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EXAMPLE APPLICATION OF THE FEMA P695 (ATC-63) METHODOLOGY FOR THE COLLAPSE PERFORMANCE EVALUATION OF WOOD LIGHT-FRAME SYSTEMS

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ABSTRACT

This paper describes an example application of the newly developed FEMA P695 (ATC-63) methodology for the collapse performance evaluation of wood light-frame construction, an existing lateral-force-resisting system of ASCE/SEI 7-05. The example includes all the steps required in the application of the FEMA P695 (ATC-63) methodology including: the identification and design of wood light-frame archetype configurations, the development of nonlinear numerical models for these archetype configurations, the execution of nonlinear incremental dynamic analyses of these archetype configurations using the far-field ground motions set of the FEMA P695 (ATC-63) methodology, and the collapse performance evaluation of these archetype configurations considering composite uncertainties in the ground motions, design requirements, numerical models and quality of the test data. The results of this example shows that current seismic provisions for engineered wood light-frame construction included in the ASCE/SEI 7-05 are adequate to provide an acceptable level of collapse safety. It is also recognized that the collapse safety of actual engineering wood light-frame construction is higher than calculated in this example because of the beneficial effects of interior and exterior wall finishes. These wall finishes were not included in this example because they are not defined as part of the lateral structural system, and therefore are not governed by the seismic design provisions.

Introduction

The recently developed FEMA P695 (ATC-63) methodology (FEMA 2009), referred to in this paper as the Methodology, is a procedure to establish consistent and rational building system performance and response parameters (R, C_d, Ω_0) for the linear design methods traditionally used in current building codes. The primary application of the procedure is for the evaluation of new structural systems with equivalent seismic performance. The primary design performance objective is taken to minimize the risk of structural collapse under the maximum

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considered earthquake.

In this paper, wood light-frame system design requirements of ASCE/SEI 7-05 are used as the framework for this example application of the Methodology for the collapse performance evaluation of wood light-frame construction. A set of structural archetypes are developed for wood light-frame buildings, nonlinear models are developed to simulate structural collapse, models are analyzed to predict the collapse capacities of each design, and the adjusted collapse margin ratio, ACMR, is evaluated and compared to acceptance criteria. This example considers a value of R = 6 and checks if such designs pass the acceptance criteria of Section 7.4 of the Methodology. This value is different from the current value of R = 6.5 for wood light-frame shear wall systems with wood structural panel sheathing in ASCE/SEI 7-05 (ASCE 2006). It has been rounded to the nearest whole number for simplicity, and because developmental studies have shown that there is no discernable difference in collapse performance of structures design for fractional R factors (e.g., R = 6 versus R = 6.5). The Ω_0 factor is not assumed initially, but is determined from the actual overstrength factors, Ω, calculated for the archetype designs.

Structural System Information

Design Requirements

This example utilizes design requirements for engineered wood light-frame buildings included in ASCE/SEI 7-05 (ASCE 2006), in place of the requirements that would need to be developed for a newly proposed system. For the purpose of assessing uncertainty within the Methodology, the ASCE/SEI 7-05 design requirements are categorized as “A-Superior” since they represent many years of development, include lessons learned from a number of major earthquakes, and consider recent results obtained from large research programs on wood light-frame systems, such as the FEMA-funded CUREE-Caltech Woodframe Project and the NSF/NEES-funded NEESWood Project.

Test Data

The quality of the test data is an important consideration of the Methodology when quantifying the uncertainty in the overall collapse assessment process. Cyclic test data were provided by the wood industry (Line et al. 2008) for each of the archetype configurations used in this example. In addition, more data were used by the authors to calibrate and validate the numerical model; these include monotonic and cyclic tests which cover a wide range of wood sheathing types and thicknesses (e.g. Oriented Strand Board and Plywood), framing grades, species, and connector types (e.g. common vs box nails). All loading protocols were continued to deformations large enough for the capping strength to be observed, which allows better calibration of models for structural collapse assessment. Nevertheless, some uncertainties still exist with these test data sets including a) premature failures in some of the data set caused by specimens with smaller connector edge distances than specified, b) the use of the Sequential Phased Displacement, SPD, loading protocol in tests that tends to cause premature specimen failure by connectors fatigue, which is seldom observed after real earthquakes, c) the inherent large variability associated with the material properties of wood, and d) a lack of duplicate tests of the same specimen. Therefore, this test data set is categorized as “B-Good.”

Identification of Wood Light-Frame Archetype Configurations

The archetypes are established according to the requirements of Chapter 4, and separated into Performance Groups according to Section 7.4 of the Methodology. The first step in archetype development is to establish the possible building design configurations. Two different building configurations are assumed to be representative for the purpose of defining the two-dimensional archetype configurations for wood light-frame shear wall systems with wood structural panel sheathing. The first configuration is representative of residential buildings, while the second configuration is associated with office, retail, educational, and warehouse/light-manufacturing wood buildings.

Table 1 lists the range of design parameters considered for the development of the two-dimensional archetype wall models. According to Section 5.3 of the Methodology (FEMA 2009), two-dimensional archetype wall models, not accounting for torsional effects, are considered acceptable because the intended use of the methodology is to verify the performance of a full class of buildings, rather than one specific building with a unique torsional issue. According to the requirements of Section 4.2.3 of the Methodology, nonstructural wall finishes, such as stucco and gypsum wallboard, were not considered in the modeling of the archetypes. These finishes are excluded because they are not defined as part of the lateral structural system, and therefore are not governed by the seismic design provisions. Depending on their type, wall finishes may greatly influence the seismic response of wood buildings. Note that the Methodology would allow such elements to be included in the structural model, if one defines them as part of the lateral structural system, and design provisions are included to govern their design.

Table 1. Range of Variables Considered for the Definition of Wood Light-Frame Archetype Buildings

Variable	Range
Number of stories	1 to 5
Seismic Design Categories (SDC)	D_{max} and D_{min}
Story height	10 ft
Interior and exterior nonstructural wall finishes	Not considered
Wood shear wall pier aspect ratios	High/Low

Following the guidelines of Section 4.3 of the Methodology, low aspect ratio (1:1 to 1.43:1) and high aspect ratio (2.70:1 to 3.33:1) walls were used as the two basic configurations in the archetype designs. This was done to evaluate the influence of the aspect ratio strength adjustment factor contained in ASCE/SEI 7-05, which effectively increases the required strength of high aspect ratio wood shear walls.

Table 2 shows the Performance Groups (PG) used to evaluate the wood light-frame buildings, consistent with the requirements of Section 4.3.1 of the Methodology. To represent these ranges of design parameters, 48 archetypes could have been used to evaluate the system (three designs for each of the 16 Performance Groups shown in Table 2). However, Table 2 shows that 16 archetypes were found to be sufficient. The notes in the table explain why these specific archetypes were selected, including the rationale for why these 16 can be used in place of the full set of 48. These 16 wood archetypes were divided among five of the Performance Groups: (1) three low aspect ratio wall short-period buildings designed for SDC D_{max} (PG-1); (2) five SDC D_{max} - high aspect ratio wall short-period buildings in SDC D_{max} (PG-9); (3) one low aspect ratio shear wall moderate/long-period building designed for SDC D_{min} (PG-4); and (4) seven SDC D_{min} - high aspect ratio shear wall buildings, which are divided into four short-period buildings (PG-11) and three moderate/long-period building (PG-12). It is believed that this ensemble of 16 archetypes covers the current design space for wood light-frame buildings fairly well, but additional configurations may be desirable for a complete application of the Methodology. Detailed descriptions of the 16 archetype models developed for wood light-frame buildings are given in FEMA (2009).

Table 2. Performance Groups used in the Evaluation of Wood Light-Frame Buildings

Performance Group Summary					
Group No.	Grouping Criteria				Number of Archetypes
	Basic Config.	Design Load Level		Period Domain	
		Gravity	Seismic		
PG-1	Low Wall Aspect Ratio	High (Nominal)	SDC D_{max}	Short	3
PG-2				Long	0^{1}
PG-3			SDC D_{min}	Short	0
PG-4				Long	1^{2}
PG-5		Low (NA)	SDC D_{max}	Short	0^{3}
PG-6				Long	
PG-7			SDC D_{min}	Short	
PG-8				Long	
PG-9	High Wall Aspect Ratio	High (Nominal)	SDC D_{max}	Short	5
PG-10				Long	0^{1}
PG-11			SDC D_{min}	Short	4
PG-12				Long	3
PG-13		Low (NA)	SDC D_{max}	Short	0^{3}
PG-14				Long	
PG-15			SDC D_{min}	Short	
PG-16				Long	

1. No long-period SDC D_{max} wood-frame archetypes, because representative designs never exceed T = 0.6s.

2. Only one archetype in low-aspect/SDC D_{min}/long-period Performance Group, because only one representative design exceeds T = 0.4s.

3. No archetypes because light wood-frame archetype design and performance not influenced significantly by gravity loads (i.e., nominal gravity loads used for all designs).

Table 3 reports the properties of each of these 16 archetype designs. The high- and low-seismic demands are represented by the maximum and minimum ground motions of Seismic Design Category (SDC) D, respectively. The archetypes are designed for a soil type D and acceleration parameters S_{DS} = 1.0 g and S_{D1} = 0.6 g for SDC D_{max} (High Seismic) and S_{DS} = 0.50 g and S_{D1} = 0.20 g for SDC D_{min} (Low Seismic). The Maximum Considered Earthquake, MCE, ground motion spectral response accelerations, S_{MT}, shown in Table 3, is the MCE spectral response acceleration at the fundamental period of the building per the guidelines of Section 5.2.1 of the Methodology (FEMA 2009). In accordance with Section 5.2.4, the periods reported in Table 3 are the fundamental period of the buildings based on the upper limit of Section 12.8.2 of ASCE/SEI 7-05 ($T = C_u T_a$) with a lower bound of 0.25 sec.

Table 3. Wood Light-Frame Archetype Structural Design Properties

Arch. ID	No. of Stories	Key Archetype Design Parameters						
		Building Configuration	Wall Aspect Ratio	Seismic Design Criteria				$S_{MT}(T)$ [g]
				SDC	*T* [sec]	T_1 [sec]	*V/W* [g]	
Performance Group No. PG-1 (Short Period, Low Aspect Ratio)								
1	1	Commercial	Low	D_{max}	0.25	0.40	0.167	1.50
5	2	Commercial	Low	D_{max}	0.26	0.46	0.167	1.50
9	3	Commercial	Low	D_{max}	0.36	0.58	0.167	1.50
Performance Group No. PG-9 (Short Period, High Aspect Ratio)								
2	1	1&2 Family	High	D_{max}	0.25	0.29	0.167	1.50
6	2	1&2 Family	High	D_{max}	0.26	0.37	0.167	1.50
10	3	Multi-Family	High	D_{max}	0.36	0.44	0.167	1.50
13	4	Multi-Family	High	D_{max}	0.45	0.53	0.167	1.50
15	5	Multi-Family	High	D_{max}	0.53	0.62	0.167	1.50
Partial Performance Group No. PG-4 (Long Period, Low Aspect Ratio)								
11	3	Commercial	Low	D_{min}	0.41	0.93	0.063	0.75
Performance Group No. PG-11 (Short Period, High Aspect Ratio)								
3	1	Commercial	High	D_{min}	0.25	0.50	0.063	0.75
4	1	1&2 Family	High	D_{min}	0.25	0.41	0.063	0.75
7	2	Commercial	High	D_{min}	0.30	0.61	0.063	0.75
8	2	1&2 Family	High	D_{min}	0.30	0.62	0.063	0.75
Performance Group No. PG-12 (Long Period, High Aspect Ratio)								
12	3	Multi-Family	High	D_{min}	0.41	0.69	0.063	0.75
14	4	Multi-Family	High	D_{min}	0.51	0.81	0.063	0.75
16	5	Multi-Family	High	D_{min}	0.60	0.91	0.063	0.75

Nonlinear Model Development

The computer program SAWS: Seismic Analysis of Woodframe Structures, developed within the CUREE-Caltech Woodframe Project (Folz and Filiatrault 2004a, b), was used to analyze the wood light-frame archetype models. Because this example does not involve any buildings with torsional irregularities, only a two-dimensional model is utilized by fixing the rotational degree-of-freedom in the SAWS model.

In the SAWS model, the building structure is composed of rigid horizontal diaphragms and nonlinear lateral load resisting shear wall elements and the hysteretic behavior of each wall panel is represented by an equivalent nonlinear shear spring element. The hysteretic behavior of this shear spring includes pinching, as well as stiffness and strength degradation, and is governed by 10 different physically identifiable parameters (Folz and Filiatrault 2004a, b). Table 4 shows the sheathing-to-framing connector hysteretic parameters used in the SAWS model to construct the equivalent nonlinear shear spring elements of each of the walls contained in the archetype models. The hysteretic model used for these sheathing-to-framing connectors is the same model used for the entire wall panel assemblies.

Table 4. Sheathing-to-Framing Connector Hysteretic Parameters Used to Construct Shear Elements for Wood Light-Frame Archetype Models (Folz and Filiatrault 2004a, b)

Connector Type	K_0 (lbs/in)	r_1	r_2	r_3	r_4	F_0 (lbs)	F_I (lbs)	Δ_u (in)	α	β
7/16" OSB - 8d common nails	6643	0.026	-0.039	1.0	0.008	228	32	0.51	0.7	1.2
19/32" Plywood - 10d common nails	7777	0.031	-0.056	1.1	0.007	235	39	0.49	0.6	1.2

Uncertainty due to Model Quality

For the purpose of assessing model uncertainty, according to Section 5.7 of the Methodology, the archetype designs are assumed to be well representative of the archetype design space, even though a complete assessment may include more basic structural configurations. The structural modeling approach for the wood light-frame archetypes captures the primary shear deterioration modes of the shear walls that precipitate side-sway collapse. However, not all behavioral aspects are captured by this system-level modeling, such as axial-flexural interaction effects of the wall elements, the uplift of narrow wall ends, and the slippage of sill and top plates. These effects are secondary for walls with low aspect ratios, which deform mainly in a shear mode, but are important for archetypes incorporating walls with high aspect ratios. Therefore, the structural model for the archetypes incorporating low-aspect ratio walls is rated as “B-Good”, while the same structural model for the archetypes incorporating high-aspect ratio walls is rated as “D-Poor”.

Nonlinear Structural Analyses

To compute the system overstrength, Ω, and to help verify the structural model, monotonic static pushover analysis is used with an inverted-triangular lateral load pattern; this approach differs slightly from the final requirements of Section 6.3 of the Methodology.

Following Section 6.4 of the Methodology, to compute the collapse capacity of each wood light-frame archetype design, the Incremental Dynamic Analysis (IDA) approach is used with the Far-Field record set and the ground motion scaling method specified in Section 6.2.

The intensity of the ground motion causing collapse of the wood light-frame archetype models is defined as the point on the intensity-drift IDA plot having a nearly horizontal slope but without exceeding a peak story drift of 7% in any wall of a model. Note that the resulting collapse capacities should not be highly sensitive to this arbitrary choice of 7% drift, since the IDA curves are relatively flat at such large drifts (see Fig 1a below).

Figure 1 illustrates how the IDA method is used to compute the collapse margin ratio, *CMR*, for the two-story archetype model No. 5. The spectral acceleration at collapse is computed for each of the 44 ground motions of the Far-Field Set, as shown in Fig. 1a. The collapse fragility curve can then be constructed from the IDA plots, as shown in Fig. 1b. The collapse level earthquake spectral acceleration (spectral acceleration causing collapse in 50% of the analyses) is $S_{CT}(T = 0.26 \text{ sec}) = 2.23$ g for this example. The collapse margin ratio, *CMR*, of 1.49 is then computed as the ratio of S_{CT} and the MCE spectral acceleration value at $T = 0.26$ sec, which is $S_{MT} = 1.50$ g for this building and SDC. It should be noted that a full IDA is not required to quantify *CMR*, as discussed in Section 6.4.2 of the Methodology.

a) **b)**

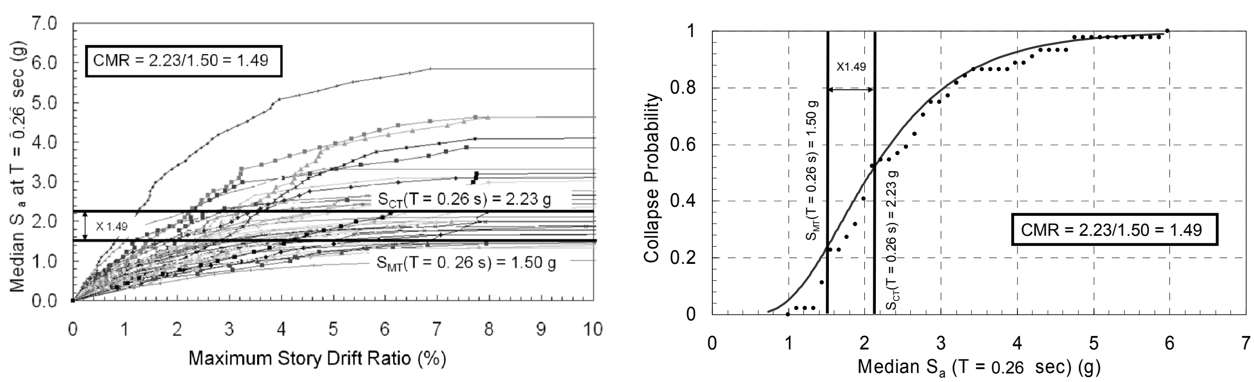

Figure 1. Illustration of IDA method for archetype model No. 5, a) Results of IDA to Collapse and b) Collapse Fragility Curve

Static pushover analyses were conducted and the IDA method was applied to each of the 16 wood light-frame archetype designs, and Table 5 summarizes the *CMR* values obtained from these analyses. These IDA results indicate that the average *CMR* is 1.43 for the SDC D_{max}, short period – low aspect ratio archetypes (PG-1), 1.90 for the SDC D_{max}, short period – high aspect ratio archetypes (PG-9), 2.64 for the SDC D_{min}, long period – low aspect ratio archetypes (partial PG-4), 2.57 for the SDC D_{min}, short period – high aspect ratio archetypes (PG-11) and 2.82 for the SDC D_{min}, long period – high aspect ratio archetypes (PG-12). These margin values, however, have not yet been adjusted for the beneficial effects of spectral shape (according to Section 7.2 of the Methodology), as discussed later. The results shown in Table 5 show that the wood light-frame archetypes designed in low-seismic regions (SDC D_{min}) have higher collapse margin ratios (lower collapse risk) compared with the archetypes designed in high-seismic regions (SDC D_{max}). Also, archetypes incorporating walls with high aspect ratios have higher collapse margin ratios than archetypes with low aspect ratio walls. This is the result of the

ASCE/SEI 7-05 strength reduction factor applied to walls with high aspect ratios, which cause an increase in required number of nails to reach a given design strength. This increased nailing density causes an increase in the shear capacity of the walls with high aspect ratios, but the model does not account for the associated increase in flexural deformations.

Performance Evaluation

The collapse margin ratios computed above do not account for the unique spectral shape of rare ground motions. According to Section 7.2 of the Methodology, spectral shape adjustment factors, *SSF*, must be applied to the *CMR* results to account for spectral shape effects. Based on Section 7.2.2, the *SSF* can be computed for each archetype based on the SDC and the archetypes' period-based ductility (μ_T), obtained from the pushover curve. The adjusted collapse margin ratio, *ACMR*, is then computed for each wood light-frame archetype as the multiple of the *SSF* (from Table 7-1b for SDC D) and *CMR* (from Table 5).

Table 5 shows the resulting *ACMR* values for the wood light-frame archetypes. To calculate acceptable values of the ACMR, the total system uncertainty is needed. Section 7.3.4 of the Methodology provides guidance for this calculation. Table 7-2 of the methodology shows these composite uncertainties, which account for the variability between ground motion records of a given intensity (defined as a constant $\beta_{\mathrm{RTR}} = 0.40$), the uncertainty in the nonlinear structural modeling, the quality of the test data used to calibrate the element models, and the quality of the structural system design requirements. For this example, the composite uncertainty was based on a "B-Good" model quality for archetypes with low aspect ratio walls and a "D-Poor" for archetypes with high aspect ratio walls, "A-Superior" quality of design requirements" and "B-Good" quality of test data. Thus, $\beta_{TOT} = 0.500$ for archetype buildings incorporating low aspect ratio walls (Table 7-2b) and $\beta_{TOT} = 0.675$ for archetype buildings incorporating high aspect ratio walls (Table 7-2d of the Methodology).

An acceptable *ACMR* must now be selected based on a composite uncertainty, β_{TOT}, and a target collapse probability. Table 7-3 of the methodology presents acceptable values of adjusted collapse margin ratio computed assuming a lognormal distribution of collapse capacity. Section 7.1.2 of the Methodology defines the collapse performance objectives as: (1) a conditional collapse probability of 20% for all individual wood light-frame archetype models, and (2) a conditional collapse probability of 10% for the average of each of the Performance Groups of wood light-frame archetypes. For archetypes incorporating low aspect ratio walls, this corresponds to an acceptable collapse margin ratio *ACMR20*% of 1.52 for every wood light-frame archetype and an *ACMR10*% of 1.90 for each Performance Group. For archetype buildings incorporating high aspect ratio walls, this corresponds to an acceptable collapse margin ratio *ACMR20*% of 1.76 for every wood light-frame archetype and an *ACMR10*% of 2.38 for each Performance Group.

Table 5 presents the final results and acceptance criteria for each of the 16 wood light-frame archetype designs. The table presents the collapse margin ratios computed directly from the collapse fragility curves, *CMR*, the period-based ductilities, μ_T, the *SSF*, and the adjusted collapse margin ratio, *ACMR*. The acceptable *ACMR* are shown and each archetype is shown to either pass or fail the acceptance criteria. Average *ACMRs* are also shown for the four complete Performance Groups of archetypes.

The results shown in Table 5 show that all archetypes pass the *ACMR20%* criteria, and the averages of each Performance Group pass the *ACMR10%* criteria. If wood light-frame buildings were a "newly proposed" seismic-force-resisting system with $R = 6$, it would meet the collapse performance objectives of the Methodology, and would be approved as a new system.

Table 5. Adjusted Collapse Margin Ratios and Acceptable Collapse Margin Ratios for Wood Light-Frame Archetype Designs

Arch. ID No.	Design Configuration			Computed Overstrength and Collapse Margin Parameters					Acceptance Check	
	No. of Stories	Building Configuration	Wall Asp. Ratio	Static Ω	*CMR*	μ_T	*SSF*	*ACMR*	Accept. *ACMR*	Pass/ Fail
Performance Group No. PG-1 (Short Period, Low Aspect Ratio)										
1	1	Commercial	Low	2.0	1.34	9.9	1.33	1.78	1.52	Pass
5	2	Commercial	Low	2.5	1.49	7.1	1.31	1.95	1.52	Pass
9	3	Commercial	Low	2.0	1.45	12.4	1.33	1.93	1.52	Pass
Mean of Performance Group:				**2.2**	**1.43**	**9.8**	**1.32**	**1.89**	**1.90**	**Pass**
Performance Group No. PG-9 (Short Period, High Aspect Ratio)										
2	1	1&2 Family	High	4.1	1.94	9.9	1.33	2.57	1.76	Pass
6	2	1&2 Family	High	3.8	2.14	9.6	1.33	2.84	1.76	Pass
10	3	Multi-Family	High	3.7	1.91	7.9	1.33	2.54	1.76	Pass
13	4	Multi-Family	High	2.9	1.73	5.8	1.28	2.21	1.76	Pass
15	5	Multi-Family	High	2.6	1.78	5.4	1.27	2.26	1.76	Pass
Mean of Performance Group:				**3.4**	**1.90**	**7.7**	**1.31**	**2.48**	**2.38**	**Pass**
Partial Performance Group No. PG-4 (Long Period, Low Aspect Ratio)										
11	3	Commercial	Low	2.1	2.64	7.0	1.13	2.98	1.52	Pass
Performance Group No. PG-11 (Short Period, High Aspect Ratio)										
3	1	Commercial	High	3.6	2.28	9.9	1.14	2.58	1.76	Pass
4	1	1&2 Family	High	5.4	2.78	9.9	1.14	3.16	1.76	Pass
7	2	Commercial	High	4.0	2.60	7.7	1.13	2.95	1.76	Pass
8	2	1&2 Family	High	3.5	2.60	7.7	1.13	2.94	1.76	Pass
Mean of Performance Group:				**4.1**	**2.57**	**8.8**	**1.13**	**2.91**	**2.38**	**Pass**
Performance Group No. PG-12 (Long Period, High Aspect Ratio)										
12	3	Multi-Family	High	4.0	3.12	7.1	1.13	3.51	1.76	Pass
14	4	Multi-Family	High	3.4	2.78	6.2	1.12	3.12	1.76	Pass
16	5	Multi-Family	High	3.3	2.56	5.7	1.13	2.90	1.76	Pass
Mean of Performance Group:				**3.6**	**2.82**	**6.3**	**1.13**	**3.18**	**2.38**	**Pass**

Calculation of Ω_0 using Set of Archetype Designs

This section determines the value of the overstrength factor, Ω_0 that would be used in the design provisions for the "newly-proposed" wood light-frame system.

Table 5 shows the calculated Ω values for each of the archetypes, with a range of values from 2.0 to 5.4. The average values for each Performance Group, are 2.2, 3.4, 4.1, and 3.6, with the largest value of 4.1 being for the high aspect ratio walls in short-period buildings designed for low-seismic demands (PG-11). According to Section 7.6 of the Methodology, the largest possible Ω_o = 3.0 is warranted, due the average values being greater than 3.0 for three of the Performance Groups.

Conclusions

This example application of the Methodology shows that current seismic provisions for engineered wood light-frame construction included in the ASCE/SEI 7-05 (with use of R = 6 rather than R = 6.5) are adequate to provide an acceptable level of collapse safety. Note that the collapse safety of actual engineering wood light-frame construction is most likely higher than calculated in this example because of the beneficial effects of interior and exterior wall finishes. These wall finishes were not included in this example because they are not currently defined as part of the lateral structural system, and therefore are not governed by the seismic design provisions.

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