Valley, California (Shipp (2010) and Brandow, 2010). There are fewer reports of damage to steel deck diaphragm RWFD buildings compared to wood diaphragm RWFD buildings; however, wood diaphragm RWFD buildings were substantially more numerous in locations where major earthquakes have occurred in the U.S., i.e., Northern and Southern California. In the last 20 years, steel deck diaphragms for this style building have become more common in these locations.

An argument could be made that the steel deck diaphragms have been tested in earthquakes because the roof level diaphragm of multi-story, steel-framed buildings is often an untopped steel deck. However, most multi-story buildings have shorter diaphragm spans, have roof diaphragms that are conservatively designed, do not have heavy exterior walls attached to the diaphragm, have gravity load columns and walls that cantilever above the more rigid top floor level to the roof which reduces the shear forces and deformation demands in the roof diaphragms, and may have been protected by yielding and energy dissipation of the vertical elements of the SFRS. Additionally, several proprietary connectors have been introduced in the last 20 years that have yet to be tested by strong earthquake ground motion. Also, the good performance of steel deck diaphragms of light metal buildings with steel siding should not be extrapolated to the one-story RWFD buildings with heavy walls because the relatively heavy walls create larger diaphragm shears and forces on diaphragm connectors that do not occur in the light metal buildings. Also, the strength of the light metal buildings is often based on wind loads resulting in inherent overstrength for earthquake design.

Responses of low-rise buildings with flexible roof diaphragms instrumented to record the accelerations and displacements during earthquakes have been obtained. Celebi et al. (1989) report the response of a gymnasium building in Saratoga, California with plan dimensions of 144 ft (north-south) by 112 ft (east-west) and a roof diaphragm of 3/8 in. plywood over tongue-and-groove straight sheathing. The response of this building to the 1984 Morgan Hill earthquake is reported. Peak accelerations of the ground-level slab were 0.10g in the north-south (longitudinal) direction and 0.04g in the east-west (transverse) direction. Accelerations at the center of the roof diaphragm, 0.42g and 0.20g in the longitudinal and transverse directions respectively, were about three times the accelerations at the edges of the roof diaphragm in each direction, 0.13g and 0.15g for the longitudinal direction and 0.06g and 0.07g for the transverse direction. Celebi, et al. report fundamental frequencies of 3.8 Hz and 3.9 Hz in the longitudinal and transverse directions respectively, for a fundamental period in each direction of about 0.26 sec. Damping is estimated to be about 5% to 6% of critical damping.

Bouwkamp, et al. (1994), report the response of three buildings to multiple California earthquakes. The buildings are the gymnasium in Saratoga that Celebi, et al. (1989) studied, a warehouse in Hollister, and a two-story building in Milpitas.

During the Loma Prieta earthquake, the gymnasium in Saratoga experienced peak accelerations at the ground-level slab of 0.35g and 0.24g in the transverse and longitudinal directions, respectively. From the ground-level slab to the diaphragm edge, accelerations were amplified by a factor of 1.27 and 1.46 for the transverse and longitudinal directions respectively. Accelerations were amplified from the diaphragm edge to the diaphragm center by factors of 1.95 and 2.07 for the transverse and longitudinal directions. Comparisons of analyses to measured responses indicated that the damping was approximately equal to 5% of critical damping in each direction.

The Hollister warehouse has plan dimensions of 300 ft by 100 ft and a plywood roof diaphragm. Accelerations are reported only in the transverse direction. The warehouse experienced peak accelerations of the ground-level slab of about 0.08g, 0.11g, and 0.25g during 1984 Morgan Hill, 1986 Hollister, and 1989 Loma Prieta earthquakes, respectively. Response of the warehouse indicates that very little amplification occurs between the ground-level slab and the edge of the roof diaphragm, 0.08g amplified to 0.09g, 0.11g amplified to 0.13g, and 0.25g amplified to 0.25g for the three earthquakes. This small amount of amplification indicates that walls act as rigid bodies for in-plane accelerations. From the edge of the diaphragm to the center of the diaphragm, accelerations are amplified from 0.09g to 0.25g, 0.13g to 0.29g, and 0.25g to 0.79g for the three earthquakes. The acceleration amplifications indicate that the response of this building was dominated by the dynamic properties of the roof diaphragm. The effective fundamental period for this building in the transverse direction was 0.56 sec in the 1984 Morgan Hill earthquake, 0.58 sec in the 1986 Hollister earthquake, and 0.81 sec in the Loma Prieta earthquake. The longer period during the Loma Prieta earthquake is consistent with the higher accelerations and increased levels of yielding relative to those that occurred during the other two earthquakes. Analyses indicated that the damping was 2% of critical damping.

The building in Milpitas is 120 ft by 168 ft in plan and the roof diaphragm is plywood nailed to wood framing. Accelerations were measured during the 1988 Alum Rock earthquake and the 1989 Loma Prieta Earthquake. During the Loma Prieta earthquake, the two-story Milpitas building experienced peak accelerations at the ground level of 0.138g and 0.090g in the longitudinal and transverse directions, respectively. In the longitudinal direction, the acceleration at the ground level is amplified by a factor of 4.2 to an acceleration of 0.58g at the center of roof diaphragm. Acceleration at the edge of roof diaphragm is not provided for the longitudinal direction. In the transverse direction, accelerations were amplified by a factor of 1.26 to 0.11g between the ground-level slab and the edge of the roof diaphragm and amplified by a factor of 2.75 to 0.31g between the edge of the roof diaphragm and the center of the diaphragm. The larger acceleration amplification factor between the ground-level slab and the center of the diaphragm for the longitudinal versus transverse direction, 4.2 versus 3.46 (1.26x2.75) is consistent with the greater diaphragm overstrength leading to higher accelerations in the longitudinal versus transverse direction. Damping was found to be less than 5% of critical damping.

The damping is close to 5% of critical damping for two buildings and about 2% for one of the buildings. The large variation may be due in part to different roofing systems. The roofs with higher damping may have consisted of composition roofing, which would lead to higher damping than for a roof with a single-ply membrane. For new buildings with single-ply roofing membranes and minimal architectural finishes, damping is likely to be closer to 2% of critical.

Hamburger, et al. (1997) studied the response of three tilt up buildings (A, B, and C) that experienced the 1994 Northridge earthquake. The three buildings were located near one another in Chatsworth, California. They were of similar size but had different out-of-plane wall connection details. Building A was constructed in 1969 and retrofitted in 1993 with improved out-of-plane wall support. Building B was constructed in 1970 and Building C was constructed in 1977. Buildings A and B have plan dimensions of 180 ft north-south and 207 ft east-west. Building C has plan dimensions of 185 ft north-south and 200 ft east-west. In the Northridge Earthquake, Building A performed well, Buildings B and C each had roof/wall separation in the north-south loading direction and collapse in the east-west direction.

A non-linear model of each diaphragm was developed and response history analyses were performed. The fundamental period varied between 0.36 and 0.56 sec. The analysis results indicate that the out-of-plane wall anchorage forces are limited by yielding of the diaphragm and that the ground accelerations were amplified on the order of 2.5 times at the center of the roof diaphragm. This was less than 3.0 to 3.5 times amplification for the linear-elastic model. The investigators conclude that for conservatively designed diaphragms or for long, narrow diaphragms loaded in the longitudinal direction, the out-of-plane wall anchorage force may easily exceed forces generated from accelerations equal to more than three times the ground acceleration. The investigators recommended that ductile wall anchorage be provided because no analysis, not even a non-linear response-history analysis can accurately predict these forces to guarantee that they will not be exceeded.

RP5-3 CURRENT SEISMIC DESIGN APPROACH

The current seismic design approach for one-story buildings with flexible roof diaphragms is to base the building’s seismic force level, including that of the diaphragm, on parameters defined by the vertical elements of the seismic-force-resisting system (SFRS) (Table 12.2-1, ASCE/SEI 7-10). The wall’s out-of-plane force level and anchorage force level are separately derived and account for amplified ground accelerations as identified by Bouwkamp, et al. (1994) and Celebi, et al. (1989), observed failures, and consideration of whether yielding should be allowed. While some out-of-plane yielding of the wall is considered acceptable, top of wall anchorage forces are expected to be resisted by connections and members that behave elastically.

RP5-3.1 Seismic Systems and Response Modification Factors

For commonly used systems in these buildings, the applicable response modification coefficients, R, are listed in Table 1 as given in ASCE/SEI 7-10. In buildings designed for high and moderate levels of earthquake shaking, intermediate precast shear walls with an R of 4 are commonly used for buildings constructed with site-cast tilt-up walls and plant-cast precast walls. Buildings with masonry walls designed for high levels of earthquake shaking utilize special reinforced masonry shear walls, and those designed for moderate levels of earthquake shaking utilize intermediate or ordinary reinforced masonry shear walls. The lateral strengths required for vertical elements of buildings designed for low levels of earthquake shaking are often governed by wind forces. Regardless of whether the buildings are designed for high, moderate, or low seismic force levels, the diaphragm is designed for the same force level as the vertical elements regardless of whether the expected yielding is in the vertical elements or the horizontal diaphragm.

Table 1 Commonly used R values for One-story Flexible Diaphragm Buildings

Seismic-Force-Resisting System	Response Modification Coefficient, R	Limitations
Special reinforced concrete shear walls	5	160 ft roof height for SDCs D to F
Intermediate precast shear wall	4	40 ft or 45 ft roof height for one-story warehouse structures in SDCs D to F
Ordinary precast shear walls	3	Not permitted in SDCs C to F
Special reinforced masonry shear walls	5	160 ft roof height limit for SDCs D to F
Intermediate reinforced masonry shear walls	3-1/2	Not permitted in SDCs D to F
Ordinary reinforced masonry shear walls	2	160 ft roof height limit for SDC C; not permitted for SDCs D to F
Steel buckling-restrained braced frames	8	160 ft limit for SDCs D and E and 100 ft limit for SDC F
Steel special concentrically braced frames	6	160 ft limit for SDCs D and E and 100 ft limit for SDC F
Steel ordinary concentrically braced frames	3-1/4	60 ft roof structure height limit for one-story buildings with a roof dead load is less than 20 psf in SDCs D and E; not permitted in SDC F
Braced frames designed as steel system not specifically detailed for seismic resistance	3	Not permitted in SDCs D to F

RP5-3.2 Seismic Design Forces for the Diaphragm and Diaphragm Strength

In determining the seismic design forces for these buildings the approximate period, T_a, computed with Eq. 3-1, is used to estimate the fundamental period of the building.

$$T_a = C_t h_n^x \qquad \text{(Eq.3-1)}$$

Where h_n is the structural height in feet,

C_t is a coefficient equal to 0.02 for these buildings, and

x is a coefficient equal to 0.75 for these buildings.

For most practical cases, the period computed with Eq. 3-1 is in the constant acceleration portion of the design spectrum. In common practice the diaphragm flexibility is not accounted for to obtain a longer period and lower design forces for new building design and there is no specific allowance to do so in code design; however, for evaluation and strengthening of existing buildings, ASCE/SEI 41-06 (ASCE, 2006) has a special provision to allow consideration of diaphragm flexibility that is sometimes used. The force level is equal to the seismic response coefficient, C_s, times the effective seismic weight, W, where C_s is computed in accordance with Eq. 3-2.

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \qquad \text{(Eq. 3-2)}$$

Where S_{DS} is the design spectral acceleration parameter in the short period range,

R is the response modification factor based on the vertical elements, and

I_e is the importance factor.

Structural wood panel and steel deck diaphragms are designed for the in-plane shear forces, and the chords around the perimeter of the diaphragm are designed for the in-plane flexure. The same force level C_sW, which is based on the vertical SFRS element's period, ductility, and overstrength, is used to determine these in-plane shear and flexural forces acting on the horizontal diaphragm. The effective seismic weight includes a portion of the weight of the walls oriented perpendicular to the direction of the seismic force, the weight of the roof, vertical parapet, and equipment on the roof, and if applicable, a portion of the snow load. Chord forces are almost always determined by dividing the flexural moments by the distance between chords, i.e., the diaphragm depth. Collector elements, if present, are designed for an amplified force equal to the design force times the system over-strength factor, Ω_o, if designing for moderate or high levels of earthquake shaking, i.e., Seismic Design Categories (SDCs) C to F.

The diaphragm shear strengths are determined in accordance with industry documents. Structural wood panel diaphragms are designed in accordance with tables in AF&*PA Special Design Provisions for Wind and Seismic, 2008 Edition* (AF&PA SDPWS) [AF&PA, 2009]. Tables for combinations of panels, framing, and nailing are provided to determine the diaphragm shear strength based on monotonic test data. Steel deck diaphragms are designed in accordance with the Steel Deck Institute's *Diaphragm Design Manual* (SDI-DDM) [SDI, 2004] or more commonly they are based on evaluation services testing reports such as those produced by the International Code Council's Evaluation Services (ICC-ES). These reports are usually based on monotonic test data of diaphragms tested in accordance with ICC-ES AC43 [ICC-ES, 2010].

Seismic demands for each direction are computed and compared to the diaphragm shear strength provided. Although diaphragms resist shear in both directions with the same panels and connectors, ASCE/SEI 7-10 does not require the shear demands of the two orthogonal directions to be combined in the design of diaphragms.

RP5-3.3 Out-of-Plane Wall and Wall Anchorage Forces

During an earthquake, the ground shakes and moves the base of walls in-plane and out-of-plane, which generates inertial forces in the wall. The wall movement causes the diaphragm to move, which generates inertial forces in the diaphragm. The diaphragm movement causes movement of the tops of the walls, which also leads to inertial forces generated in the walls. These out-of-plane wall anchorage forces can be much larger as determined by the percentage of effective mass than the overall forces for the SFRS.

ASCE/SEI 7-10 requires that walls be designed for an out-of-plane force, F_p, in accordance with the following formula.

$$F_p = 0.4 S_{DS} I_e W_p \geq 0.1 W_p \qquad \text{(Eq. 3-3)}$$

Where W_p is the weight of the wall.

ASCE/SEI 7-10 requires that the anchorage of walls and transfer of forces into the diaphragm be designed for an out-of-plane wall force in accordance with the following formulas.

$$F_p = 0.4 S_{DS} k_a I_e W_p \geq 0.2 k_a I_e W_p \qquad \text{(Eq. 3-4)}$$

$$k_a = 1.0 + \frac{L_f}{100} \leq 2.0 \qquad \text{(Eq. 3-5)}$$

Where k_a is an amplification factor for diaphragm flexibility, and

L_f is the span, in feet, of a flexible diaphragm between resisting walls or rigid frames.

In Eqs. 3.4 and 3.5, K_a and L_f account for diaphragm flexibility. If a diaphragm is rigid, L_f equals 0 but if it is flexible, L_f equals the span length. For many buildings with rigid walls and flexible roof diaphragms, the diaphragm span between supporting walls or frames will be longer than 100 ft so k_a will often be equal to 2.0. For diaphragms in which K_a equals 2.0, the acceleration parameter used to compute the out-of-plane wall anchorage force is $0.8S_{DS}$, which is 80% of the maximum design spectral acceleration parameter. The intent is that the wall anchorage force is resisted elastically for a force level computed using the maximum design spectral acceleration. The 0.8 factor is included to recognize that some connection and member overstrength may be relied upon to resist the top of wall anchorage force.

Eq. 3-4 allows for top of wall anchorage forces less than those computed using the maximum design response spectrum parameter for cases in which the diaphragm span is less than 100 ft. This reduction is inconsistent with the common condition in which a short span diaphragm often has significant overstrength that can result in greater forces developing at the top of wall support.

RP5-3.4 Stiffness of Wall Anchorage Connections

The stiffness of wall-to-diaphragm connections is also considered in design. In previous earthquakes flexible wall-to-diaphragm connections have led to premature failures and loss of wall support. Strap anchors have buckled and fractured prematurely. Also, as the straps elongate the ledger was then loaded in cross grain bending. A similar issue occurs with hold downs that have oversized bolt holes. The slip that occurs as the hold down moves to engage the bolt can also allow the diaphragm shear connectors to load the ledger board in cross grain bending. The cross grain bending and the impact of the loose connections are more likely to lead to loss of wall support. Designs should include wall-to-diaphragm connections that have adequate stiffness to avoid cross grain bending of ledger boards, buckling and subsequent fracture of straps, and other detrimental effects of connections that are too flexible.

Current practice for connecting diaphragms to walls avoids some of the potential problems. For example, steel angles have largely replaced wood ledgers. This eliminates the possibility of cross grain bending. Hybrid roof systems have significantly reduced the number of diaphragms with wood framing, thus eliminating the use of hold downs. For steel deck diaphragms, the welded connections are stiff. Although other issues may exist with the newer connections, such issues may not become evident until the next major earthquake occurs.

RP5-3.5 Transfer of Wall Anchorage Forces to Continuous Diaphragm Ties

Although the interaction of the diaphragm and walls is dynamic in nature, following the flow of forces as if only static forces were involved makes it easier to understand how the connections and roof elements should be designed for support of the walls out-of-plane. For wood diaphragms, the static simplification of the load path for support of out-of-plane wall forces has wall anchorage forces being transferred to sub-diaphragms in the end regions of the diaphragm that transfer anchorage forces into continuity ties that distribute forces into the main diaphragm. Current practice is for the sub-diaphragm to be designed for the top of wall anchorage forces but the shears generated within the sub-diaphragm are not combined with the shears from the main diaphragm even-though they occur simultaneously. For steel deck diaphragms, some designers use sub-diaphragms similar to the approach used for wood diaphragms. More commonly for steel deck diaphragms, the static simplification of the load path for support of out-of-plane wall forces has wall anchorage forces transferred to the end of the deck sheets for out-of-plane forces parallel to the direction that the deck sheets span and into the joists for out-of-plane forces perpendicular to the direction that the joists span.

For buildings with concrete and masonry walls, Section 12.11.2.2 of ASCE/SEI 7-10 requires continuous diaphragm ties that resist the out-of-plane top of wall connection forces. Many engineers interpret the provision to require that the force at the edge of the wall be transferred across the entire diaphragm through the continuity ties. Other engineers interpret the requirement as allowing the force in the continuity tie to be reduced as the force is transferred into the main diaphragm. This approach is very common when analyzing existing buildings. If using this approach of variable tie force, the required tie strength is small near the center of the diaphragm assuming equal wall mass at each diaphragm edge acting with motions in phase. To address the small tie force, engineers often use a minimum tie force equal to that required by Section 12.1.3 (SEAOC 2008 and SEAOC 2010).

Section 12.1.3 ASCE/SEI 7-10 requires that all parts of buildings between seismic separation joints be interconnected. If the minimum force level determined using this section is applied, the ties must also be designed for a seismic force, F_p, equal to 0.133 times short period acceleration, S_{DS}, times the weight of the smaller portion of the building being connected to the larger portion but not less than 5% of the weight of the smaller portion. For one-story buildings, the weight includes the wall and roof weights. Therefore, the maximum tie force will be located where the weight to each side of the tie is equal. For symmetrical buildings, this will be the center of the diaphragm.

RP5-3.6 Deformation Compatibility

ASCE/SEI 7-10, Section 12.11.1, requires that "interconnection of structural wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces." Section 12.12.2 requires that "the deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements." Despite these requirements, diaphragm deflections are rarely computed for this building type, so these deformation compatibility requirements are rarely checked.

RP5-3.7 Diaphragm Design for Strength

The current design approach does not require a hierarchy of shear strength between the diaphragm and walls. Because the walls are stronger than the diaphragm and diaphragm connections to the wall for most practical design cases, the connections for the diaphragms yield prior to yielding of the vertical elements of the SFRS. Thus, the performance of this type of building is very dependent on the overstrength and ductility of the diaphragm and its connectors.

RP5-3.7.1 Wood Diaphragms

Wood structural panels are commonly nailed to wood framing or to nailers that are fastened to the top of steel joists or beams. At the perimeter of the diaphragm, the wood panels are nailed to wood ledgers and sometimes fastened directly to steel ledgers using pneumatic actuated pin fasteners. Less commonly used fasteners are staples for attaching structural wood panels to wood framing or powder actuated fasteners for attaching structural wood panels directly to the steel framing members.

Wood diaphragms are designed in accordance with *Special Design Provisions for Wind and Seismic* (SDPWS-2008) (AF&PA, 2009). Wood diaphragms included in SDPWS consist of wood structural panels such as oriented-strand board or plywood nailed to wood members or wood nailers. It does not address staples used as fasteners. Wood diaphragms are described as unblocked if some panel edges are unsupported by framing members or blocked if all panel edges are supported. Unblocked diaphragms are used in low and moderate seismic regions and are generally not used for RWFD buildings. Blocked structural wood panel diaphragms are commonly used for RWFD buildings in some western states. The aspect ratio (diaphragm span length to depth) for blocked diaphragms must be less than 4:1. For short and moderate diaphragm spans, a single line of nails is used along panel edges. The fastener spacing along panel edges vary from a maximum spacing of 6 in. to a minimum spacing of 3 in. except at diaphragm boundaries and continuous panel edges parallel to loading for which the spacing is a minimum of 2 in. on center. Panel edges nailed at 3 in. on center and greater may be supported by nominal 2 in. wide lumber. For nail spacing less than 3 in. on center and at all diaphragm boundaries, the lumber must be nominally at least 3 in. wide to mitigate splitting. The spacing of nails to intermediate supports is 12 in. on center maximum. The nominal strength of blocked diaphragms varies from a minimum of 370 pounds/linear foot (plf) to a maximum of 1,640 plf. The nominal strength must be reduced by a strength reduction factor of 0.80 for load and resistance factored design (LRFD) or by 1/2 to obtain a factor of safety of 2 for allowable stress design (ASD).

For longer diaphragm spans in RWFD buildings designed for high-seismic risk, high-load diaphragm nailing may be used to obtain shear strengths more than the maximum for a typical blocked diaphragm. The high-load diaphragm nailing patterns were developed as part of a testing program in the 1970s in which the goal was to determine the diaphragm nailing required to develop the strength of the wood panels (SEAOC, 2008); however, the high-load nailing patterns in the table do not develop the strength of the panels. The high-load nail patterns consist of two and three lines of nails along panel edges. Nail spacing along the lines at typical staggered panel edges varies from a maximum of 6 in. to a minimum of 3 in. on center with nail locations for adjacent lines being staggered. At diaphragm boundaries and continuous panel edges, nail spacing varies from a maximum spacing of 4 in. to a minimum spacing of

2-1/2 in. on center. Framing members are required to be nominally 3 or 4 in. wide depending on the number of lines and spacing of the nails. The nominal strengths vary from a minimum of 1,210 plf to a maximum of 3,130 plf. A strength reduction factor of 0.80 applies to design strength for LRFD and a factor of safety of 2 applies to ASD.

The nailing pattern is usually specified in zones that account for increasing shear demand from the center of the diaphragm to the edges. In areas of high seismic demands such as California, as many as six nailing zones are used for a larger diaphragm.

RP5-3.7.2 Steel Deck Diaphragms

For steel deck diaphragms, deck sheets are attached to open web steel joists or steel beams using arc-spot welds, powder-actuated fasteners, or self-drilling screws. Deck sheets are laid over multiple steel framing members, usually to cover three or more spans, and bottom surfaces of the deck flutes are attached to the framing members. In building specifications the pattern in which the deck is to be attached to framing members is described as the deck width followed by the number of fasteners within the width, i.e., 36/7 indicates the deck sheet is 36 in. wide and seven connectors attach the deck to each framing member. Various patterns are shown in Figure 1. Where the end of one sheet meets the end of the next sheet, the sheets are typically overlapped and the deck-to-framing connections attach both sheets to the underlying framing.

The edges of deck sheets can be either nested or interlocked, as shown in Figure 1. Nested deck is common in the Eastern U.S. but is also used in the Western U.S. Interlocking deck is primarily used in the Western U.S. For nested deck, not only do the ends of sheets overlap but so do adjacent edges of deck; therefore, the corner deck-to-framing connectors pass through four sheets and then attach to the underlying framing. For nested deck, the corner panel connector is shared by all four deck sheets meeting at that corner, and the edge deck-to-framing connectors are shared by the two overlapping sheets. For interlocking deck, the corner connectors are only shared by the two sheets that overlap one another at ends of deck sheets, and the edge connectors are not shared by the adjacent sheets. Therefore, for the same deck-to-framing connection pattern, interlocking deck averages one extra connector for each deck sheet width. The extra connector results in a less than proportionate increase in diaphragm shear strength due to the details of the connections as described below. For a nested deck, shear force at a connector is transferred directly from one sheet to the adjacent connected sheet through the edge connector. For the interlocking deck, shear force is transferred from the edge connectors into the steel frame and from the steel framing to the adjacent deck sheet at the additional connector. The difference in strength is only the difference in strength of transferring shear directly from one sheet to the next versus the shear strength of a connector transferring from the deck to the framing.

The sides of adjacent deck sheets are interconnected with button punches, top seam welds, self-drilling screws, or a variety of proprietary connectors and punches. These inter-panel connections along the sheet edges are referred to as sidelaps. Button punching requires an interlocking deck, so it is used in the Western U.S. Top seam welds can be used on either interlocking deck or nested deck though in the case of the nested deck, the sidelap connection is more of a fillet weld along the edge of one of the sheets. Screws need to be installed at 90 degrees to the deck surface, so they are mostly installed in nested deck but can also be installed horizontally into specially formed interlocking deck. The proprietary sidelap connectors and punches are mostly used in the Western U.S. In the Western U.S., sidelap connector spacing is specified, while in the Eastern U.S., the number of sidelap connectors for each deck span is specified.

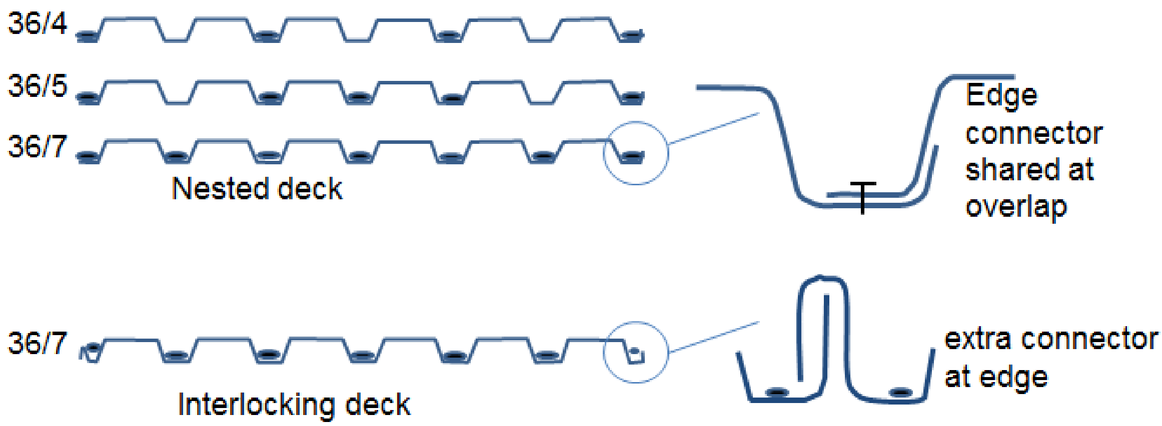

FIGURE 1 Fastener Configurations for Nested and Interlocking Deck

Steel deck diaphragms are designed using deck suppliers' evaluation services reports (ESRs) or using the design approach in the Steel Deck Institute's Diaphragm Design Manual (SDI-DDM) (SDI 2004). The diaphragm shear design values in ESRs are based on monotonic testing of relatively small diaphragms that rarely contain end lap connections. Shear strength values for 20 and 22 gage deck with spans in the range of 5 to 7 ft are generally in the range of 200 plf to 1,600 plf for allowable stress design, which includes a factor of safety of 2.5 for mechanical fasteners and 3.0 for welds for design for earthquakes. For load and resistance factor design (LRFD), strengths are generally in the range of 300 plf to 2,600 plf, which includes a strength reduction factor of 0.65 for mechanical connectors and 0.55 for welds.

Zoning of connectors for steel deck diaphragms is common in regions where seismic forces are high but the number of zones used is often limited to three. In regions of low and moderate seismic demand, diaphragm connector zoning is also used but less commonly than in regions of higher seismicity.

RP5-4 EVALUATION OF CURRENT DESIGN APPROACH

Although the current design approach is based on the assumption of inelastic response in the vertical elements (walls, rigid frames), yielding of the diaphragm commonly dominates the inelastic response of the buildings to earthquakes. Archetype buildings using the current design approach were designed, and P695 analyses (FEMA P695, 2009) were applied to these designs to provide a baseline of performance and to demonstrate that changes to the design approach are necessary. The FEMA P695 methodology is used to reliably quantify building system performance and provide guidance in the selection of appropriate design criteria when ASCE/SEI 7-10 linear design methods are applied. The primary objectives of FEMA P695 are to obtain an acceptably low probability of collapse of the SFRS under maximum considered earthquake (MCE) ground motions, and to provide uniform protection against collapse across various structural systems. An appropriate P695 evaluation must use a representative nonlinear model that includes both detailed design information of the system as well as comprehensive test data on the post-yield performance of system components and subassemblies.

A proposed structural system, or in this case a proposed design methodology of an existing system, is evaluated through the use of collapse fragility curves. Incremental Dynamic Analyses (IDA) on a representative sample of nonlinear numerical building models that account for the range of the design space are conducted to build the collapse fragility curves using a pre-determined ensemble of earthquake ground motions. The number of archetypes selected is based on building an appropriate representation of the typical RWFD structure including the range of variation reasonably expected and likely to affect performance. The archetypes are assigned to performance groups for the evaluation process. The collapse margin ratio (CMR) is determined from the IDA and fragility curves. The CMR is defined as the median spectral collapse intensity at the fundamental elastic period of the building archetype under analysis, $S_{CT}[T]$ obtained from nonlinear dynamic analyses divided by the ground motion spectral demand $S_{MT}[T]$ at the maximum considered earthquake (MCE) intensity level at the same fundamental elastic period. The collapse margin ratio is multiplied by a spectral shape factor (SSF) to obtain an adjusted collapse margin ratio (ACMR). The spectral shape factor is a function of the fundamental period, the period-based ductility, μ_T, and the applicable seismic design category of the archetype under analysis. The ACMR for both individual archetypes and the archetype performance groups are compared to acceptable ACMRs that accounts for uncertainties judged to be within the evaluation process. The acceptable ACMR for a performance group targets less than 10% probability of collapse in an MCE event, and the acceptable ACMR for individual archetypes targets less than 20% probability of collapse in an MCE event.

RP5-4.1 DESCRIPTION OF ARCHETYPES

Evaluations were performed on archetypes designed for high and moderate seismic risk. For the evaluations, high-seismic risk is defined as having design acceleration parameters S_{DS} equal to 1.0 and S_{D1} equal to 0.60. These acceleration parameters are used for the SDC D_{max} evaluation in accordance with FEMA P695. Moderate-seismic risk is defined as having design acceleration parameters S_{DS} equal to 0.499 and S_{D1} equal to 0.199, which are the boundary values for SDCs C and D and is referred to as SDC C_{max} for the evaluations.

For these evaluations, archetypes were designed with either steel deck or wood panel diaphragms. The archetype naming convention is shown in Figure 2. Summary tables and tables with detailed descriptions of diaphragm zones for the archetypes are in Appendix A. The walls for these archetypes are reinforced-concrete wall panels 25 ft in width, 33 ft tall measured from the top of the slab-on-grade, and 9-1/4 in. thick for high seismic risk and 7-1/4 in. thick for moderate seismic risk. The roof level is 30 ft above the top of the slab-on-grade with the walls cantilevering as a parapet 3 ft higher than the roof level.

The archetypes are grouped by diaphragm type, either wood or steel deck, and whether the diaphragm is relatively large or small. The archetypes have plan aspect ratios of 1:1, 2:1 and 1:2. Large diaphragms are 400 ft long and either 400 ft or 200 ft wide. The small diaphragms with aspect ratios of 2:1 and 1:2 are 200 ft long by 100 ft wide, and for the 1:1 aspect ratios the diaphragms are 100 ft by 100 ft. The wood diaphragms are wood structural panels nailed to wood nailers that are attached to open web steel joists. The steel deck diaphragms are attached to the joists with arc spot welds, powder actuated fasteners, or self-drilling screws. Adjacent steel deck sheets are attached along their sides (sidelaps) with top seam welds, button punches, or self-drilling screws. Many steel deck diaphragms include proprietary sidelap connections in the western portion of the U.S., which includes regions of high seismicity. Archetypes are not included with the proprietary sidelap connections because cyclic test data for the response of these connectors is not available. The steel deck diaphragms were designed using 22 and 20 gage deck because connection data was available for connectors with these deck gages. Note that thicker gage deck results in less ductile connections so the steel deck diaphragm results presented herein are limited to only diaphragms with 22 or 20 gage deck.

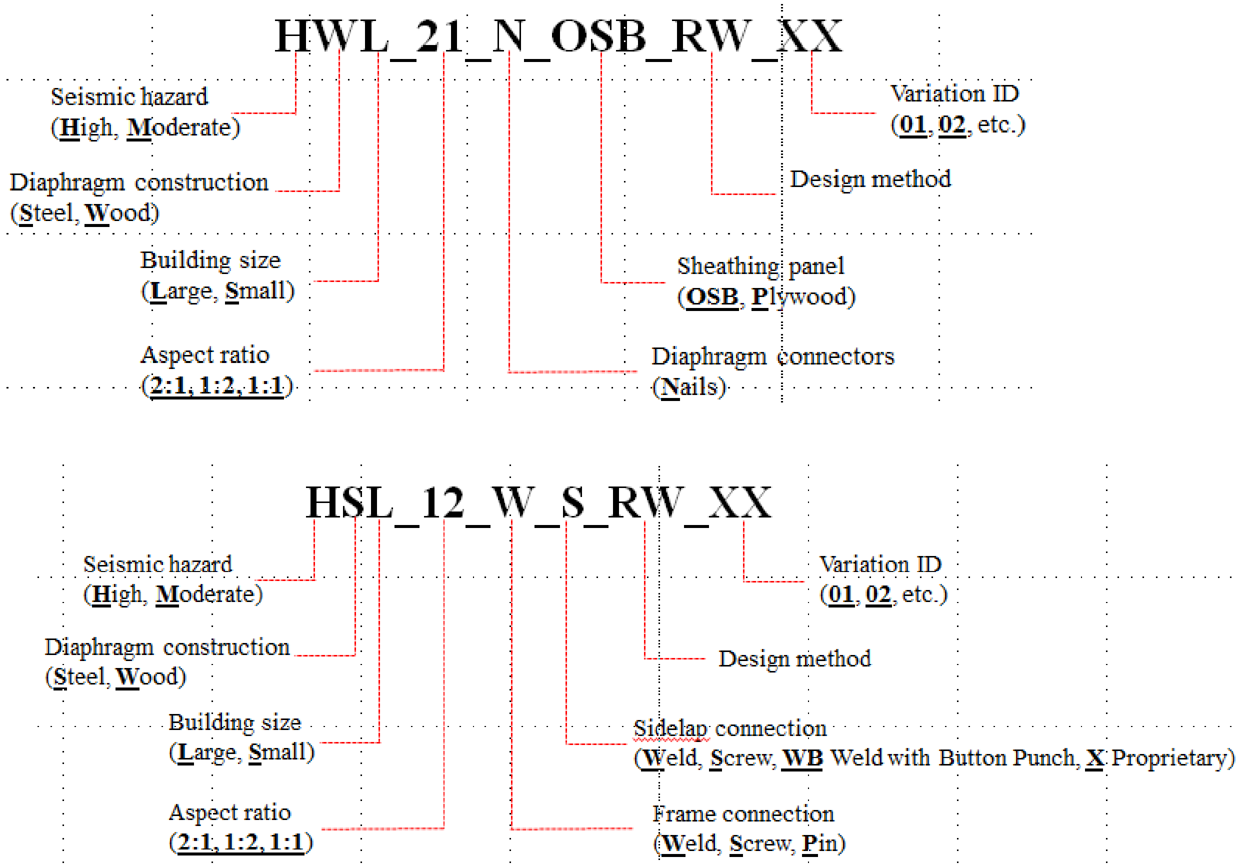

FIGURE 2 Archetype Naming Convention

The archetypes were designed as intermediate precast shear walls with a response modification factor of 4. The period for each direction of each archetype was computed using the approximate period equation, Eq. 3-1, for T_a. For the 30 ft roof height of the archetypes, the approximate period is 0.26 sec. This value is less than the transition period, T_s, equal to 0.60 sec for high-seismic risk (SDC D) archetypes and 0.40 sec for moderate seismic-risk (SDC C_{max}) archetypes. Therefore, the archetypes for the existing design approach, i.e., current code, are designed using a base shear coefficient of 0.25 for the high-seismic risk (SDC D) archetypes and for a base shear coefficient of 0.125 for the moderate seismic risk (SDC C_{max}) cases.

The diaphragms for the archetypes are zoned with different fastener spacings similar to zoning commonly used in practice. For the wood diaphragms, the diaphragms are zoned as follows:

- Large wood diaphragms with 1:1 aspect ratio – four zones of concentric squares for high-seismic risk and two zones of concentric squares for moderate seismic risk.
- Large wood diaphragm with 2:1 and 1:2 aspect ratio for high-seismic risk – six zones total with the first two zones near the center of diaphragm as concentric rectangles and the remaining zones banded for loading in the transverse direction
- Large wood diaphragm with 2:1 and 1:2 aspect ratio for moderate seismic risk – three banded zones for loading in the transverse direction
- Small wood diaphragm with 1:1 aspect ratio – two zones of concentric squares
- Small wood diaphragms with 2:1 and 1:2 aspect ratio – five banded zones for high-seismic risk and two banded zones for moderate seismic risk for loading in the transverse direction

For the steel deck diaphragms, the diaphragms are zoned as follows:

- Large steel deck diaphragm with 1:1 aspect ratio – three zones of concentric squares
- Large steel deck diaphragm with 2:1 and 1:2 aspect ratio – three zones banded for transverse direction loading
- Small steel deck diaphragms with 1:1 aspect ratio – one zone used for entire diaphragm
- Small steel deck diaphragms with 2:1 and 1:2 aspect ratio – two zones banded for transverse direction loading

RP5-4.2 Modeling Framework

The computational time necessary to perform nonlinear response history analyses, i.e., the time it takes for the computer to run the analyses, is too long to allow for analyses of detailed finite element models of each archetype, so an approach was developed that allows for a relatively simplified model to be used. The approach includes three steps and is illustrated in Figure 3:

1. Create a data base of connectors that have been cyclically tested and based on the test results determine the hysteretic parameters of Wayne Stewart (Stewart 1987) or SAWS (Folz and Filiatrault, 2001) nonlinear cyclic springs for each connector and shown in Figure 4. Examples for nailed and welded connectors are illustrated in Figures 5 and 6.
2. Create a mathematical model in MATLAB of one-half of each archetype's diaphragm that accounts for the stiffness of the panels, diaphragm chords, and each diaphragm connector. Divide the diaphragm model into horizontal segments and load the diaphragm by a cyclic point load at its mid-span. Combine the response of the elements and connectors within each diaphragm segment to form a single nonlinear hysteretic spring that represents the cyclic response of the segment. This step is illustrated in Figure 7. Variable connector spacing is accounted for in this model.
3. Create a model in RUAUMOKO2D (Carr, 2007) that consists of one-half of the archetype and includes a spring and mass for each horizontal diaphragm segment, beam elements with nonlinear hinges to represent the out-of-plane wall response, and springs and masses to represent the in-plane wall at the end of the diaphragm. A representation of the simplified model is shown in Figure 8. P-delta effects are incorporated into the model by applying the roof weight on leaning columns between the ground and the roof diaphragm. Localized effects that account for shear concentration in sub-diaphragms or regions receiving wall anchorage forces are not accounted for in this analysis.

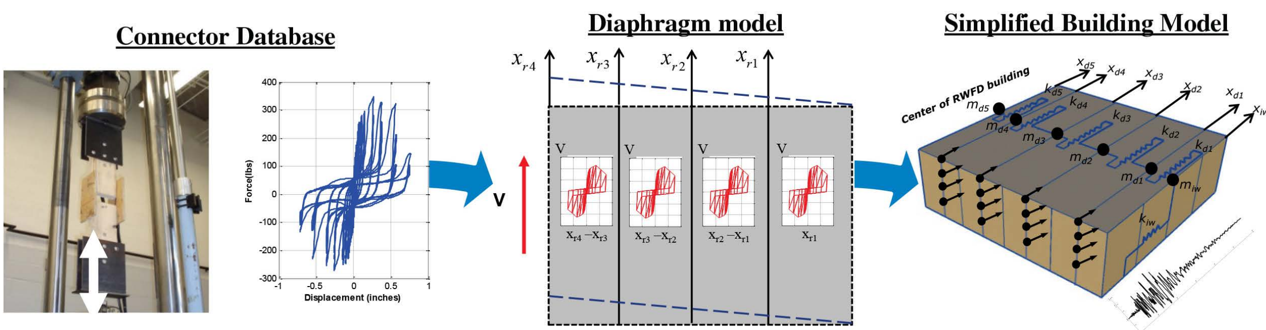

FIGURE 3 Three Step Modeling Approach

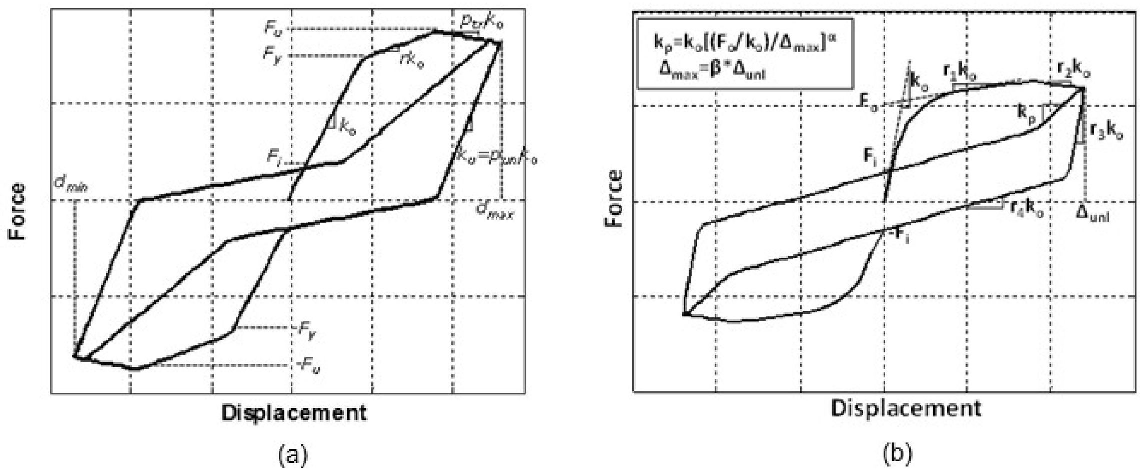

FIGURE 4: Illustration of: (a) Wayne-Stewart Hysteretic Model (after Stewart, 1987) and CUREE-SAWS Hysteretic Model (after Folz and Filiatrault, 2001)

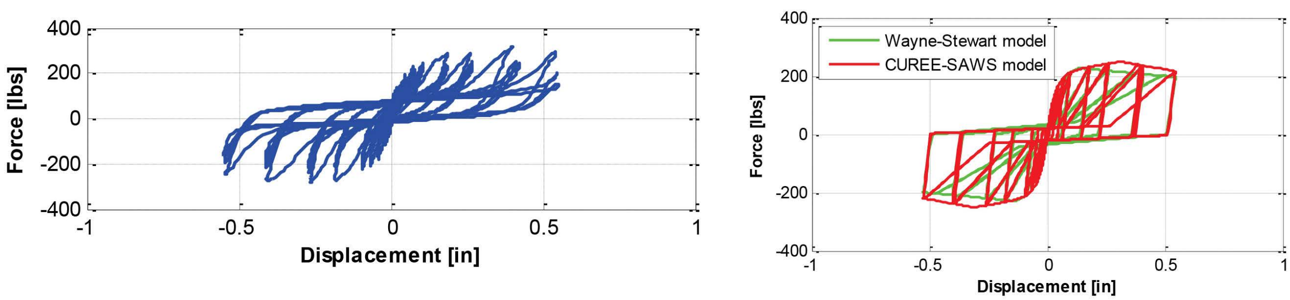

FIGURE 5: Comparison of Hysteretic Response for 10d Common Nail and Wood Deck: (a) Example of Experimental Data (Coyne, 2007) and (b) Best Fit Numerical Model Based on Data from Several Tests (Koliou, 2014)

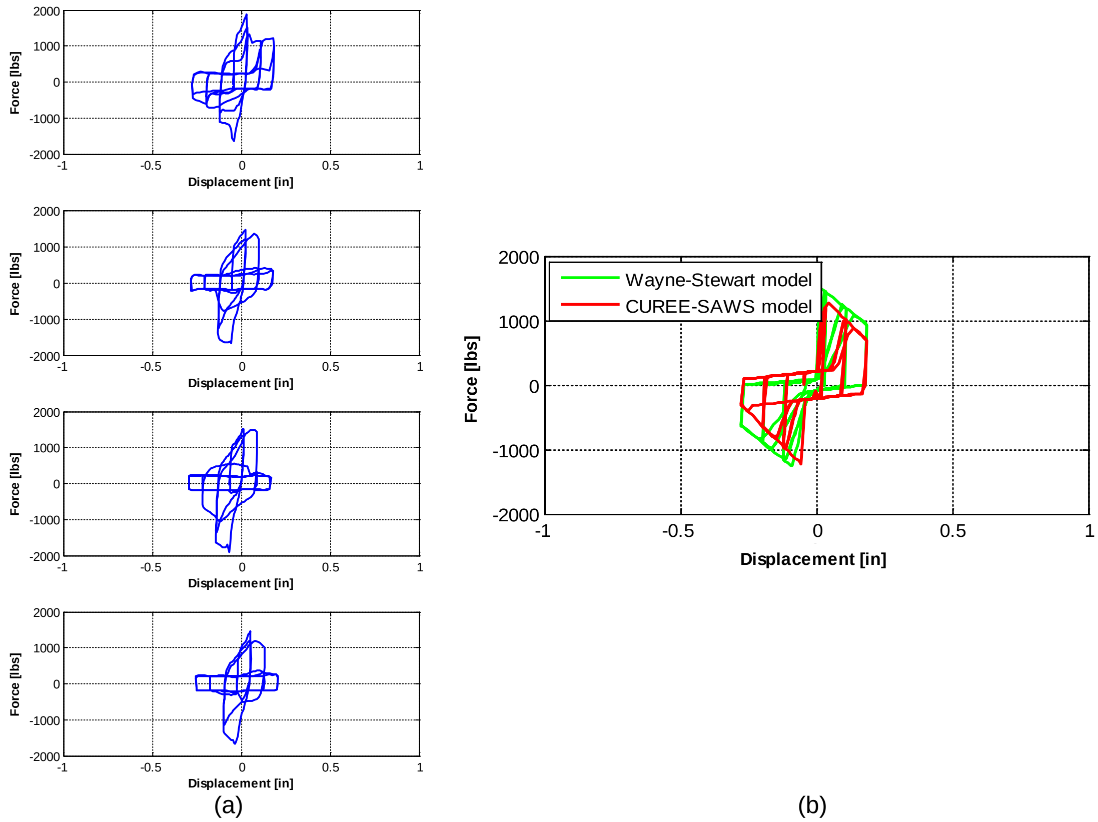

FIGURE 6 Comparison of hysteretic response for 2-ply-22-ga framing welds for to 0.25 in. plate: (a) experimental (Guenfoud, et. al., 2010) and (b) fitted optimal hysteretic models

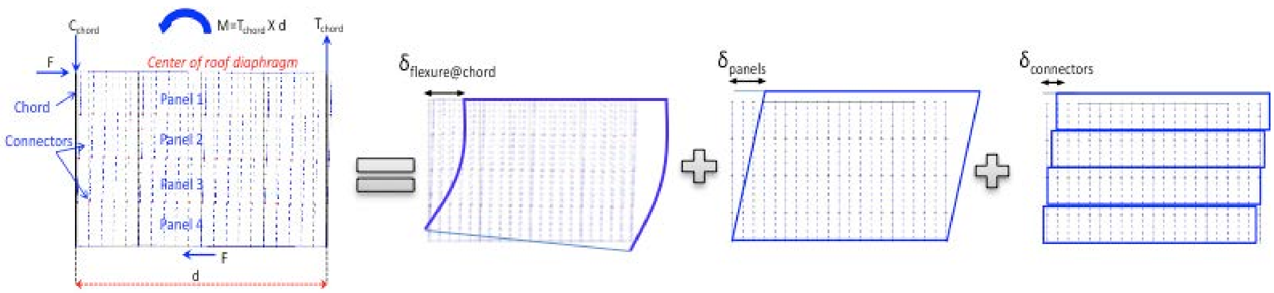

FIGURE 7 In-plane displacement components of the analytical inelastic roof diaphragm model

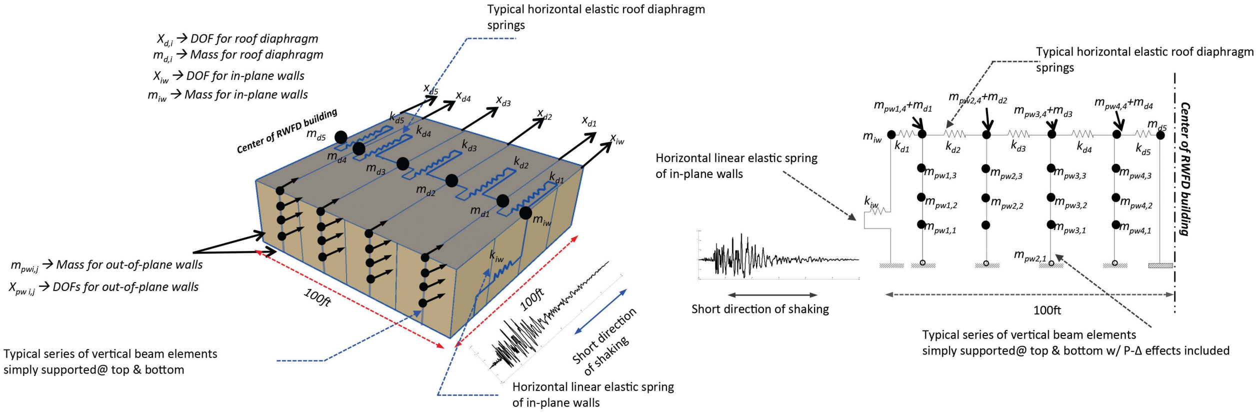

FIGURE 8 Illustration of a simplified rigid wall flexible diaphragm (RWFD) building model developed in the RUAUMOKO2D platform

Cyclic test data for the 10d common nails are from Coyne (2007), Fonseca and Campbell (2002), and Christovasilis, et al (2009). Cyclic data for button punch, screw, and top seam weld sidelap connectors between deck sheets are from Rogers and Tremblay (2003b). Cyclic data for welds, powder actuated fasteners, and screws that connect steel deck to steel plates are from Guenfoud, et al. (2010) and from Rogers and Tremblay (2003a). A list of the database of cyclic test data for nails and steel deck connectors is included in Tables B1, B2, and B3 of Appendix B.

RP5-4.3 Modeling Framework Validation

The two-dimensional model of the diaphragm, Step 2, was validated by comparing predicted results to cyclic diaphragm test results of Tremblay et al. (2004). The predicted and tested shear for monotonic loading are normalized to the applicable maximum shear and compared in Figure 9. The results for shear versus deformation for one main cycle predicted by the diaphragm sub model versus the test results are compared in Figure 10. Results for the simplified model, Step 3, are compared to those of a detailed finite element model by Olund (2009). The periods are obtained from eigenvalue analyses using the simplified building model and are initial elastic periods. The natural periods of the simplified model for the first and second modes are about 3% longer than those predicted by Olund's detailed model.

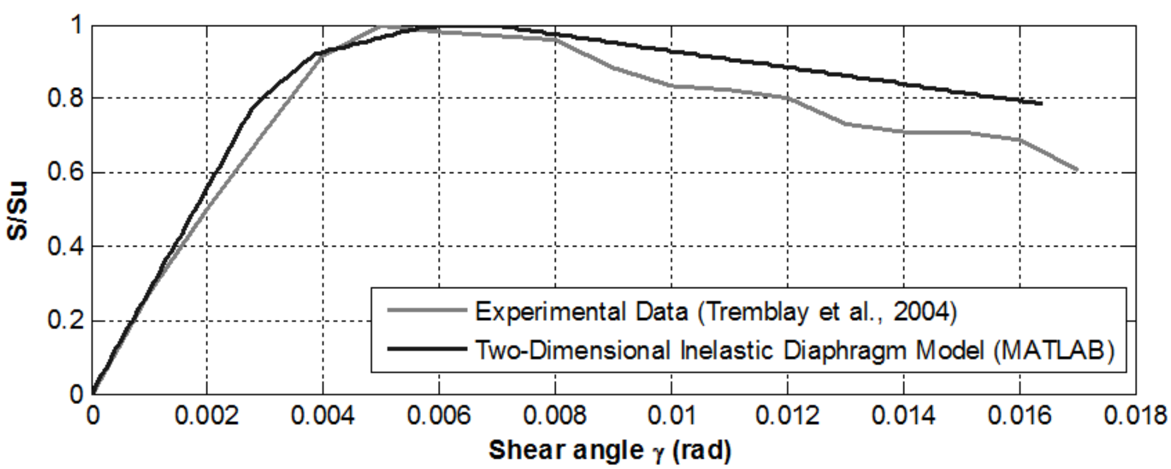

FIGURE 9 Validation/comparison of inelastic roof diaphragm for monotonic loading

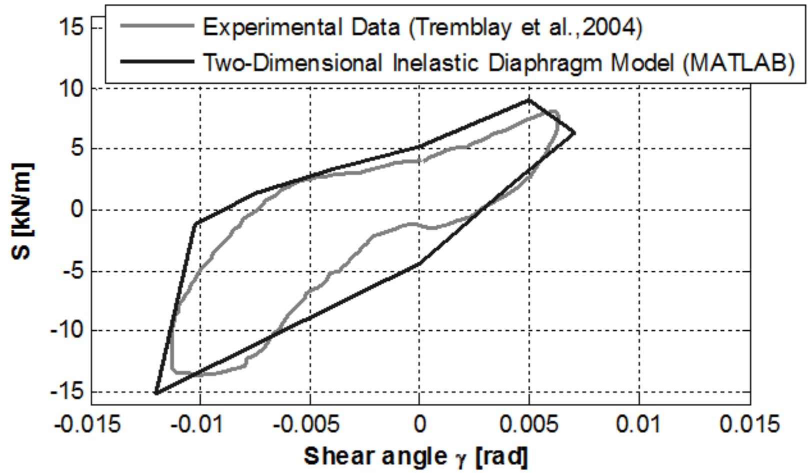

FIGURE 10 Validation/comparison of inelastic roof diaphragm for cyclic loading (one main cycle)

The collapse assessment of the rigid wall flexible diaphragm (RWFD) building was evaluated by conducting Incremental Dynamic Analyses (IDA) (Vamvatsikos and Cornell, 2002) using the FEMA P695 Far Field Ground Motion Ensemble (FEMA P695, 2009) and computing the median collapse intensity. To monitor the state of the structure at the end of each nonlinear time history dynamic analysis, the diaphragm (roof) drift ratio was considered as the damage measure (DM), while the intensity measure (IM) was defined by the spectral acceleration ($S_a(T_1)$) at the fundamental period of the RWFD building; T_1 is the fundamental period obtained from eigenvalue analyses of the initially elastic simplified building model. Olund (2009) set the limit state for a RWFD as a roof drift equal to 3%. For the purpose of comparing results of Olund's detailed model to those of the simplified building model the limit state is set equal to 3% roof drift, but for analysis results reported later in this report, the limit state is equated to a load-displacement (P-delta) sidesway instability. As shown in Figure 11, the simplified building model slightly underestimates (by approximately 9%) the median collapse intensity compared to the detailed FEM developed by Olund (2009).

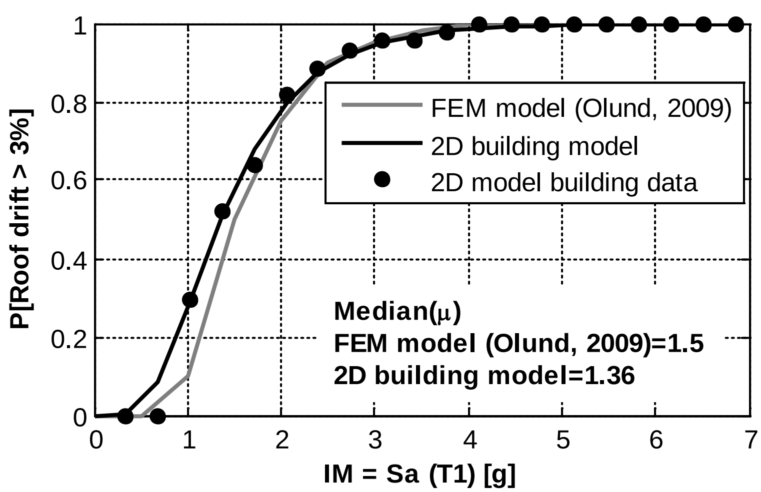

FIGURE 11 Incremental dynamic analysis results/comparison

A limitation of the modeling framework is that it cannot account for the simultaneous loading of framing connectors at ends of steel deck sheets from the in-plane diaphragm shears and the continuity tie or out-of-plane wall anchorage forces. Therefore, the archetypes designed and analyzed are based on the requirements to resist shear only. However, the magnitudes of these two sources of loading are not compatible with one another. Specifically, out-of-plane wall forces are not reduced to account for ductility and the connections supporting the wall are expected to behave elastically, while the forces acting on these same connectors from diaphragm shear resistance are based on an assumption that the connections will yield. There seems to be a philosophical conflict in the intended behavior of the fastener when exposed to both diaphragm shear forces and wall anchorage forces, which occur simultaneously in some regions of the diaphragm. For steel deck diaphragms, consideration should be given to supporting the out-of-plane wall forces with separate continuity ties rather than transferring them into the deck sheets.

RP5-4.4 FEMA P695 Analysis Results for Current Design

Incremental Dynamic Analyses (IDA) (Vamvatsikos and Cornell, 2002) using the FEMA P695 Far Field Ground Motion Ensemble (FEMA P695, 2009) were performed using the simplified building models to compute the median collapse intensity. For the analysis results, the median collapse intensity is defined as the median spectral intensity at the fundamental period that causes side-sway P-delta instability. Summaries of collapse results for individual wood and steel deck diaphragm archetypes are included in Appendix C. Values for the median collapse intensity, $S_{CT}[T]$, ground motion spectral demand, $S_{MT}[T]$, building drift ratios (BDR) for the design level earthquake and maximum considered earthquake, the spectral shape factor (SSF), fundamental elastic period, and period based ductility, μ_T, are provided. The BDR, which equals the sum of the wall drift ratio (WDR) and the diaphragm drift ratio (DDR), is used as measure of building damage for this study. Collapse margin ratios (CMRs), adjusted collapse margin ratios (ACMRs), and acceptable ACMRs for the individual archetypes and the archetype groups are also included in Appendix C. The acceptable ACMRs are computed using the uncertainty (beta) factors listed in Table 2. Note that the value for the design requirement uncertainty is based on an assumed improved design procedure so a larger value might be more appropriate for evaluating the current design approach. The combined uncertainty, β_{Total}, is computed as the square root sum of the squares of the individual uncertainties. For β_{Total} equal to 0.66, the acceptable ACMR is 1.73 and 2.30 for 20% and 10% probability of collapse at Maximum Considered Earthquake (MCE) ground motion, respectively. The acceptable ACMR applicable for the performance of individual archetypes is 1.73, and the acceptable ACMR of performance groups is 2.30. Summaries of the collapse margin ratio results for the performance groups of wood and steel archetypes are provided in Tables 3 and 4.

The current design approach did not pass the P695 collapse criteria for the small archetype performance groups with wood diaphragms. Each small wood diaphragm archetype had an individual ACMR that exceeded the acceptable value of 1.73 indicating each archetype passed the criterion. However, the small archetype performance groups for both high- and moderate-seismic risk had ACMR values of 2.11 and 1.98, respectively, which is less than the acceptable ACMR of 2.30, so these performance groups did not pass the criterion. The small archetypes had short periods and remained in the short period range as yielding occurred. In this region, period shifts can result in resonance with large spikes in the short period range that can lead to much larger spectral accelerations and displacements than predicted using the design spectrum plateau. This issue of short-period structures having higher seismic demands leading to the lowest level of collapse performance has been well documented, starting with Newmark and Hall (1973). Section 9.5.1 of FEMA P695 (2009) describes this issue with short-period structures in greater detail.

The larger archetype wood diaphragm performance groups had ACMRs of 2.68 and 2.80 for the high- and moderate risk performance groups, which exceed the acceptable ACMR of 2.30 to pass the criterion. Each large wood diaphragm archetype had an individual ACMR that exceeded the acceptable value of 1.73. The large wood archetypes performed better because the elastic periods were often off of the design response spectrum's plateau but the diaphragm was designed as though it was on the plateau of the spectrum. Also, as the diaphragm yields its period lengthens, which in general leads to lower forces.

Both the large and small steel deck diaphragm performance groups did not pass the criteria. For individual archetypes with high-seismic risk 5 of 7 large archetypes and 1 of 9 small archetypes had ACMRs less than the acceptable ACMR of 1.73, and for individual archetypes with moderate-seismic risk 3 of 12 large archetypes and 3 of 9 small archetypes had ACMRs less than acceptable. The ACMRs for the large archetype groups of 1.71 and 1.87 for high-seismic risk and 1.95 and 1.88 for moderate seismic risk performed slightly worse than the small archetype groups. The small archetype groups had values of 2.23 and 1.92 for high-seismic risk and a value of 1.86 for moderate seismic risk. The reason for the difference relative to what was observed for the wood diaphragms is that the larger steel diaphragms were relatively stiffer than the larger wood diaphragms and had periods that were on or close to the design spectrum plateau. Differences in ACMRs of the archetypes with various combinations of welds, PAFs, screws and button punches were not significant enough to distinguish definitive differences in performance of the connectors.

Table 2 Beta factors for determining acceptable collapse margin ratios

Description	Beta factor	Value
Record-to-record uncertainty	β_{TR}	0.40
Design requirements uncertainty	B_{DR}	0.20
Test data uncertainty	β_{TD}	0.35
Modeling uncertainty	B_{MDL}	0.35
Combined uncertainty	β_{Total}	0.66

Table 3 Summary of collapse margin ratio results for wood diaphragm archetype performance groups for current design

Performance Group	Seismicity	Size	Computed ACMR	Acceptable ACMR	Pass/Fail
PG-1E	High	Large	2.68	2.30	Pass
PG-2E	High	Small	2.11		Fail
PG-3E	Moderate	Large	2.80		Pass
PG-4E	Moderate	Small	1.98		Fail

Table 4 Summary of collapse margin ratio results for steel deck diaphragm archetype performance groups

Performance Group	Seismicity	Size	Framing Connectors	Sidelaps	Computed ACMR	Acceptable ACMR	Pass/Fail
PG-5E	High	Large	Welds	Welds/button punches	1.71	2.30	Fail
PG-6E	High	Large	PAFs/screws	Screws	1.87		Fail
PG-7E	High	Small	Welds	Button punches	2.23		Fail
PG-8E	High	Small	PAFs/screws	Screws	1.92		Fail
PG-9E	Moderate	Large	Welds/PAFs/screws	Screws	1.95		Fail
PG-10E	Moderate	Large	Welds/PAFs/screws	Welds	1.88		Fail
PG-11E	Moderate	Small	Welds/PAFs/screws	Screws	1.86		Fail

The results indicate that the current design rules are not adequate based on the FEMA P695 methodology for both wood structural panel and steel deck diaphragms. Therefore, a modified or new design approach is needed. The results also indicate that if a single response modification factor were to be used for the diaphragm parallel to current design of SFRS, its value would need to be less than 4. For the wood diaphragms, considering that the lowest ratio of ACMR to acceptable ACMR (1.98/2.3) is 0.86 leads to the expectation that current design using an R-factor (0.86x4) of about 3 to 3.5 is appropriate. For steel deck diaphragms, consider that the lowest ratio of ACMR to acceptable ACMR (1.71/2.3) is 0.74 leads to the expectation that current design using an R-factor of about 2.5 to 3 may be appropriate for

20 and 22 ga steel deck diaphragms. However, analyses should be performed to determine the appropriate R values. This expectation is not appropriate for steel deck diaphragms with gages of 18 or thicker as they are anticipated to result in less ductile diaphragms.

The analysis results also showed that the diaphragm's inelastic behavior was concentrated at the ends of the diaphragm. The global ductility of the diaphragm is lower than the ductility of the connectors because connector yielding is not spread well into the diaphragm. Response of these buildings would improve if a means of spreading connection yielding deeper into the diaphragm were developed. For the steel deck diaphragms, the response would also be improved if the post-yield stiffness of the connector were positive to a larger displacement value. Tests indicate positive post-yield stiffness only to 1 or 2 mm. The exact displacement needed for improved behavior is not known at this time but a value such as 10 mm is expected to be more than adequate.

RP5-4.5 Fundamental Period of the Archetypes

The results of the simplified building model analyses show that the fundamental period of each archetype is dominated by the diaphragm response. The elastic fundamental periods for the archetypes were obtained from the analyses and are included in Tables C1 and C3 of Appendix C. A summary of the elastic periods are included in Table 5.

Table 5 Summary of Elastic Periods for Archetypes

Wood diaphragm span length	Period for high-seismic archetypes	Period for moderate-seismic archetypes
400 ft	0.85 to 0.87 sec	0.90 to 0.92 sec
200 ft	0.49 to 0.54 sec	0.55 to 0.58 sec
100 ft	0.36 to 0.38 sec	0.43 to 0.45 sec
Steel diaphragm span length		
400 ft	0.49 to 0.56 sec	0.61 to 0.73 sec
200 ft	0.35 to 0.42 sec	0.51 to 0.59 sec
100 ft	0.21 to 0.26 sec	0.28 to 0.33 sec

Using the elastic periods from the analyses of the high-seismic wood diaphragm archetypes and the high-seismic steel deck diaphragm archetypes, empirical formulas were developed for the fundamental periods of wood and steel deck diaphragms, respectively. The following formula is proposed for computing the fundamental period, T_{wood}, of the wood diaphragm buildings with concrete or masonry shear walls.

$$T_{wood} = \frac{0.0019}{\sqrt{C_w}} h_n + 0.002L \qquad \text{(Eq. 4-1)}$$

Where, L is the diaphragm span in feet between vertical elements of the seismic-force-resisting system,

C_w is a coefficient defined by Equation 12.8-10 in ASCE/SEI 7-10

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]} \qquad \text{(Eq. 4-2)}$$

A_B is the area of the base of the building in ft^2,

A_i is the web area of the in-plane shear wall i in ft^2,

D_i is the length of the in-plane shear wall i in ft^2, and

h_i is the height of the in-plane shear wall i in ft^2.

h_n is the height of the roof framing in ft.

The following formula is proposed for computing the fundamental period, T_{steel}, of the steel deck diaphragm buildings.

$$T_{steel} = \frac{0.0019}{\sqrt{C_w}} h_n + 0.001L \qquad \text{(Eq. 4-3)}$$

The proposed period equations are plotted along with the periods determined from the simplified building model analyses in Figure 12 for the wood diaphragm archetypes and in Figure 13 for the steel diaphragm archetypes. Although the periods are derived from analyses of single-span diaphragms, the deformations were shear dominated. Therefore, the period equations are valid for multi-span diaphragms. The archetypes used to develop these formulas included heavy walls.

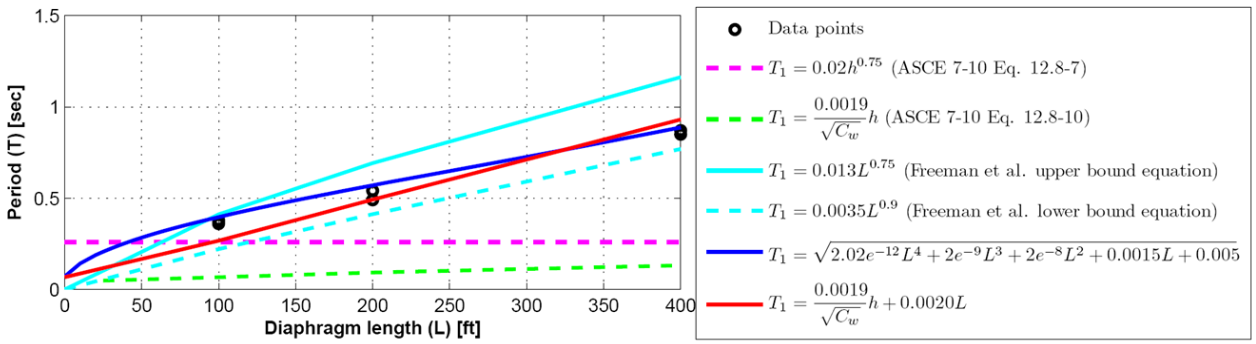

FIGURE 12 Comparison of fundamental periods from analyses of wood panel archetypes to those predicted by the proposed formula (Eq. 4-1), ASCE/SEI 7-10 equations, a best fit curve, and those proposed by Freeman et. al (1995)

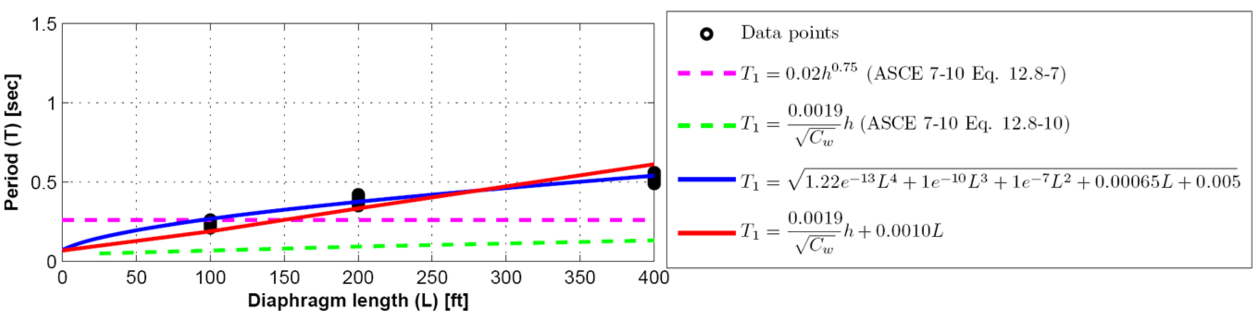

FIGURE 13 Comparison of fundamental periods from analyses of steel deck archetypes to those predicted by the proposed formula (Eq. 4-3), ASCE/SEI 7-10 equations, and a best fit curve

RP5-4.6 Out-of-plane Wall Anchorage Forces

A study was performed to determine the magnitude of the force required to anchor the top of walls for out-of-plane forces. Archetype HSL_21_W_WB_RW4 was analyzed to determine the out-of-plane wall forces. This archetype is 400 ft long by 200 ft wide with the roof height at 30 ft. The wall panels are 9-1/4 in. thick and 33 ft tall. The roof diaphragm is steel deck with welded framing connectors, top seam weld sidelap connectors in the outer two zones and button punches at the center zone. The top of wall anchorage force computed in accordance with Eq. 3-4, i.e., ASCE/SEI 7-10, is 1,680 lbs/ft, which is based on the design level earthquake. The model of the archetype was loaded with the forty-four ground-motion records from FEMA P695 and amplified to spectral acceleration intensities up to 2.5g. The median out-of-plane wall anchorage forces for the forty-four records at the roof level are shown in

Figures 14 and 15 respectively for the transverse and longitudinal direction of loading. In the figures, the blue lines represent the model that accounts for out-of-plane wall flexibility and the red line represents the behavior if the wall is treated infinitely rigid out-of-plane. The MCE intensities are where the dashed vertical blue lines cross the curves.

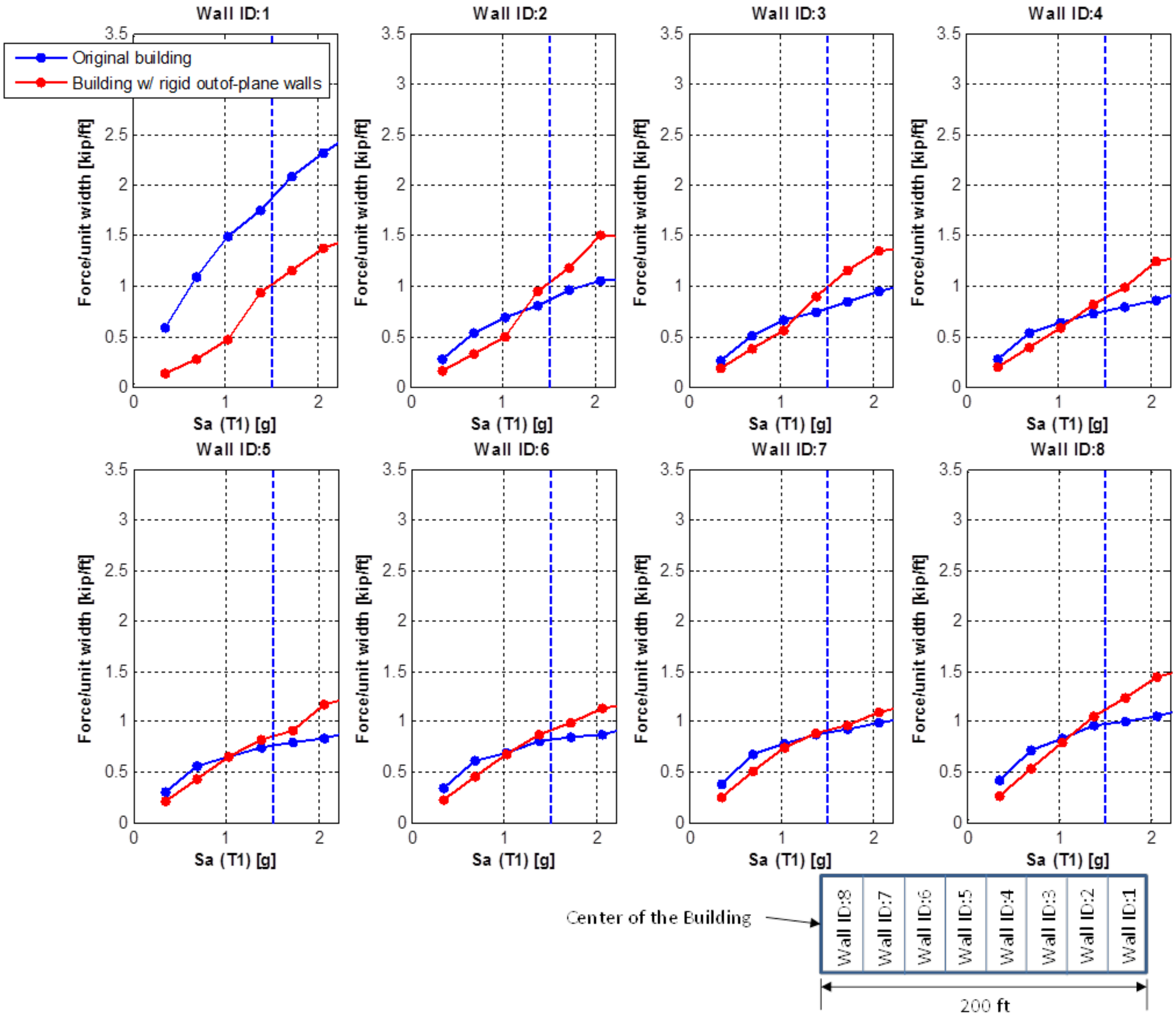

FIGURE 14 Median out-of-plane wall anchorage force for transverse direction of loading

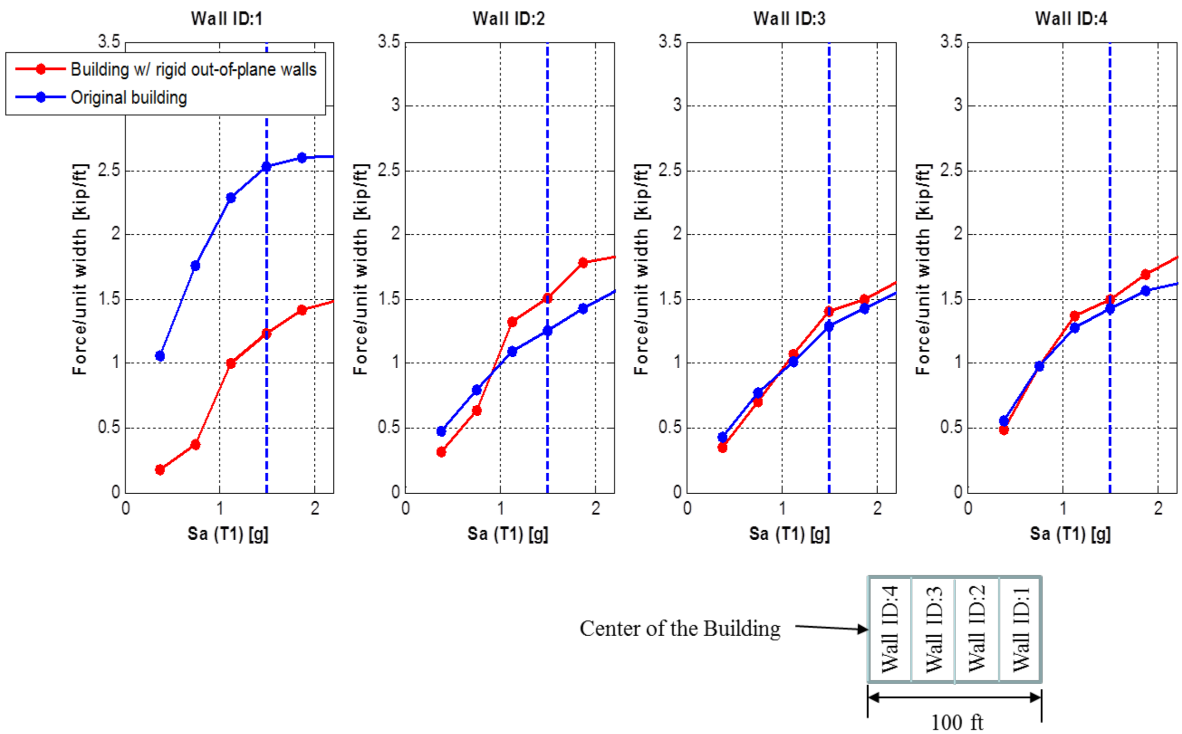

FIGURE 15 Median out-of-plane wall anchorage force for the longitudinal direction of loading

The study showed that forces are higher for walls loaded out-of-plane when the diaphragm is loaded in the longitudinal direction than they are for walls loaded out-of-plane when the diaphragm is loaded in the transverse direction. At the center of the diaphragm span, the out-of-plane wall anchorage forces are about 1.2 kips/ft for the transverse direction loading and 1.5 kips/ft in the longitudinal direction loading for MCE level of shaking. Out-of-plane anchorage forces in the longitudinal direction are likely greater than in the transverse direction because the diaphragm has more overstrength in the longitudinal versus the transverse direction. The study also showed that anchorage forces were higher at the corners than in the middle of the diaphragm. This occurs because input motions between the bottom and the top of the wall panels are more likely to be in-phase adjacent to the corners, which dynamically amplifies the wall accelerations near the corner. Also, the panel flexibility affects the load developed near the corners more than away from the corners. Forces generated in the analyses are less than the required design forces computed in accordance with ASCE/SEI 7-10 using Eq. 3-4 except at the corner in the longitudinal direction. The results suggest that the formula for out-of-plane wall anchorage should be revised to eliminate the k_a variable and change the 0.4 coefficient to 0.8.

RP5-5 PROPOSED DESIGN APPROACH

An approach to design rigid wall-flexible diaphragm buildings for earthquake effects is proposed. The approach consists of two design paths: (1) one in which yielding is expected in the vertical elements, and (2) a second in which diaphragm yielding is expected. The design path for which diaphragm yielding is expected includes proportionally strengthened end regions, which has the potential to spread yielding deeper into the diaphragm as described in Section 5.4. The proposed approach is based on the results of a study in which the performance of an archetype was improved by weakening the diaphragm away from the end region Koliou, et al. (2014). For the purposes of this white paper ductile diaphragms are wood structural panels with common nails for connectors. At this time, the proposed design approach is meant to apply to RWFD buildings with wood diaphragms, though portions of the approach could be applied to the design of RWFD buildings with steel deck diaphragms as well. The proposed design approach is not intended to apply to standing seam metal roofs or similar metal roofs that are attached to framing members with clips that allow for relative movement between the metal roofing and the framing. It is also not intended to apply to steel deck attached to wood framing.

RP5-5.1 Description of Proposed Design Approach

The proposed design approach allows for two design paths based on whether

- vertical elements are to yield and dissipate energy or
- the diaphragm is to yield and dissipate energy.

These paths are described in the Sections 5.1.1 and 5.1.2. Regardless of which design path is chosen certain design requirements apply. These requirements are included in Section 5.1.3.

RP5-5.1.1 Vertical Element Yielding Design Path

If the vertical elements are to yield and dissipate energy, the diaphragm design force level should be based on the force level for the vertical elements but amplified by the overstrength factor, Ω_o, for the SFRS. The overstrength factor, Ω_o, is 2-1/2 for the wall systems listed in Table 1. If the system is an intermediate precast shear wall with R equal to 4 and Ω_o equal to 2-1/2, the effective R_{dia} for the diaphragm design is 1.6. Accounting for the diaphragm overstrength and designing the diaphragm for an effective diaphragm response modification factor, R_{dia}, equal to 1.6 results in limited ductility demand for design earthquake ground motions. Designing for yielding of the vertical elements applies to RWFD buildings with regular or irregular plan geometries. In using such a low effective diaphragm response modification factor, consideration could be given to using a greater magnitude strength reduction factor on the diaphragm design strength, but this would require a study to determine the appropriate value to use. Until such a study is performed, the current strength reduction factor, ϕ, of 0.8 for wood diaphragms should be used. Note that all limitations and requirements for the vertical system in ASCE/SEI 7-10 apply when following this design path.

Diaphragm chords and collectors should be designed using the forces generated using the effective diaphragm response modification factor, R_{dia}. An additional diaphragm overstrength factor does not need to be applied to the design of the collectors.

Designing a RWFD building that will actually yield the vertical elements is difficult because the vertical elements usually have significant overstrength, particularly concrete wall systems. Obtaining yielding and energy dissipation in a wall system rather than the roof diaphragm generally requires designing and detailing wall-to-foundation or wall-to-slab connections that yield as wall panels and their connections are loaded by overturning moments.

If a RWFD building has more than one type of vertical element, for example tilt-up walls on the perimeter with special concentrically steel braced frames at an interior expansion joint, the diaphragm design should be based on the system with the lowest response modification factor, R, and that systems overstrength factor.

RP5-5.1.2 Diaphragm Yielding Design Path

This design path is only appropriate for RWFD buildings with regular shaped wood diaphragms at this time. If the diaphragm is regularly-shaped in plan such that it can be divided into rectangular plan areas supported on each edge, the following design procedure can be used in which diaphragm yielding and energy dissipation is expected. The following requirements apply.

1. The vertical elements of the SFRS, i.e., shear walls and frames, should be designed for in-plane forces using the response modification factor, R, for the type of system being engineered but not

greater than 4. The detailing requirements and system limitations still apply even though the R factor may have been reduced.

2. Diaphragm design shears are determined in the transverse direction using a response modification factor of the diaphragm, Rdia, equal to 4.5, an approximate period from Eq. 4-1 of this report, and the formulas for Cs in Section 12.8.1 of ASCE/SEI 7-10. For buildings with steel-braced frames for the vertical elements instead of walls, the first term in Eq. 4-1 may be replaced by 0.02hn0.75 where hn is the height of the roof framing in ft.
3. The diaphragm design shears computed in Item 2 for the transverse direction are increased by a factor of 1.5 for the end 10% of length regions of the diaphragm span. This is shown in Figure 16. In addition to designing the end regions for 1.5 times the force, the contract documents should specify diaphragm design strength in the end regions at least 1.5 times the design strength specified for the remainder of the diaphragm.
4. Diaphragm design shears for the longitudinal direction shall be computed using a response modification factor of the diaphragm equal to 4.5 with the shears increased by a factor of 1.5 in the end 10% of diaphragm width regions.
5. Diaphragm chords shall be designed for the diaphragm flexure resulting from diaphragm loads equal to those computed using Rdia.
6. Collectors and their connections are designed for the forces computed from the design diaphragm shears being collected and amplified by a diaphragm overstrength factor, Ωodia. An applicable value for the diaphragm overstrength factor is currently being developed. In the meantime a value of 2.5 is appropriate.

RP5-5.1.3 Design Requirements Applicable to All RWFD Buildings

The following requirements apply to diaphragm design of both design paths.

1. Out-of-plane wall forces and out-of-plane top of wall anchorage forces shall be designed in accordance with Section 12.11 of ASCE/SEI 7-10 except that ka shall be set equal to 2 for all diaphragm spans. Wood ledgers may not be loaded in cross-grain bending as part of the wall anchorage connection and wood structural panels may not resist tensile or compressive wall anchorage loads.
2. Connections and members that support out-of-plane anchorage forces at the tops of walls must be capable of resisting the design force while undergoing expected building deformations including those from diaphragm deflections. The roof displacement may cause rotation at these connections that opens and closes the angle between the roof and the wall, as shown in Figure 17. For connections similar to that shown in Figure 17, prying force develops due to force eccentricities and due to distortion. For the vertical leg of the ledger angle shown in the figure, a force eccentricity exists between the tension in the horizontal leg of the angle and the connectors that attach the angle to the wall. The top of the vertical leg of the angle is being separated from the wall face and this causes the bottom of the angle to bear against the wall surface. The result is additional prying forces at connectors attaching the vertical angle leg to the wall. Prying can also occur between the structural wood panel and the horizontal leg of the angle due to the distortion between the wall and roof diaphragm. As distortion reduces the geometric angle between the plane of the roof deck and the plane of the wall, the structural wood panel will bear against the end of the horizontal leg of the ledger angle. This will be resisted by tension in the connectors between the panel or deck and the horizontal leg of the angle. As distortion increases this geometric angle, the edge of the panel or deck will bear against the top of the horizontal leg of the ledger angle, which will also cause tension at the connectors between the panel or deck and the horizontal angle leg. A study is needed to determine how to compute the total diaphragm displacement. In the meantime, the elastic diaphragm deflection should be computed using the SDPWS-2008 approach at the design force level and then this displacement should be multiplied by the effective diaphragm response modification factor, R_{dia}, to obtain the total

diaphragm displacement. For example, if the wall yielding path is chosen using R equal to 4 and Ω_o equal to 2.5, the elastic diaphragm displacement should be multiplied by 1.6. The elastic displacement of the wall should be multiplied by the C_d factor to obtain the total wall deflection. The deformation compatibility check should include both the wall and diaphragm deflection. For the diaphragm yielding design path, the elastic wall and diaphragm deflections should be computed based on a force level reduced by R_{dia} equal to 4.5. These elastic deflections should each be amplified by a factor equal to R_{dia} and the two added together to obtain the total deflection that must be considered for deflection compatibility. Note that the actual total displacement of the wall is likely to be smaller and the total displacement of the diaphragm larger than predicted but the combined total displacement will likely be close to the actual value that should be anticipated.

3. Continuity ties that extend across the diaphragm shall be provided in each direction.

   - Out-of-plane wall anchorage forces must be transferred deep into the diaphragm interior using continuity ties to properly develop resistance. The ties must be connected to the main diaphragm to transfer the out-of-plane wall anchorage forces into the main diaphragm. It is reasonable that the design force for a continuous tie reduces as it develops into the diaphragm; however, the tie force shall not be reduced below that required of Section 12.1.3 of ASCE/SEI 7-10.
   - For diaphragms with sub-diaphragms, out-of-plane wall anchorage forces shall be transferred into sub-diaphragms as currently allowed and from sub-diaphragms to continuity ties that extend across the diaphragm.

RP5-5.2 Limitations of the Proposed Design Path for Diaphragm Yielding

The proposed approach is limited to buildings of regular plan geometry that can be divided into rectangular diaphragm segments. Each diaphragm segment must be supported on all 4 sides, i.e., this does not apply to diaphragms that cantilever or resist load in rotation. The approach is not applicable to irregular-shaped diaphragms that are not devisable into rectangular sections because defining how to spread ductility in the diaphragm for the potential non-rectangular configurations is beyond the scope of this project. For irregular shapes and as an alternative for rectangular-shaped diaphragms, a design approach in which R_{dia} on the order of 2 to 3 is used without strengthened edges might be appropriate but would require additional analyses to confirm the adequacy of such an approach. The approach in which in-plane yielding of vertical elements is expected and the diaphragm is designed for diaphragm shears using the R factor for the vertical system but amplified by the overstrength factor, Ω_o, is applicable.

The proposed approach is only applicable to diaphragms that will perform equivalent to the diaphragms studied to develop the design procedure. For wood diaphragms, the connectors studied were common nails that connect the structural wood panel to wood framing members or nailers on steel framing. Steel deck diaphragms were also studied but as will be discussed in Section 5.3, the proposed approach should not be used for them at this time.

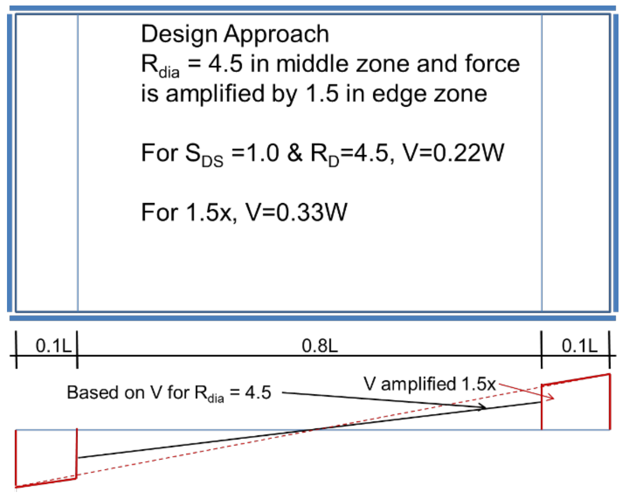

FIGURE 16 Description of approach for a period on the plateau of the design spectrum

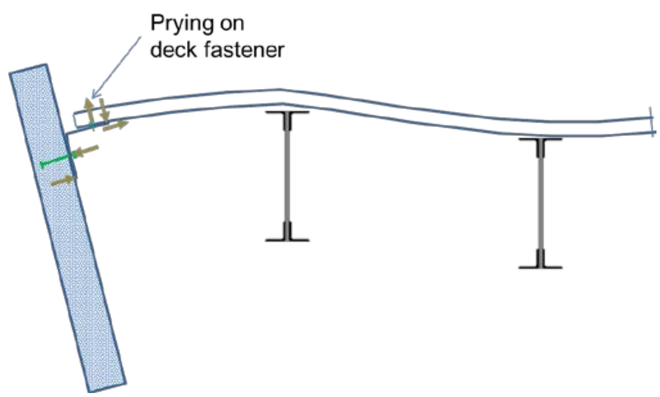

FIGURE 17 Deformations and change in angle between roof diaphragm and wall and prying on the connection to ledger

The proposed procedure also assumes that wall panels do not have pilasters. Currently, new construction of RWFD buildings rarely includes pilasters, especially for warehouse and retail use where pilasters obstruct installation of racks. Occasionally, they are included in gymnasiums and other rigid wall-flexible diaphragm buildings. Where they are used, the wall panels should be design for two-way action and the concentration of wall force at the top of the pilaster should be accounted for. This is usually addressed by adequately connecting the girder resting on the pilaster for higher out-of-plane forces and including these forces in the design of the girder.

RP5-5.3 Applicability of Proposed Design Approach to RWFD Buildings with Steel Deck Diaphragms

At this time the proposed design approach is not intended to apply to RWFD buildings with steel deck diaphragms. There are several reasons for not recommending its use even though it has been studied (Koliou, 2014). Reasons include (1) tests results of a large scale diaphragm showed significantly less distribution of yielding than the analyses show when loaded parallel to deck edges (Massarelli, et al., 2012), (2) steel deck diaphragms were never intended to be yielding elements as such design strengths are based on monotonic tests, (3) data for reverse cyclically loaded connections is sparse and missing for many commonly used deck gages, (4) the post-yield stiffness of connectors is positive for only a short deformation, 1 to 2 mm, (5) few reverse cyclically loaded diaphragm tests have been performed while

FEMA P695 requires comprehensive test data, and (6) many diaphragms in high seismic regions are designed using proprietary sidelaps for which no test data was available to include in the study. Additionally, the behavior of connectors simultaneously resisting forces from diaphragm shear with ductile connector behavior expected and forces from out-of-plane wall anchorage in which elastic connector behavior is expected must be understood. This issue will likely require design limitations that have yet to be identified.

This study included 20 and 22 ga steel deck diaphragms with puddle welds, powder actuated fasteners, and self-drilling screws for framing connectors. Sidelap connectors included in the study were button punches, self-drilling screws, and top seam welds. There is a potential that the proposed design approach could be applied to 20 and 22 ga diaphragms with the connectors studied. However, the concern of yielding not spreading as reported by Massarelli, et al. (2012) when the diaphragm is loaded parallel to the direction the deck sheets span must be understood and addressed. It could also be applied to new or proprietary connectors with positive post-yield stiffness to an acceptable deformation limit. Currently that limit is not known exactly but a value of 10 mm is likely sufficient.

Once a set of non-proprietary connections can be shown to result in acceptable diaphragm performance, additional connectors could be approved using the procedures of FEMA P795 (2011) *Quantification of Building Performance Factors: Component Equivalency Methodology*.

If the proposed procedure were to be applicable to diaphragms with welded connections the weld quality must be consistent with the weld quality used for the cyclic testing that led to the applicability. Detailed welding requirements should be developed as part of developing a design procedure from the proposed design approach. Recommendations for welding are provided in Guenfoud, et al. (2010). The factors to control for arc-spot welds (puddle welds) are high current setting, electrode type (E6011), and proper welding technique. Recommendations are made to increase the welding times relative to those recommended in SDI Manual (2004) for 1/8 in electrodes.

RP5-5.4 Discussion of Diaphragm Yielding

The design path for which diaphragm yielding is expected has the potential to provide improved diaphragm yielding relative to that of a diaphragm designed with current design procedures. However, the degree of improvement is dependent on how diaphragm shear strength is distributed for each design. For the proposed design approach in which the diaphragm yielding path is chosen, yielding generally starts in the general diaphragm field near the interface with the strengthened perimeter. Diaphragm yielding can then spread towards the center of the diaphragm span and to a lesser degree into the perimeter zone in which the diaphragm design strength is amplified.

Diaphragm yielding spreads towards the center of the diaphragm span because of (1) the post-yield strength characteristics of the common nails and (2) the distribution of forces acting on the diaphragm. The common nails exhibit positive post yield stiffness and substantial increases in strength that allows higher forces and yielding to develop in the diaphragm in the direction of the center of the diaphragm span. Additionally, the magnitude of average diaphragm shears along the length of the diaphragm span is not linear as typically assumed for design, including the proposed procedure. Instead, the diaphragm shear distribution from response-history analyses show the peak shears along the diaphragm span are more evenly distributed.

The degree to which the yielding spreads towards the diaphragm span center depends on the distribution of diaphragm shears for a particular time-history and the changes in diaphragm nailing along the length of the diaphragm. The most distributed yielding occurs if the diaphragm nailing has many zones that follow a linearly varying distribution of diaphragm shear strength with the weakest zone at the center and increasing in strength until it reaches the perimeter zone for which an over-strength factor is applied to the design.

The reason that diaphragm yielding may extend into the strengthened perimeter zone is that force is generated from the mass of walls and the roof. The diaphragm shears in this zone can continue to increase even if yielding occurs outside this zone. Although some yielding occurs in the strengthened perimeter zone, the intention of the design approach is too prevent concentrated yielding and significant diaphragm degradation in this region.

RP5-5.5 Quality of Design, Construction and Inspection

Like many seismic-force-resisting systems, the effectiveness of the proposed approach will be dependent on complying with design requirements. However, the design requirements are relatively simple in comparison to many other systems. The main change from current design is how strength is proportioned along the diaphragm span. As mentioned in Section 5.3, improved distribution of yielding can be obtained by using multiple zones of diaphragm strength but the procedure is not dependent on providing multiple zones of yielding.

Diaphragm designs using the proposed design approach are no more likely to be constructed improperly than current diaphragm designs. A common concern is the use of wrong nailing patterns or wrong nail type. If this is a concern, inspection of the nailing pattern and type of nails used should be included in the project inspection requirements. For larger footprint buildings, the roof diaphragm is usually part of panelized framing constructed on the ground and lifted into place. There is a small chance that panels could be placed in the wrong locations; however, inspection should include checking the nails in the panels where they are constructed and confirming that they are placed correctly within the roof structure.

RP5-5.6 Diaphragm Modifications and Deterioration

After initial construction, diaphragm strength can be affected by modifications and deterioration. The most common modification that reduces strength is adding openings through the diaphragm, which will reduce the strength approximately in proportion to the size of the openings in a line relative to the overall depth of the diaphragm. Large openings or many small ones can further reduce the strength and cause a concentration of deformations. This is also a concern with the current design approach. Openings near the center of the diaphragm are likely to have little or no effect on performance, and openings close to and within the strengthened region are likely to have a detrimental effect.

Deterioration of a diaphragm can occur due to roof leaks. Such damage is usually concentrated at or near the diaphragm to wall connections and at openings through the diaphragm. Deterioration is a concern but it is no more likely for the proposed design than it is for existing design.

RP5-6 EVALUATION OF PROPOSED DESIGN APPROACH

To validate the proposed design approach, archetype buildings were designed and P695 analyses were performed.

RP5-6.1 Description of Archetypes

Evaluations were performed on archetypes designed for high and moderate seismic risk as defined in Section 4.1. For these evaluations, archetypes were designed with either steel deck or wood panel diaphragms. Summary tables with detailed descriptions of diaphragm zones for the archetypes are in Appendix D for the wood diaphragms. The results of the steel deck diaphragms are included in Koliou (2014). The walls for these archetypes are reinforced-concrete wall panels 25 ft in length, 33 ft tall measured from the top of slab-on-grade, and 9-1/4 in. thick for high seismic risk and 7-1/4 in. thick for moderate seismic risk. The roof level is at 30 ft above the top of slab-on-grade with parapet walls cantilevering 3 ft higher than the roof level.

The archetypes are grouped by whether the diaphragm is large or small. The archetypes have plan aspect ratios of 1:1, 2:1 and 1:2. Large diaphragms are 400 ft long and either 400 ft or 200 ft wide. The small diaphragms with aspect ratios of 2:1 and 1:2 are 200 ft long by 100 ft wide, and for the 1:1 aspect ratios the diaphragms are 100 ft by 100 ft. The wood diaphragms are wood structural panels nailed to nailers that are attached to open web steel joists.

RP5-6.2 P695 Analysis Results for Proposed Design Approach

Incremental Dynamic Analyses (IDA) (Vamvatsikos and Cornell, 2002) using the FEMA P695 Far Field Ground Motion Ensemble (FEMA P695, 2009) were performed using the simplified models to compute the median collapse intensity. Summaries of collapse results for individual wood diaphragm archetypes are included in Appendix E. Adjusted collapse margin ratios (ACMR) and acceptable ACMR values for the individual archetypes and the archetype groups are also included in Appendix E. The acceptable ACMR values are computed using the beta factors listed in Table 2. The combined uncertainty, β_{Total}, is computed as the square root sum of the squares of the individual uncertainties. For β_{Total} equal to 0.66, the acceptable ACMR is 1.73 and 2.30 for 20% and 10% probability of collapse at Maximum Considered Earthquake (MCE) ground motion, respectively. The acceptable ACMR applicable for the performance of individual archetypes is 1.73, and the ACMR of performance groups is 2.30. Summaries of the adjusted collapse margin ratio results for the performance groups of wood archetypes are provided in Table 6. The acceptable ACMR of at least 2.30 is achieved for each performance group.

Table 6 Summary of collapse margin ratio results for wood structural panel diaphragms archetype performance groups

Performance Group	Seismicity	Size	Computed ACMR	Acceptable ACMR	Pass/Fail
PG-1N	High	Large	3.68	2.30	Pass
PG-2N	High	Small	2.51		Pass
PG-3N	Moderate	Large	3.90		Pass
PG-4N	Moderate	Small	3.07		Pass

The elastic periods for the archetypes designed in accordance with the proposed approach are listed in Table 7. In almost all cases the range of periods are 0 to 20% longer than the equivalent periods in Table 5 for the archetypes designed using the current code design approach.

The archetypes with larger diaphragms did better than the archetypes with small diaphragms, and the archetypes designed for moderate seismicity performed better than the ones designed for high seismicity. The small archetypes designed for high seismicity had the lowest ACMR. The range of fundamental elastic periods indicated in Table 7 is less than the transition period of 0.6 sec for most of these 100 and 200 ft long diaphragms. Thus, these archetypes are experiencing the short period issue described in Section 4.4 regarding the FEMA P695 analyses for the archetypes using the current design approach. The ACMR for small archetypes designed for moderate seismicity have longer periods than those designed for high seismicity and the transition period is only 0.4 sec. The short period issue is less of a factor for the small buildings designed for moderate seismicity. The elastic period values in Table 7 are all well above the transition period for the wood archetypes that are 400 ft long, thus, the high ACMRs for the large wood diaphragm performance groups, PG1 and PG3.

Table 7 Summary of Elastic Periods for Archetypes Designed Using the Proposed Approach

<table>
<tr><th colspan="2">Wood diaphragm span length</th><th>Period for high-seismic archetypes</th><th>Period for moderate-seismic archetypes</th></tr>
<tr><td colspan="2">400 ft</td><td>0.92 to 0.96 sec</td><td>0.92 to 0.93 sec</td></tr>
<tr><td colspan="2">200 ft</td><td>0.53 to 0.65 sec</td><td>0.57 to 0.61 sec</td></tr>
<tr><td>100 ft</td><td>0.39 to 0.42 sec</td><td colspan="2">0.45 to 0.46 sec</td></tr>
</table>

The ranges of building drift ratios for the performance groups are provided in Table 8. The building drift ratio is the sum of the wall drift ratio (WDR) and the diaphragm drift ratio (DDR) and serves as a damage index. The WDR is computed as the in-plane wall deformation divided by the height of the roof, and the DDR is computed as the diaphragm displacement divided by one-half of the diaphragm span. In general shorter diaphragm spans result in larger BDRs, especially for the wood diaphragms. This likely results from the wall drift ratio (WDR) being a larger percentage of the BDR for the smaller buildings.

Table 8 Range of building drift ratios (BDRs) for the wood diaphragm performance groups

Performance Group	Seismicity	Size	Median BDR @ DE (%)	Median BDR @ MCE (%)
PG-1N	High	Large	0.34 to 0.52	0.54 to 0.81
PG-2N	High	Small	0.79 to 0.85	1.14 to 1.31
PG-3N	Moderate	Large	0.34 to 0.39	0.55 to 0.65
PG-4N	Moderate	Small	0.53 to 0.61	0.90 to 1.21

RP5-7 SUMMARY

The existing design approach for one-story buildings with flexible roof diaphragms was evaluated and found to be inadequate according to the criteria of the FEMA P695 methodology. An issue with the existing approach is that it does not spread inelastic response (yielding) into the diaphragm. A new approach for the design of these buildings with wood diaphragms has been proposed that includes recognizing the diaphragm's own dynamic response. It requires strengthened regions at the ends of the diaphragm. Strengthening the diaphragm edges improves the distribution of yielding within the diaphragm. A simplified modeling framework was developed to perform the required incremental dynamic analyses. Simplified period formulas that account for both diaphragm and vertical element deformations were developed and incorporated into the proposed procedure.

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RP5-APPENDIX A

Refer to Koliou (2014) for additional information about the archetypes.

Table A1 Archetype Descriptions for Wood Diaphragms

Archetype ID	Diaphragm construction	Building size	Diaphragm aspect ratio	Building dimensions (ft)	Connector type (F=framing S=sidelap)	Seismic design category
HWL_21_N_OSB_RW4_04	Wood	Large	2:1	400x200	Common nails	D_{max}
HWL_12_N_OSB_RW4_02	Wood	Large	1:2	200x400	Common nails	D_{max}
HWL_11_N_OSB_RW4_01	Wood	Large	1:1	400x400	Common nails	D_{max}
HWS_21_N_OSB_RW4_01	Wood	Small	2:1	200x100	Common nails	D_{max}
HWS_12_N_OSB_RW4_01	Wood	Small	1:2	100x200	Common nails	D_{max}
HWS_11_N_OSB_RW4_01	Wood	Small	1:1	100x100	Common nails	D_{max}
HWS_11_N_OSB_RW4_02	Wood	Small	1:1	100x100	Common nails	D_{max}
MWL_11_N_OSB_RW4_01	Wood	Large	1:1	400x400	Common nails	C_{max}
MWL_21_N_OSB_RW4_01	Wood	Large	2:1	400x200	Common nails	C_{max}
MWL_12_N_OSB_RW4_01	Wood	Large	1:2	200x400	Common nails	C_{max}
MWS_11_N_OSB_RW4_01	Wood	Small	1:1	100x100	Common nails	C_{max}
MWS_21_N_OSB_RW4_01	Wood	Small	2:1	200x100	Common nails	C_{max}
MWS_12_N_OSB_RW4_01	Wood	Small	1:2	100x200	Common nails	C_{max}

Table A2 Archetype Descriptions for Steel Diaphragms for High-Seismic Risk – Existing Design

Archetype Designation	Diaphragm Dimensions	Zone	Zone Width (ft)	Connector Pattern / Deck Gage	Framing Connectors	Sidelap Connectors	Sidelap Spacing
High Seismicity – Large Buildings							
HSL_11_P_S_RW4	400 x 400 ft	3	56	36/11 – 22 ga	PAF	#10 screw	6 in.
		2	24	36/9 – 22 ga	PAF	#10 screw	9 in.
		1	120	36/9 – 22 ga	PAF	#10 screw	18 in.
HSL_11_W_WB_RW4	400 x 400 ft	3	56	36/7 – 22 ga	1/2 in. dia. welds	TSW	12 in.
		2	24	36/7 – 22 ga	1/2 in. dia. welds	BP	12 in.
		1	120	36/7 – 22 ga	1/2 in. dia. welds	BP	24 in.
HSL_21_W_WB_RW4_01	400 x 200 ft	3	50	36/7 – 22 ga	1/2 in. dia. Effective area welds (5/8 and 3/4 in. dia. at the surface)	TSW	6 in.
		2	54	36/7 – 22 ga	1/2 in. dia. Effective area welds (5/8 and 3/4 in. dia. at the surface)	TSW	12 in.
		1	96	36/7 – 22 ga	1/2 in. dia. Effective area welds (5/8 and 3/4 in. dia. at the surface)	BP	12 in.
HSL_21_P_S_RW4_01	400 x 200 ft	3	50	36/9 – 20 ga	PAF	#10 screws	3 in.
		2	60	36/9 – 22 ga	PAF	#10 screws	6 in.
		1	90	36/5 – 22 ga	PAF	#10 screws	6 in.
HSL_11_S_S_RW4	400 x 400 ft	3	66	36/11 – 22 ga	#12 or #14 HWH self-drilling screws by Hilti	#10 screws	3 in.
		2	24	36/9 – 22 ga	#12 or #14 HWH self-drilling screws by Hilti	#10 screws	6 in.
		1	120	36/9 – 22 ga	#12 or #14 HWH self-drilling screws by Hilti	#10 screws	9 in.
HSL_12_W_WB_RW4	Refer to HSL_21_W_WB for design						
HSL_12_P_S_RW4	Refer to HSL_21_P_S for design						
High Seismicity – Small Buildings							
HSS_11_W_B_RW4	100 x 100 ft	3	50	36/5 – 22 ga	1/2 in. dia. effective welds (5/8 in. and 3/4 in. at the surface)	BP	12 in.
		2	50	36/5 – 22 ga	1/2 in. dia. effective welds (5/8 in. and 3/4 in. at the surface)	BP	12 in.

Archetype Designation	Diaphragm Dimensions	Zone	Zone Width (ft)	Connector Pattern / Deck Gage	Framing Connectors	Sidelap Connectors	Sidelap Spacing
		1	50	36/5 – 22 ga	1/2 in. dia. effective welds (5/8 in. and 3/4 in. at the surface)	BP	12 in.
HSS_21_W_WB_RW4	200 x 100 ft	3	0	36/7 – 22 ga	1/2 in. dia. effective welds (5/8 in. and 3/4 in. dia. puddle welds at the surface)	TSW	9 in.
		2	40	36/7 – 22 ga	1/2 in. dia. effective welds (5/8 in. and 3/4 in. dia. puddle welds at the surface)	TSW	9 in.
		1	60	36/7 – 22 ga	1/2 in. dia. effective welds (5/8 in. and 3/4 in. dia. puddle welds at the surface)	BP	12 in.
HSS_12_W_WB_RW4	Refer to HSS_21_W_WB_RW4						
HSS_11_P_S_RW4	100 x 100 ft	3	0	36/9 – 22 ga	PAF	#10 screws	18 in.
		2	0	36/9 – 22 ga	PAF	#10 screws	18 in.
		1	50	36/9 – 22 ga	PAF	#10 screws	18 in.
HSS_11_S_S_RW4	100 x 100 ft	3	0	36/9 – 22 ga	#12	#10 screws	9 in.
		2	0	36/9 – 22 ga	#12	#10 screws	9 in.
		1	50	36/9 – 22 ga	#12	#10 screws	9 in.
HSS_21_P_S_RW4	200 x 100 ft	3	0	36/9 – 22 ga	PAF	#10 screws	3 in.
		2	40	36/9 – 22 ga	PAF	#10 screws	3 in.
		1	60	36/9 – 22 ga	PAF	#10 screws	9 in.
HSS_12_P_S_RW4	Refer to HSS_21_P_S_RW4						
HSS_21_S_S_RW4	200 x 100 ft	3	0	36/11 – 20 ga	#12	#10 screws	3 in.
		2	40	36/11 – 20 ga	#12	#10 screws	3 in.
		1	60	36/9 – 22 ga	#12	#10 screws	6 in.
HSS_12_S_S_RW4	Refer to HSS_21_S_S_RW4						

TABLE A3 Archetype Descriptions for Steel Diaphragms for Moderate-Seismic Risk – Existing Design

Archetype Designation	Diaphragm Dimensions	Zone	Zone Width (ft)	Connector Pattern / Deck Gage	Framing Connectors	Sidelap Connectors	Sidelap Spacing
Moderate Seismicity – Large Buildings							
MSL_21_P_S_RW4	400 x 200 ft	3	32	36/9 – 22 ga	PAF	#10 screws	15 in.
		2	36	36/9 – 22 ga	PAF	#10 screws	27 in.
		1	132	36/5 – 22 ga	PAF	#10 screws	15 in.
MSL_12_P_S_RW4	Refer to MSL_21_P_S_RW4						
MSL_21_S_S_RW4	400 x 200 ft	3	32	36/9 – 22 ga	#12 Hilti HWH screws	#10 screws	6 in.
		2	36	36/9 – 22 ga	#12 Hilti HWH screws	#10 screws	12 in.
		1	132	36/5 – 22 ga	#12 Hilti HWH screws	#10 screws	9 in.
MSL_12_S_S_RW4	Refer to MSL_21_S_S_RW4						
MSL_21_W_S_RW4	400 x 200 ft	3	32	36/7 – 22 ga	3/4 in. welds	#10 screws	6 in.
		2	36	36/7 – 22 ga	3/4 in. welds	#10 screws	12 in.
		1	132	36/5 – 22 ga	5/8 in. welds	#10 screws	9 in.
MSL_12_W_S_RW4	Refer to MSL_21_W_S_RW4						
MSL_11_P_S_RW4	400 x 400 ft	3	32	36/5 – 22 ga	PAF	#10 screws	12 in.
		2	36	36/5 – 22 ga	PAF	#10 screws	21 in.
		1	132	36/5 – 22 ga	PAF	#10 screws	36 in.
MSL_11_S_S_RW4	400 x 400 ft	3	32	36/5 – 22 ga	#12 Hilti HWH screws	#10 screws	9 in.
		2	36	36/5 – 22 ga	#12 Hilti HWH screws	#10 screws	12 in.
		1	132	36/5 – 22 ga	#12 Hilti HWH screws	#10 screws	21 in.
MSL_11_W_S_RW4	400 x 400 ft	3	32	36/5 – 22 ga	5/8 in. dia. welds	#10 screws	9 in.
		2	36	36/5 – 22 ga	5/8 in. dia. welds	#10 screws	18 in.
		1	132	36/5 – 22 ga	5/8 in. dia. welds	#10 screws	21 in.
MSL_21_W_W_RW4_01	400 x 200 ft	3	40	36/7 – 22 ga	¾ in. dia. welds	TSW	18 in.
		2	40	36/5 – 22 ga	¾ in. dia. welds	TSW	24 in.
		1	120	36/5 – 22 ga	¾ in. dia. welds	TSW	36 in.
MSL_12_W_W_RW4_01	Refer to MSL_21_W_W_RW4_01						
MSL_11_W_W_RW4_01	400 x 400 ft	3	40	36/5 – 22 ga	¾ in. dia. welds	TSW	36 in.
		2	40	36/4 – 22 ga	¾ in. dia. welds	TSW	36 in.
		1	120	36/4 – 22 ga	¾ in. dia. welds	TSW	72 in.
Moderate Seismicity – Small Buildings							
MSS_11_S_S_RW4	100 x 100 ft	3	0	36/5 – 22 ga	#12 HWH Hilti screw	#10 screws	36 in.
		2	0	36/5 – 22 ga	#12 HWH Hilti screw	#10 screws	36 in.
		1	50	36/5 – 22 ga	#12 HWH Hilti screw	#10 screws	36 in.
MSS_11_P_S_RW4	100 x 100 ft	3	0	36/5 – 22 ga	PAF	#10 screws	36 in.
		2	0	36/5 – 22 ga	PAF	#10 screws	36 in.

Archetype Designation	Diaphragm Dimensions	Zone	Zone Width (ft)	Connector Pattern / Deck Gage	Framing Connectors	Sidelap Connectors	Sidelap Spacing
		1	50	36/5 – 22 ga	PAF	#10 screws	36 in.
MSS_11_W_S_RW4	100 x 100 ft	3	0	36/5 – 22 ga	5/8 in. puddle welds	#10 screws	36 in.
		2	0	36/5 – 22 ga	5/8 in. puddle welds	#10 screws	36 in.
		1	50	36/5 – 22 ga	5/8 in. puddle welds	#10 screws	36 in.
MSS_21_S_S_RW4	200 x 100 ft	3	22	36/7 – 22 ga	#12 HWH Hilti screw	#10 screws	9 in.
		2	0	36/5 – 22 ga	#12 HWH Hilti screw	#10 screws	12 in.
		1	78	36/5 – 22 ga	#12 HWH Hilti screw	#10 screws	12 in.
MSS_12_S_S_RW4	Refer to MSS_21_S_S_RW4						
MSS_21_P_S_RW4	200 x 100 ft	3	22	36/7 – 22 ga	PAF	#10 screws	12 in.
		2	0	36/5 – 22 ga	PAF	#10 screws	21 in.
		1	78	36/5 – 22 ga	PAF	#10 screws	21 in.
MSS_12_P_S_RW4	Refer to MSS_21_P_S_RW4						
MSS_21_W_S_RW4	200 x 100 ft	3	22	36/7 – 22 ga	5/8 in. puddle welds	#10 screws	9 in.
		2	0	36/5 – 22 ga	5/8 in. puddle welds	#10 screws	12 in.
		1	78	36/5 – 22 ga	5/8 in. puddle welds	#10 screws	12 in.
MSS_12_W_S_RW4	Refer to MSS_21_W_S_RW4						

RP5-APPENDIX B

Table B1 Cyclic test data for nails

Connection type	Specimen characteristics	Number of specimens tested	Source
6d common nails (d=0.113in, l=2.0in)	7/16 OSB std.	10	Coyne (2007)
8d common nails (d=0.113in, l=2.5in)	2x4 Hem Fir. & 7/16 OSB std.	19	Christovasilis et al. (2009)
	2x6 Hem Fir. & 7/16 OSB std.	17	
8d common nails (d=0.131in, l=2.5in)	7/16 OSB std.	10	Coyne (2007)
10d common nails (d=0.148in, l=3.0in)	7/16 OSB std.	10	
	5/8 OSB std.	10	
	3/4 OSB std.	10	
10d box nails (d=0.131in, l=3.0in)	DF-L & 19/32 T&G	20	Fonseca et al. (2002)
	DF-L & 19/32 OSB std.	20	
#10 Rolled – Hardened screws (d=0.113in, l=2.0in)	DF-L & 7/16 OSB std.	20	

Table B2 Cyclic test data for steel deck sidelap connectors

Connection type	Specimen characteristics	Number of specimens tested	Corresponding Publication
Button punch (0.39in. diameter)	22 ga deck	2	Rogers and Tremblay (2003b)
	20 ga deck	2	
Screws (10-14x7/8 in.)	22 ga deck	2	
	20 ga deck	2	
Welds (1.38in. length)	22 ga deck	2	

Table B3 Cyclic test data for steel deck framing connectors

Connection type	Specimen characteristics	Number of specimens tested	Corresponding Publication
Powder-Actuated Fasteners (Hilti EDNK22-THO12 and Buildex BX12)	22 ga deck to 0.12 in. plate	4	Rogers and Tremblay (2003a)
	20 ga deck to 0.12 in. plate	4	
Powder-Actuated Fasteners (Hilti ENPH2-21-L15 and Buildex BX14)	22 ga deck to 0.79 in. plate	4	
	20 ga deck to 0.79 in. plate	4	
Buildex Screws (12-14x in.)	22 ga deck to 0.12 in. plate	2	
	20 ga deck to 0.12 in. plate	2	
Hilti Screws (12-24x7/8 in.)	22 ga deck to 0.12 in. plate	2	
	20 ga deck to 0.12 in. plate	2	
Welds (0.63 in. arc spot) & Washer	22 ga deck to 0.12 in. plate	1	
Welds (0.63 in. arc spot)	22 ga deck to 0.12 in. plate	2	
	22 ga deck to 0.79 in. plate	1	
	20 ga deck to 0.12 in. plate	2	
Welds (0.63in. – 0.75in. arc spot)	2 ply – 16 ga deck to 0.25 in. plate	4	Guenfoud, Tremblay & Rogers (2010)
	2 ply – 18 ga deck to 0.25 in. plate	4	
	2 ply – 20 ga deck to 0.25 in. plate	4	
	2 ply – 22 ga deck to 0.25 in. plate	4	
	4 ply – 16 ga deck to 0.25 in. plate	4	
	4 ply – 18 ga deck to 0.25 in. plate	3	
	4 ply – 20 ga deck to 0.25 in. plate	4	
	4 ply – 22 ga deck to 0.25 in. plate	4	

RP5-APPENDIX C

Results for the FEMA P695 collapse analyses for the current design approach are presented in this appendix. The following notations and abbreviations are used in the tables:

ACMR – Adjusted Collapse Margin Ratio where: $ACMR = SSF_i \times CMR_i$

BDR – Building Drift Ratio where: $BDR(\%) = DDR(\%) + WDR(\%)$

CMR – Collapse Margin Ratio where: $CMR_i = \frac{S_{CT}}{S_{MT}}$

DE – Design Earthquake

DDR – Diaphragm Drift Ratio (%) where: $DDR(\%) = \frac{X_{mid,roof}}{L_{roof}/2} \times 100$

$X_{mid,roof}$ is the diaphragm deflection at mid length

L_{roof} is the diaphragm span

MCE – Maximum Considered Earthquake

SSF – Spectral Shape Function

S_{CT} [T]– Median collapse intensity obtained from nonlinear dynamic analysis at the fundamental elastic period

S_{MT} [T]– Ground motion spectral demand at maximum considered earthquake (MCE) intensity level at the fundamental elastic period

WDR(%) – In-plane Wall Drift Ratio where: $WDR(\%) = \frac{X_{in-plane\ walls}}{h_{wall}} \times 100$

$X_{in\text{-}plane\ walls}$ is the deflection at the roof level of the in-plane walls

h_{wall} is the height of roof above the foundation

μ_T – Period based ductility where: $\mu_T = \frac{\delta_u}{\delta_{y,eff}}$

δ_u is the ultimate building drift ratio (BDR)

$\delta_{y,eff}$ is the effective yield building drift ratio (BDR)

Table C1 Summary of collapse results for RWFD buildings incorporating wood roof diaphragm archetype designs

Archetype ID	Design configuration				Pushover and IDA results			Relevant analysis parameters		
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	S_{MT}[T] (g)	S_{CT}[T] (g)	CMR	Period (sec)	Median BDR @ DE (%)	Median BDR @ MCE (%)
Performance Group No. PG-1E (Wood, Large Building, Existing Design)										
HWL_21_N_OSB_RW4_07	Large	2:1	Wood Panelized	D_{max}	1.06	2.00	1.89	0.85	0.29	0.55
HWL_12_N_OSB_RW4_02	Large	1:2	Wood Panelized	D_{max}	1.50	3.36	2.24	0.49	0.42	0.64
HWL_11_N_OSB_RW4_01	Large	1:1	Wood Panelized	D_{max}	1.05	1.68	1.60	0.87	0.32	0.53
Performance Group No. PG-2E (Wood, Small Building, Existing Design)										
HWS_21_N_OSB_RW4_01	Small	2:1	Wood Panelized	D_{max}	1.41	1.98	1.40	0.54	0.56	1.16
HWS_12_N_OSB_RW4_01	Small	1:2	Wood Panelized	D_{max}	1.50	2.64	1.76	0.38	0.72	1.72
HWS_11_N_OSB_RW4_01	Small	1:1	Wood Panelized	D_{max}	1.50	2.21	1.47	0.36	0.65	1.68
HWS_11_N_OSB_RW4_02	Small	1:1	Wood Panelized	D_{max}	1.50	2.32	1.55	0.37	0.68	1.63
Performance Group No. PG-3E (Wood, Large Building, Existing Design)										
MWL_21_N_OSB_RW4_01	Large	2:1	Wood Panelized	C_{max}	0.50	1.28	2.56	0.90	0.31	0.52
MWL_12_N_OSB_RW4_01	Large	1:2	Wood Panelized	C_{max}	0.75	1.75	2.33	0.55	0.39	0.63
MWL_11_N_OSB_RW4_01	Large	1:1	Wood Panelized	C_{max}	0.51	1.09	2.14	0.92	0.35	0.55
Performance Group No. PG-4E (Wood, Small Building, Existing Design)										
MWS_21_N_OSB_RW4_01	Small	2:1	Wood Panelized	C_{max}	0.75	1.15	1.50	0.58	0.51	0.89
MWS_12_N_OSB_RW4_01	Small	1:2	Wood Panelized	C_{max}	0.75	1.35	1.72	0.43	0.55	0.98
MWS_11_N_OSB_RW4_01	Small	1:1	Wood Panelized	C_{max}	0.75	1.47	1.96	0.45	0.59	1.15

Table C2 Adjusted collapse margin ratios and acceptable collapse margin ratios for RWFD buildings incorporating wood roof diaphragm archetype designs

Archetype ID	Design configuration				Collapse margin parameters				Acceptance check	
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	CMR	μ_T	SSF	ACMR	Accept. ACMR	Pass/Fail
Performance Group No. PG-1E (Wood, Large Building, Existing Design)										
HWL_21_N_OSB_RW4_07	Large	2:1	Wood	D_{max}	1.89	8.07	1.43	2.70	1.73	Pass
HWL_12_N_OSB_RW4_02	Large	1:2	Wood	D_{max}	2.24	8.95	1.36	3.05	1.73	Pass
HWL_11_N_OSB_RW4_01	Large	1:1	Wood	D_{max}	1.60	8.42	1.43	2.29	1.73	Pass
Mean of Performance Group:					1.91	8.48	1.41	2.68	2.30	Pass
Performance Group No. PG-2E (Wood, Small Building, Existing Design)										
HWS_21_N_OSB_RW4_01	Small	2:1	Wood	D_{max}	1.40	7.20	1.31	1.84	1.73	Pass
HWS_12_N_OSB_RW4_01	Small	1:2	Wood	D_{max}	1.76	8.95	1.36	2.39	1.73	Pass
HWS_11_N_OSB_RW4_01	Small	1:1	Wood	D_{max}	1.55	8.32	1.36	2.10	1.73	Pass
HWS_11_N_OSB_RW4_02	Small	1:1	Wood	D_{max}	1.55	8.51	1.36	2.11	1.73	Pass
Mean of Performance Group:					1.57	8.25	1.35	2.11	2.30	Fail
Performance Group No. PG-3E (Wood, Large Building, Existing Design)										
MWL_21_N_OSB_RW4_01	Large	2:1	Wood	C_{max}	2.56	8.15	1.14	2.92	1.73	Pass
MWL_12_N_OSB_RW4_01	Large	1:2	Wood	C_{max}	2.33	8.79	1.14	2.66	1.73	Pass
MWL_11_N_OSB_RW4_01	Large	1:1	Wood	C_{max}	2.14	8.54	1.14	2.44	1.73	Pass
Mean of Performance Group:					2.34	8.49	1.14	2.67	2.30	Pass
Performance Group No. PG-4E (Wood, Small Building, Existing Design)										
MWS_21_N_OSB_RW4_01	Small	2:1	Wood	C_{max}	1.50	8.05	1.16	1.74	1.73	Pass
MWS_12_N_OSB_RW4_01	Small	1:2	Wood	C_{max}	1.72	8.49	1.14	1.96	1.73	Pass
MWS_11_N_OSB_RW4_01	Small	1:1	Wood	C_{max}	1.96	8.09	1.14	2.23	1.73	Pass
Mean of Performance Group:					1.73	8.21	1.15	1.98	2.30	Fail

Table C3 Summary of collapse results for RWFD buildings incorporating steel roof diaphragm archetype designs

Archetype ID	Design configuration				Pushover and IDA results			Relevant analysis parameters		
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	S_{MT}[T] (g)	S_{CT}[T] (g)	CMR	Period (sec)	Median BDR @ DE (%)	Median BDR @ MCE (%)
Performance Group No. PG-5E (Steel, Large Building, Welds and Button Punches as sidelap Connectors, Existing Design)										
HSL_21_W_WB_RW4_01	Large	2:1	Steel (WR Deck)	D_{max}	1.50	1.56	0.99	0.52	0.19	0.45
HSL_12_W_WB_RW4_01	Large	1:2	Steel (WR Deck)	D_{max}	1.50	2.85	1.90	0.39	0.28	0.71
HSL_11_W_WB_RW4_01	Large	1:1	Steel (WR Deck)	D_{max}	1.50	1.43	0.95	0.49	0.13	0.42
Performance Group No. PG-6E (Steel, Large Building, Screws as sidelap Connectors, Existing Design)										
HSL_21_P_S_RW4_01	Large	2:1	Steel (WR Deck)	D_{max}	1.50	1.85	1.23	0.54	0.15	0.33
HSL_12_P_S_RW4_01	Large	1:2	Steel (WR Deck)	D_{max}	1.50	3.10	2.07	0.41	0.25	0.56
HSL_11_P_S_RW4_01	Large	1:1	Steel (WR Deck)	D_{max}	1.50	1.69	1.13	0.56	0.18	0.34
HSL_11_S_S_RW4_01	Large	1:1	Steel (WR Deck)	D_{max}	1.50	1.73	1.15	0.51	0.16	0.29
Performance Group No. PG-7E (Steel, Small Building, Button Punches as sidelap Connectors, Existing Design)										
HSS_11_W_B_RW4_01	Small	1:1	Steel (WR Deck)	D_{max}	1.50	2.59	1.73	0.21	0.17	0.39
HSS_21_W_B_RW4_01	Small	2:1	Steel (WR Deck)	D_{max}	1.50	2.13	1.42	0.35	0.21	0.44
HSS_12_W_B_RW4_01	Small	1:2	Steel (WR Deck)	D_{max}	1.50	2.85	1.90	0.26	0.15	0.35
Performance Group No. PG-8E (Steel, Small Building, Screws as sidelap Connectors, Existing Design)										
HSS_11_P_S_RW4_01	Small	1:1	Steel (WR Deck)	D_{max}	1.50	2.33	1.55	0.23	0.16	0.41
HSS_11_S_S_RW4_01	Small	1:1	Steel (WR Deck)	D_{max}	1.50	2.15	1.43	0.25	0.14	0.36
HSS_21_P_S_RW4_01	Small	2:1	Steel (WR Deck)	D_{max}	1.50	1.99	1.33	0.37	0.19	0.35
HSS_12_P_S_RW4_01	Small	1:2	Steel (WR Deck)	D_{max}	1.50	2.56	1.71	0.24	0.17	0.31
HSS_21_S_S_RW4_01	Small	2:1	Steel (WR Deck)	D_{max}	1.50	1.87	1.25	0.42	0.15	0.42
HSS_12_S_S_RW4_01	Small	1:2	Steel (WR Deck)	D_{max}	1.50	2.13	1.42	0.22	0.14	0.39
Performance Group No. PG-9E (Steel, Large Building, Screws as sidelap Connectors, Existing Design)										
MSL_21_P_S_RW4_01	Large	2:1	Steel (WR Deck)	C_{max}	0.68	0.93	1.35	0.66	0.29	0.41
MSL_12_P_S_RW4_01	Large	1:2	Steel (WR Deck)	C_{max}	0.75	1.50	2.01	0.53	0.19	0.33
MSL_21_S_S_RW4_01	Large	2:1	Steel (WR Deck)	C_{max}	0.63	0.96	1.52	0.71	0.22	0.35
MSL_12_S_S_RW4_01	Large	1:2	Steel (WR Deck)	C_{max}	0.75	1.41	1.88	0.55	0.21	0.32

Archetype ID	Design configuration				Pushover and IDA results			Relevant analysis parameters		
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	S_{MT}[T] (g)	S_{CT}[T] (g)	CMR	Period (sec)	Median BDR @ DE (%)	Median BDR @ MCE (%)
MSL_21_W_S_RW4_01	Large	2:1	Steel (WR Deck)	C_{max}	0.62	1.15	1.41	0.73	0.20	0.31
MSL_12_W_S_RW4_01	Large	1:2	Steel (WR Deck)	C_{max}	0.75	1.47	1.96	0.52	0.23	0.34
MSL_11_P_S_RW4_01	Large	1:1	Steel (WR Deck)	C_{max}	0.70	1.27	1.81	0.65	0.24	0.39
MSL_11_S_S_RW4_01	Large	1:1	Steel (WR Deck)	C_{max}	0.70	1.10	1.58	0.65	0.21	0.45
MSL_11_W_S_RW4_01	Large	1:1	Steel (WR Deck)	C_{max}	0.67	1.11	1.66	0.68	0.25	0.47
Performance Group No. PG-10E (Steel, Large Building, Welds as sidelap Connectors, Existing Design)										
MSL_21__W_W_RW4_01	Large	2:1	Steel (WR Deck)	C_{max}	0.73	1.02	1.39	0.62	0.19	0.35
MSL_12__W_W_RW4_01	Large	1:2	Steel (WR Deck)	C_{max}	0.75	1.50	1.99	0.51	0.24	0.41
MSL_11__W_W_RW4_01	Large	1:1	Steel (WR Deck)	C_{max}	0.74	1.13	1.52	0.61	0.18	0.36
Performance Group No. PG-11E (Steel, Small Building, Screws as sidelap Connectors, Existing Design)										
MSS_11_S_S_RW4_01	Small	1:1	Steel (WR Deck)	C_{max}	0.75	1.26	1.67	0.32	0.15	0.36
MSS_11_P_S_RW4_01	Small	1:1	Steel (WR Deck)	C_{max}	0.75	1.34	1.78	0.30	0.17	0.41
MSS_11_W_S_RW4_01	Small	1:1	Steel (WR Deck)	C_{max}	0.75	0.98	1.30	0.28	0.14	0.37
MSS_21_S_S_RW4_01	Small	2:1	Steel (WR Deck)	C_{max}	0.75	1.05	1.39	0.54	0.19	0.48
MSS_12_S_S_RW4_01	Small	1:2	Steel (WR Deck)	C_{max}	0.75	1.39	1.85	0.31	0.15	0.29
MSS_21_P_S_RW4_01	Small	2:1	Steel (WR Deck)	C_{max}	0.75	1.17	1.55	0.59	0.20	0.44
MSS_12_P_S_RW4_01	Small	1:2	Steel (WR Deck)	C_{max}	0.75	1.48	1.97	0.28	0.16	0.32
MSS_21_W_S_RW4_01	Small	2:1	Steel (WR Deck)	C_{max}	0.75	1.08	1.43	0.55	0.18	0.36
MSS_12_W_S_RW4_01	Small	1:2	Steel (WR Deck)	C_{max}	0.75	1.28	1.71	0.33	0.17	0.31

Table C4 Computed and acceptable adjusted collapse margin ratios for RWFD buildings incorporating steel roof diaphragm archetype designs

Archetype ID	Design configuration				Collapse margin parameters				Acceptance check	
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	CMR	μ_T	SSF	ACMR	Accept. ACMR	Pass/Fail
Performance Group No. PG-5E (Steel, Large Building, Welds and Button Punches as sidelap Connectors, Existing Design)										
HSL_21_W_WB_RW4_01	Large	2:1	Steel	D_{max}	0.99	8.09	1.34	1.33	1.73	Fail
HSL_12_W_WB_RW4_01	Large	1:2	Steel	D_{max}	1.90	8.26	1.33	2.53	1.73	Pass
HSL_11_W_WB_RW4_01	Large	1:1	Steel	D_{max}	0.95	8.16	1.33	1.27	1.73	Fail
Mean of Performance Group:					1.28	8.17	1.33	1.71	2.30	Fail
Performance Group No. PG-6E (Steel, Large Building, Screws as sidelap Connectors, Existing Design)										
HSL_21_P_S_RW4_01	Large	2:1	Steel	D_{max}	1.23	8.24	1.35	1.67	1.73	Fail
HSL_12_P_S_RW4_01	Large	1:2	Steel	D_{max}	2.07	8.14	1.33	2.75	1.73	Pass
HSL_11_P_S_RW4_01	Large	1:1	Steel	D_{max}	1.13	8.26	1.36	1.53	1.73	Fail
HSL_11_S_S_RW4_01	Large	1:1	Steel	D_{max}	1.15	8.01	1.33	1.53	2.73	Fail
Mean of Performance Group:					1.40	8.16	1.34	1.87	2.30	Fail
Performance Group No. PG-7E (Steel, Small Building, Button Punches as sidelap Connectors, Existing Design)										
HSS_11_W_B_RW4_01	Small	1:1	Steel	D_{max}	1.73	7.94	1.32	2.28	1.73	Pass
HSS_21_W_B_RW4_01	Small	2:1	Steel	D_{max}	1.42	8.05	1.33	1.89	1.73	Pass
HSS_12_W_B_RW4_01	Small	1:2	Steel	D_{max}	1.90	7.91	1.32	2.51	1.73	Pass
Mean of Performance Group:					1.68	7.97	1.32	2.23	2.30	Fail
Performance Group No. PG-8E (Steel, Small Building, Screws as sidelap Connectors, Existing Design)										
HSS_11_P_S_RW4_01	Small	1:1	Steel	D_{max}	1.55	8.02	1.33	2.07	1.73	Pass
HSS_11_S_S_RW4_01	Small	1:1	Steel	D_{max}	1.43	8.15	1.33	1.91	1.73	Pass
HSS_21_P_S_RW4_01	Small	2:1	Steel	D_{max}	1.33	8.33	1.33	1.76	1.73	Pass
HSS_12_P_S_RW4_01	Small	1:2	Steel	D_{max}	1.71	8.25	1.33	2.27	1.73	Pass
HSS_21_S_S_RW4_01	Small	2:1	Steel	D_{max}	1.25	7.85	1.32	1.65	1.73	Fail
HSS_12_S_S_RW4_01	Small	1:2	Steel	D_{max}	1.42	8.06	1.33	1.89	1.73	Pass
Mean of Performance Group:					1.45	8.11	1.33	1.92	2.30	Fail
Performance Group No. PG-9E (Steel, Large Building, Screws as sidelap Connectors, Existing Design)										
MSL_21_P_S_RW4_01	Large	2:1	Steel	C_{max}	1.35	8.32	1.15	1.55	1.73	Fail

Archetype ID	Design configuration				Collapse margin parameters				Acceptance check	
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	CMR	μ_T	SSF	ACMR	Accept. ACMR	Pass/Fail
MSL_12_P_S_RW4_01	Large	1:2	Steel	C_{max}	2.01	7.85	1.13	2.27	1.73	Pass
MSL_21_S_S_RW4_01	Large	2:1	Steel	C_{max}	1.52	8.02	1.18	1.80	1.73	Pass
MSL_12_S_S_RW4_01	Large	1:2	Steel	C_{max}	1.88	8.38	1.15	2.16	1.73	Pass
MSL_21_W_S_RW4_01	Large	2:1	Steel	C_{max}	1.41	8.20	1.18	1.66	1.73	Fail
MSL_12_W_S_RW4_01	Large	1:2	Steel	C_{max}	1.96	8.02	1.15	2.25	1.73	Pass
MSL_11_P_S_RW4_01	Large	1:1	Steel	C_{max}	1.81	7.99	1.15	2.08	1.73	Pass
MSL_11_S_S_RW4_01	Large	1:1	Steel	C_{max}	1.58	8.15	1.15	1.81	1.73	Pass
MSL_11_W_S_RW4_01	Large	1:1	Steel	C_{max}	1.66	8.22	1.17	1.94	1.73	Pass
Mean of Performance Group:					1.69	8.13	1.16	1.95	2.30	Fail
Performance Group No. PG-10E (Steel, Large Buildings, Welds as Sidelap connectors, Existing Design)										
MSL_21__W_W_RW4_01	Large	2:1	Steel	C_{max}	1.39	7.76	1.15	1.59	1.73	Fail
MSL_12__W_W_RW4_01	Large	1:2	Steel	C_{max}	1.99	8.15	1.14	2.27	1.73	Pass
MSL_11__W_W_RW4_01	Large	1:1	Steel	C_{max}	1.52	8.09	1.16	1.76	1.73	Pass
Mean of Performance Group:					1.63	8.00	1.15	1.88	2.30	Fail
Performance Group No. PG-11E (Steel, Small Building, Screws as sidelap Connectors, Existing Design)										
MSS_11_S_S_RW4_01	Small	1:1	Steel	C_{max}	1.67	8.15	1.14	1.91	1.73	Pass
MSS_11_P_S_RW4_01	Small	1:1	Steel	C_{max}	1.78	8.26	1.14	2.03	1.73	Pass
MSS_11_W_S_RW4_01	Small	1:1	Steel	C_{max}	1.30	7.78	1.13	1.47	1.73	Fail
MSS_21_S_S_RW4_01	Small	2:1	Steel	C_{max}	1.39	8.05	1.15	1.60	1.73	Fail
MSS_12_S_S_RW4_01	Small	1:2	Steel	C_{max}	1.85	8.56	1.14	2.11	1.73	Pass
MSS_21_P_S_RW4_01	Small	2:1	Steel	C_{max}	1.55	8.12	1.16	1.80	1.73	Pass
MSS_12_P_S_RW4_01	Small	1:2	Steel	C_{max}	1.97	7.88	1.14	2.25	1.73	Pass
MSS_21_W_S_RW4_01	Small	2:1	Steel	C_{max}	1.43	7.97	1.15	1.65	1.73	Fail
MSS_12_W_S_RW4_01	Small	1:2	Steel	C_{max}	1.71	8.02	1.14	1.95	1.73	Pass
Mean of Performance Group:					1.63	8.09	1.14	1.86	2.30	Fail

RP5-APPENDIX D

Table D1 Archetype Descriptions for Wood Diaphragms for High- and Moderate-Seismic Risk – Proposed Design

	Diaphragm Dimensions	Zone	Zone Width (ft)	Sheathing	10d Common Nail Spacing		
					Boundaries and Continuous Edges	Other Edges	Intermediate Field Area
High-Seismicity Wood, Large Buildings							
HWL_21_N_OSB_RD4.5-1.5_01 HWL_12_N_OSB_RD4.5-1.5_01	400 x 200 ft	End	40 ft	15/32 in. OSB	2 in. o.c.	3 in. o.c.	12 in. o.c.
		Middle	320 ft	15/32 in. OSB	2-1/2 in. o.c.	4 in. o.c.	12 in. o.c.
HWL_21_N_OSB_RD4.5-1.5_02 HWL_12_N_OSB_RD4.5-1.5_02	400 x 200 ft	End	40 ft	15/32 in. OSB	2 lines each at 2-1/2 in. o.c.	2 lines each at 4 in. o.c.*	12 in. o.c.
		Middle	320 ft	15/32 in. OSB	2-1/2 in. o.c.	4 in. o.c.	12 in. o.c.
HWL_21_N_OSB_RD4.5_1.5_03	400 x 200 ft	End	40 ft	15/32 in. OSB	2 lines each at 2-1/2 in. o.c.	2 lines each at 3 in. o.c.*	12 in. o.c.
		Middle	320 ft	15/32 in. OSB	2 in. o.c.	3 in. o.c.	12 in. o.c.
HWL_21_N_OSB_RD4.5_1.5_04	400 x 200 ft	End	40 ft	15/32 in. OSB	2 lines each at 2-1/2 in. o.c.	2 lines each at 3 in. o.c.*	12 in. o.c.
		Middle	320 ft	15/32 in. OSB	2-1/2 in. o.c.	4 in. o.c.	12 in. o.c.
HWL_11_N_OSB_RD4.5-1.5_01	400 x 400 ft	End	40 ft	15/32 in. OSB	2 in. o.c.	3 in. o.c.**	12 in. o.c.
		Middle	320 ft	15/32 in. OSB	4 in. o.c.	6 in. o.c.	12 in. o.c.
HWL_11_N_OSB_RD4.5-1.5_02	400 x 400 ft	End	40 ft	15/32 in. OSB			
		Middle	320 ft	15/32 in. OSB			
High-Seismicity Wood, Small Buildings							
HWS_21_N_OSB_RD4.5-1.5_01 HWS_12_N_OSB_RD4.5-1.5_01	200 x 100 ft	End	20 ft	15/32 in. OSB	2 lines each at 2-1/2 in. o.c.	2 lines each at 3 in. o.c.*	12 in. o.c.
		Middle	160 ft	15/32 in. OSB	2 in. o.c.	3 in. o.c.**	12 in. o.c.
HWS_11_N_OSB_RD4.5_1.5_01	100 x 100 ft	End	10 ft	15/32 in. OSB	2-1/2 in. o.c.	4 in. o.c.	12 in. o.c.
		Middle	80 ft	15/32 in. OSB	4 in. o.c.	6 in. o.c.	12 in. o.c.
Moderate-Seismicity Wood, Large Buildings							
MWL_21_N_OSB_RD4.5-1.5_01 MWL_12_N_OSB_RD4.5-1.5_01	400 x 200 ft	End	0 ft	15/32 in. OSB	6 in. o.c.	6 in. o.c.	12 in. o.c.
		Middle	400 ft	15/32 in. OSB	6 in. o.c.	6 in. o.c.	12 in. o.c.
MWL_11_N_OSB_RD4.5-1.5_01	400 x 400 ft	End	0 ft	15/32 in. OSB	6 in. o.c.	6 in. o.c.	12 in. o.c.
		Middle	400 ft	15/32 in. OSB	6 in. o.c.	6 in. o.c.	12 in. o.c.
Moderate-Seismicity Wood, Small Buildings							
MWS_21_N_OSB_RD4.5-1.5_01 MWS_12_N_OSB_RD4.5-1.5_01	200 x 100 ft	End	20 ft	15/32 in. OSB	4 in. o.c.	6 in. o.c.	12 in. o.c.
		Middle	160 ft	15/32 in. OSB	6 in. o.c.	6 in. o.c.	12 in. o.c.

	Diaphragm Dimensions	Zone	Zone Width (ft)	Sheathing	10d Common Nail Spacing		
					Boundaries and Continuous Edges	Other Edges	Intermediate Field Area
MWS_11_N_OSB_RD4.5-1.5_01	100 x 100 ft	End	0 ft	15/32 in. OSB	6 in. o.c.	6 in. o.c.	12 in. o.c.
		Middle	100 ft	15/32 in. OSB	6 in. o.c.	6 in. o.c.	12 in. o.c.

*Special 4x4 subpurlins at other edges in lieu of 2x4s
**Special 3x4 subpurlins at other edges in lieu of 2x4s

RP5-APPENDIX E

Table E1 Summary of collapse results for RWFD buildings incorporating wood roof diaphragm archetype designs

Archetype ID	Design Configuration				Pushover and IDA Results			Relevant Analysis Parameters		
	Building Size	Diaphragm Aspect Ratio	Diaphragm Construction	Seismic SDC	SMT[T] (g)	SCT[T] (g)	CMR	Period (sec)	Median BDR @ DE (%)	Median BDR @ MCE (%)
Performance Group No. PG-1N (Wood, Large Building, New Design)										
HWL_21_N_OSB_RD4.5-1.5_01	Large	2:1	Wood	D_{max}	0.95	2.56	2.69	0.93	0.44	0.60
HWL_21_N_OSB_RD4.5-1.5_02	Large	2:1	Wood	D_{max}	0.92	2.91	3.16	0.94	0.41	0.56
HWL_21_N_OSB_RD4.5-1.5_03	Large	2:1	Wood	D_{max}	0.96	2.65	2.59	0.92	0.43	0.60
HWL_21_N_OSB_RD4.5-1.5_04	Large	2:1	Wood	D_{max}	0.94	2.94	3.13	0.94	0.42	0.54
HWL_12_N_OSB_RD4.5-1.5_01	Large	1:2	Wood	D_{max}	1.50	3.65	2.43	0.53	0.51	0.78
HWL_12_N_OSB_RD4.5-1.5_02	Large	1:2	Wood	D_{max}	1.50	3.63	2.42	0.55	0.52	0.81
HWL_11_N_OSB_RD4.5-1.5_01	Large	1:1	Wood	D_{max}	0.91	2.11	2.32	0.96	0.34	0.57
HWL_11_N_OSB_RD4.5-1.5_02	Large	1:1	Wood	D_{max}	0.93	2.15	2.39	0.95	0.35	0.59
Performance Group No. PG-2N (Wood, Small Building, New Design)										
HWS_21_N_OSB_RD4.5-1.5_01	Small	2:1	Wood	D_{max}	1.36	2.45	1.80	0.65	0.83	1.14
HWS_12_N_OSB_RD4.5-1.5_01	Small	1:2	Wood	D_{max}	1.50	2.95	1.97	0.42	0.85	1.24
HWS_11_N_OSB_RD4.5-1.5_01	Small	1:1	Wood	D_{max}	1.50	2.84	1.89	0.39	0.79	1.31
Performance Group No. PG-3N (Wood, Large Building, New Design)										
MWL_21_N_OSB_RD4.5-1.5_01	Large	2:1	Wood	C_{max}	0.49	1.68	3.43	0.92	0.34	0.55
MWL_12_N_OSB_RD4.5-1.5_01	Large	1:2	Wood	C_{max}	0.75	2.08	2.77	0.57	0.41	0.65
MWL_11_N_OSB_RD4.5-1.5_01	Large	1:1	Wood	C_{max}	0.48	1.76	3.52	0.93	0.39	0.57
Performance Group No. PG-4N (Wood, Small Building, New Design)										
MWS_21_N_OSB_RD4.5-1.5_01	Small	2:1	Wood	C_{max}	0.74	1.61	2.18	0.61	0.53	0.90
MWS_12_N_OSB_RD4.5-1.5_01	Small	1:2	Wood	C_{max}	0.75	2.08	2.77	0.45	0.67	0.99
MWS_11_N_OSB_RD4.5-1.5_01	Small	1:1	Wood	C_{max}	0.75	2.35	3.13	0.46	0.61	1.21

Table E2 Adjusted collapse margin ratios and acceptable collapse margin ratios for RWFD buildings incorporating wood roof diaphragm archetype designs

Archetype ID	Design Configuration				Collapse Margin Parameters				Acceptance Check	
	Building Size	Diaphragm Aspect Ratio	Diaphragm Construction	Seismic SDC	CMR	μ_T	SSF	ACMR	Accept. ACMR	Pass/Fail
Performance Group No. PG-1N (Wood, Large Building, New Design)										
HWL_21_N_OSB_RD4.5-1.5_01	Large	2:1	Wood	D_{max}	2.69	8.44	1.45	3.91	1.73	Pass
HWL_21_N_OSB_RD4.5-1.5_02	Large	2:2	Wood	D_{max}	3.16	8.37	1.45	4.59	1.73	Pass
HWL_21_N_OSB_RD4.5-1.5_03	Large	2:1	Wood	D_{max}	2.65	8.43	1.45	3.84	1.73	Pass
HWL_21_N_OSB_RD4.5-1.5_04	Large	2:1	Wood	D_{max}	3.13	8.35	1.45	4.54	1.73	Pass
HWL_12_N_OSB_RD4.5-1.5_01	Large	1:2	Wood	D_{max}	2.43	8.84	1.34	3.26	1.73	Pass
HWL_12_N_OSB_RD4.5-1.5_02	Large	1:2	Wood	D_{max}	2.45	8.87	1.34	3.29	1.73	Pass
HWL_11_N_OSB_RD4.5-1.5_01	Large	1:1	Wood	D_{max}	2.32	8.34	1.45	3.36	1.73	Pass
HWL_11_N_OSB_RD4.5-1.5_02	Large	1:1	Wood	D_{max}	2.39	8.38	1.45	3.46	2.73	Pass
Mean of Performance Group:					2.58	8.50	1.42	3.68	2.58	8.50
Performance Group No. PG-2N (Wood, Small Building, New Design)										
HWS_21_N_OSB_RD4.5-1.5_01	Small	2:1	Wood	D_{max}	1.80	7.59	1.35	2.43	1.73	Pass
HWS_12_N_OSB_RD4.5-1.5_01	Small	1:2	Wood	D_{max}	1.97	8.15	1.33	2.62	1.73	Pass
HWS_11_N_OSB_RD4.5-1.5_01	Small	1:1	Wood	D_{max}	1.89	8.44	1.33	2.52	1.73	Pass
Mean of Performance Group:					1.88	8.06	1.34	2.51	2.30	Pass
Performance Group No. PG-3N (Wood, Large Building, New Design)										
MWL_21_N_OSB_RD4.5-1.5_01	Large	2:1	Wood	C_{max}	3.42	8.22	1.22	4.18	1.73	Pass
MWL_12_N_OSB_RD4.5-1.5_01	Large	1:2	Wood	C_{max}	2.77	8.87	1.15	3.19	1.73	Pass
MWL_11_N_OSB_RD4.5-1.5_01	Large	1:1	Wood	C_{max}	3.52	8.64	1.23	4.33	1.73	Pass
Mean of Performance Group:					3.23	8.58	1.20	3.90	2.30	Pass
Performance Group No. PG-4N (Wood, Small Building, New Design)										
MWS_21_N_OSB_RD4.5-1.5_01	Small	2:1	Wood	C_{max}	2.18	8.16	1.16	2.49	1.73	Pass
MWS_12_N_OSB_RD4.5-1.5_01	Small	1:2	Wood	C_{max}	2.77	8.54	1.14	3.16	1.73	Pass
MWS_11_N_OSB_RD4.5-1.5_01	Small	1:1	Wood	C_{max}	3.13	8.25	1.14	3.57	1.73	Pass
Mean of Performance Group:					2.69	8.32	1.15	3.07	2.30	Pass

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