Structural Engineering and Mechanics, Vol. 21, No. 1 (2005) 83-100
DOI: http://dx.doi.org/10.12989/sem.2005.21.1.083

Incorporating nonstructural finish effects and construction quality in a performance-based framework for wood shearwall design

Jun Hee Kim†

Lochsa Engineering, Inc., Las Vegas, NV, USA

David V. Rosowsky‡

Department of Civil Engineering, TAMU 3136, Texas A&M University, College Station, TX 77843-3136, USA

(Received November 26, 2004, Accepted May 23, 2005)

Abstract. This paper presents results from a study to extend a performance-based shearwall selection procedure to take into account the contributions of nonstructural finish materials (such as stucco and gypsum wallboard), construction quality issues, and their effects on the displacement performance of engineered wood shearwalls subject to seismic loading. Shearwall performance is evaluated in terms of peak displacements under seismic loading (characterized by a suite of ordinary ground motion records) considering different combinations of performance levels (drift limits) and seismic hazard. Shearwalls are analyzed using nonlinear dynamic time-history analysis with global assembly hysteretic parameters determined by fitting to actual shearwall test data. Peak displacement distributions, determined from sets of analyses using each of the ground motion records taken to characterize the seismic hazard, are post-processed into performance curves, design charts, and fragility curves which can be used for risk-based design and assessment applications.

Key words: design; earthquake engineering; probability; seismic design; shearwall; wood structures.

1. Introduction

Woodframe structures are the predominant structural form for low-rise (residential and other) buildings in North America. The structures often are permitted to be designed using prescriptive requirements specified in the applicable building codes or approved standards. Such prescriptive design generally involves selection of members, fasteners, and amount of lateral force bracing from tables containing pre-engineered (or deemed-to-comply) designs. Fully engineered design is more common for mid-rise and other large structures and for low-rise structures in regions of high seismicity, however this can still involve some member selection from tables. Fastening and lateral

† Structural Engineer, Formerly, Graduate Research Assistant, Department of Civil, Construction and Environmental Engineering, Oregon State University, Corvallis, OR, USA, E-mail: junhee_kim66@yahoo.com
‡ A.P. and Florence Wiley Chair Professor and Head, Corresponding author, E-mail: rosowsky@tamu.edu

force bracing are most often designed using methods based on engineering principles.

With the emergence of performance-based design procedures for structures built in seismic regions, there exists a need for pre-engineered prescriptive requirements which are easily interpreted and able to be used by designers as well as builders. These also could be used by engineers to provide a check of engineered designs. Wood shearwalls provide primary lateral force resisting systems in light-frame wood construction. The shearwalls typically are constructed using only a few different member sizes, framing arrangements, sheathing materials/thicknesses, and fastener types. Thus, the process of shearwall design becomes one of selecting an appropriate combination of materials, framing arrangements, and fastener schedules. A prescriptive design aid might therefore simply specify the combination of design parameters needed to meet a particular performance requirement.

The concept of performance-based design has gained interest among designers and researchers in recent years. Performance-based design applied to building systems includes selection of appropriate structural systems and configurations to ensure that the structure has adequate strength, stiffness, and energy dissipation capacity to respond to loadings, including those arising from natural hazards, without exceeding permissible damage states (AISC 2001). Although performance-based design has advanced for some materials and structural types, such as steel and reinforced concrete buildings and bridges built in high seismic regions, the application to light-frame wood structures has only recently been explored.

This paper describes a procedure for evaluating the contributions of nonstructural finish (NSF) materials such as stucco and gypsum wallboard to the seismic performance of woodframe shearwalls and incorporating that information into a performance-based shearwall selection procedure. The paper also examines selected construction quality issues and describes a possible procedure for including their effects in a performance-based selection procedure for woodframe shearwalls. Shearwalls are assumed to be constructed using typical light-frame materials, framing techniques, fasteners, and anchorage. The walls are assumed to be engineered and had full overturning restraint. These shearwalls are treated as isolated subassemblies and are not assumed to be acting as part of a structural system. A suite of 20 ordinary ground motion records is taken to characterize the (non-near fault) seismic hazard in southern California. Hysteretic parameters of shearwalls built with and without nonstructural finish (NSF) materials were fit directly to hysteretic curves obtained from actual cyclic shearwall tests. These parameters were then used as input to a nonlinear dynamic time-history analysis to predict shearwall response. For the study of shearwall behavior considering different construction quality levels, modification factors were developed to adjust the hysteretic parameters obtained from the actual shearwall tests (which were assumed to correspond to the highest quality level). These modified hysteretic parameters (i.e., global hysteretic parameters obtained from experimental test × modification factor) were then used as input to the nonlinear dynamic time history analyses to predict displacement response of shearwalls considering different construction quality levels.

2. Analysis

The hysteretic response of a typical shearwall exhibits the same defining characteristics (pinched behavior, strength and stiffness degradation, etc.) as those of the individual sheathing-to-framing connector under cyclic loading (Dolan and Madsen 1992). Consequently, the hysteretic model in the

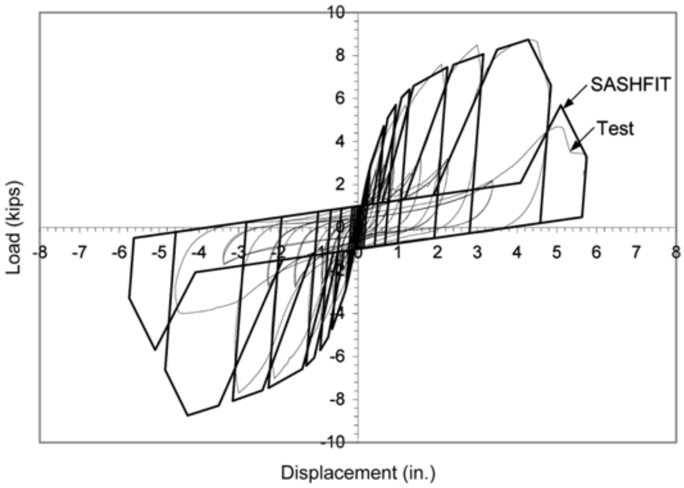

Fig. 1 Load-displacement curve using parameters determined by SASHFIT [Note: 1 in. = 25.4 mm, 1 kip = 4.448 kN] *Only the positive quadrant is considered when extracting hysteretic parameters (see: Fonseca *et al.* 2002 for more information)

numerical model (CASHEW), which is used to represent the hysteretic behavior of sheathing-to-framing connectors (Folz and Filiatrault 2001), can also be used to represent the global hysteretic response of a shearwall under cyclic loading, with appropriate model parameter values. The program SASHFIT (also developed as part of the study by the authors) can be used to extract the ten hysteretic parameters for the shearwall directly from full-scale cyclic shearwall test data. Fig. 1 shows a comparison of one example of the hysteretic model with parameters determined by SASHFIT with the original cyclic load-displacement curve from an actual shearwall test. Since fastener tear-through at the panel edge is the most common failure mode for the shearwall, the positive quadrant of the load-displacement curve is used to extract the hysteretic parameters (Fonseca *et al*. 2002).

The hysteretic parameters, defined in Fig. 2, include F_O = force intercept of the asymptotic line, F_I = zero-displacement load intercept, F_U = ultimate load, K_O = initial stiffness, r_1K_O = asymptotic stiffness under monotonic load, r_2K_O = post ultimate strength stiffness under monotonic load, r_3K_O = unloading stiffness, r_4K_O = re-loading pinched stiffness, degrading stiffness $K_P = K_O(\delta_O/\delta_{\max})^{\alpha}$, $\delta_{\max} = \beta\delta_{un}$, α and β = hysteretic parameters for stiffness degradation, and δ_{un} = final unloading displacement. Additional information on the hysteretic model and parameters may be found elsewhere (Folz and Filiatrault 2000). These parameters were used as input to the nonlinear dynamic time history analysis to predict the maximum shearwall response under actual earthquake ground motions. Specifically, the response quantity of interest was peak displacement (or “drift”) at the top of the shearwall. A suite of 20 ordinary ground motion (OGM) records, assumed to characterize the seismic hazard in California (Krawinkler *et al*. 2000), was used. The mass and damping ratio (ζ) as a percentage of critical, along with a ground motion record (scaled as appropriate), also are required as input in the nonlinear dynamic time-history analysis. The resulting response quantities (peak displacements), one for each scaled ordinary ground motion record, are used to construct the peak displacement distribution.

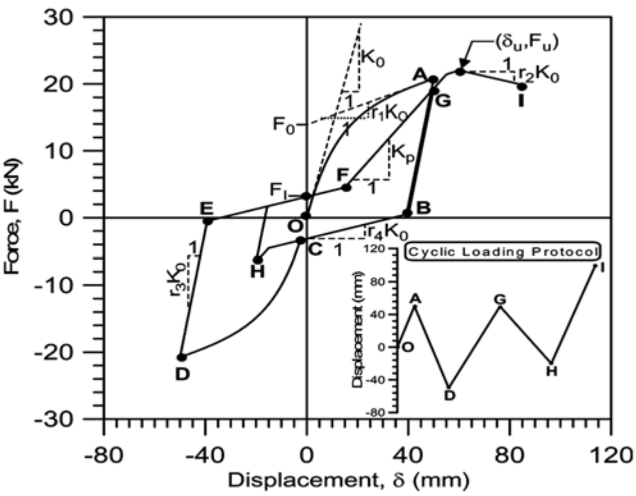

Fig. 2 Force-displacement response of a wood shearwall under cyclic loading. Hysteretic model is fit to test data for an 8 ft. × 8 ft. (2.44 m × 2.44 m) shearwall with $^3/_8$ in. (9.5 mm) thick OSB sheathing panels (from: Durham 1998). [Note: 1 in. = 25.4 mm, 1 kip = 4.448 kN]

One typical shearwall type was considered in this study, a solid 8 ft. × 8 ft. (2.44 m × 2.44 m) wall with two sheathing panels oriented vertically. This wall, designated baseline wall BW1, is shown in Fig. 3. The wall is sheathed with $^3/_8$ in (9.5 mm) OSB, and 0.113 in. (2.9 mm) diameter pneumatically-driven nails are used to attach the sheathing to the framing. Fasteners are spaced at 4 in./12 in. (100 mm/300 mm). Nonstructural finish materials (where used) included ½ in (12.7 mm)

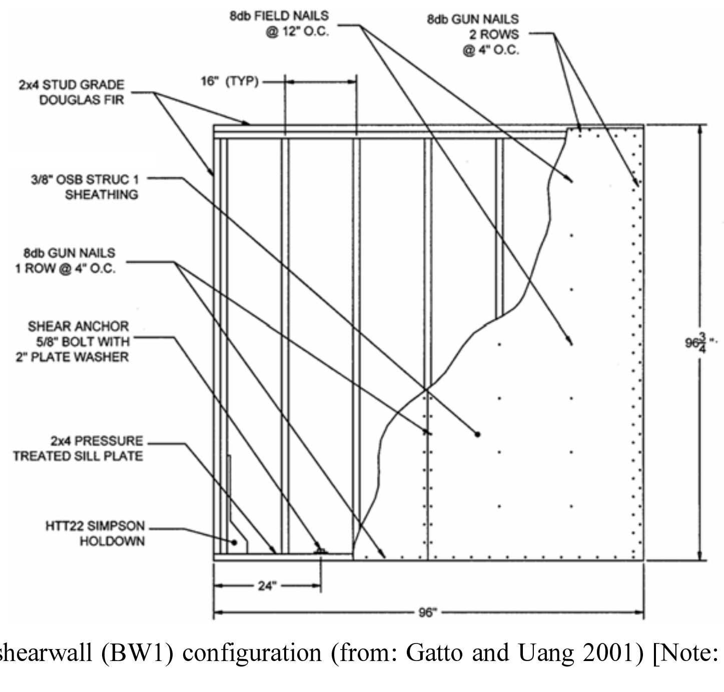

Fig. 3 Baseline shearwall (BW1) configuration (from: Gatto and Uang 2001) [Note: 1 in. = 25.4 mm]

thick gypsum wallboard and $^{7}/_{8}$ in (22.23 mm) thick exterior stucco. Further information about the shearwalls built with nonstructural finish materials may be found elsewhere (Gatto and Uang 2001).

The ground motions considered in this research were obtained from Task 1.3.2 of the CUREE-Caltech Woodframe Project. The suite of 20 ordinary ground motion (OGM) records are assumed to be representative of California seismic hazard conditions and formed the basis for the development of the CUREE-Caltech loading protocol (Krawinkler *et al.* 2000, Filiatrault and Folz 2001). Two limit states, life safety (LS) and immediate occupancy (IO) are considered in this study. The life safety limit state is paired with the 10% probability of exceedence in 50 years (10/50) hazard level, while the immediate occupancy limit state is paired with the 50% probability of exceedence in 50 years (50/50) hazard level (FEMA 2000a,b). For the life safety (LS, 10/50) limit state analyses, each record was scaled such that its mean 5% damped spectral value between periods of about 0.12 and 0.58 seconds matched the UBC design spectral value of 1.1 g for the same period range (ICBO 1997). The fundamental periods of most wood structures fall in this range. For the immediate occupancy (IO, 50/50) limit state, the records were scaled according to the procedure recommended in the NEHRP Guidelines (FEMA 2000a,b). Seismic zone 4 and soil type D were assumed for most cases in this study.

The largest contributor to the variability in peak shearwall response arises from the ground motions themselves, i.e., the suite of OGM records chosen to characterize the seismic hazard (Rosowsky and Kim 2002). All other structural and material parameters can be treated as deterministic and the peak displacements (one for each ground motion, scaled as appropriate for the limit state or hazard level of interest) can be presented in the form of a sample cumulative distribution function (CDF). These distribution functions provide a convenient method for estimating probabilities of exceedence or “non-performance”. In this study, the FEMA 356 (FEMA 2000a) drift limits were used to define the prescribed performance levels, e.g., 1% transient drift for the 50/50 hazard level, 2% transient drift for the 10/50 hazard level, and 3% transient drift for the 2/50 hazard level. Once the peak displacement distributions are determined, they can be post-

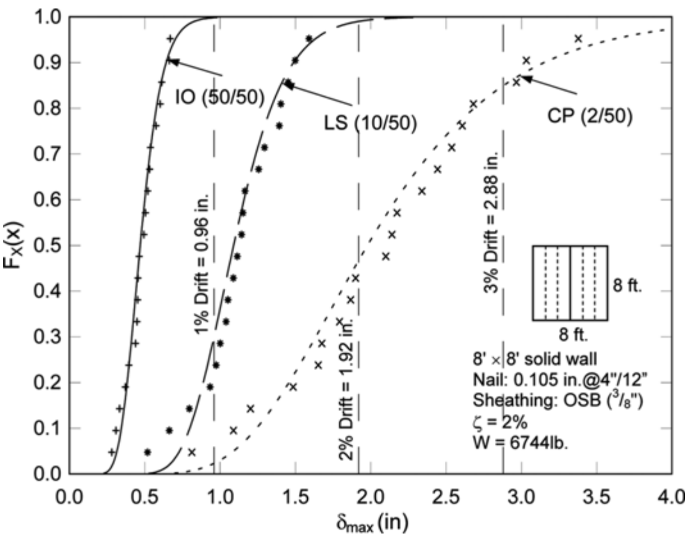

Fig. 4 Fitting a lognormal distribution to the sample CDF of peak displacements [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

processed into a form useful for design and/or assessment for given target failure probabilities. More information is provided elsewhere (Rosowsky and Kim 2002, Rosowsky 2002).

Fig. 4 illustrates the construction of sample cumulative distribution functions (CDF's) for the peak transient displacements of a particular wall considering the immediate occupancy (IO, 50/50) and the life safety (LS, 10/50) limit states. Also shown on this figure are the results for the collapse prevention (CP, 2/50) limit state. Each point represents the peak drift obtained from a nonlinear dynamic time-history analysis for a particular ground motion record, which has been appropriately scaled. The results are then rank-ordered to construct the sample CDF, which is then fit using a lognormal distribution. In addition to providing a good fit, the lognormal distribution is the most convenient distribution form for a fragility analysis as well as consideration of model uncertainty (Rosowsky and Kim 2002).

3. Results

3.1 Contribution of nonstructural finish materials

Nonstructural finish materials such as gypsum wallboard and stucco have been shown to contribute (in some cases significantly) to the lateral resistance and seismic performance of woodframe structures. Typical construction throughout North America utilizes gypsum wallboard as an interior finish material. In many parts of the west coast of the U.S., stucco is used as an exterior finish material. In most cases, the contributions of these nonstructural finish materials are ignored in the design process. However, recent experimental tests have shown that the presence of stucco and gypsum wallboard serves to increase peak strength and initial stiffness, while decreasing deformation capacity, of wood shearwalls compared to bare walls (Gatto and Uang 2001, Pardoen *et al.* 2002). Stucco applied to the sheathing panels also appears to restrain sheathing nail withdrawal and partially restrain nail head rotation (Cobeen *et al.* 2004). Thus, the inclusion of finish materials in the model can be both positive (e.g., in terms or drift reduction) and negative in that it could potentially attract higher forces, reduce deformation capacity, and result in very brittle failure modes. These potentially significant effects suggest that the contributions of these finish materials should be addressed in developing performance-based design guidelines. In this study, this was done using a visual best-fit program (called SASHFIT) to capture the global shearwall hysteretic parameters from the results of cyclic tests of shearwalls built with and without finish materials. Specifically results from cyclic tests of and 8 ft. × 8 ft. solid shearwalls built with nonstructural finish materials were used to develop the hysteretic wall parameters. The walls were tested as part of the CUREE-Caltech Woodframe Project (Gatto and Uang 2001). Only two tests of each shearwall configuration were performed and only the worst-case test results were selected. These hysteretic parameters were then used as input to nonlinear dynamic time-history analyses to develop peak displacement distributions, which were then post-processed into performance curves, design charts, and fragility curves as described next.

Peak displacement curves (developed by rank-ordering the maximum shearwall displacements for each scaled earthquake record as described previously) showing the effects of nonstructural finish (NSF) materials are shown in Fig. 5 for an assumed seismic weight of 6744 lb. (30 kN). The NSF materials can be seen to greatly enhance the performance of the shearwall. In particular, the presence of stucco serves to greatly reduce peak shearwall displacement.

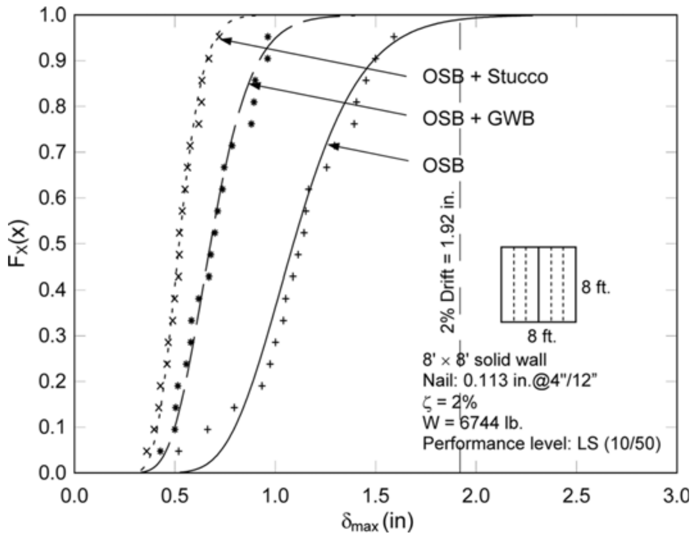

Fig. 5 Peak displacement distribution considering contribution of nonstructural finish materials [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

The information presented in the peak displacement distributions can be post-processed into a more useful form for engineering design/assessment using seismic weight as the dependent variable (design parameter). This is similar to the approach taken by Deam (1997, 2000) in which walls were rated using sustainable seismic mass as the primary design parameter. Performance curves can be constructed, using the lognormal parameters for the appropriate peak displacement distributions, for increasing values of seismic weight. Each performance curve therefore corresponds to a particular limit state (e.g., LS, 10/50) and non-exceedence probability (e.g., 84%, 90%, 95%, 99%). Design charts can then be constructed using the information in the performance curves. These charts allow selection of particular combinations of sheathing materials and fastener type/schedule for a given seismic weight and a particular performance level (non-exceedence probability). The quantities shown on the axes in both cases (performance curves and design charts) are peak displacement and seismic weight.

Performance curves for the baseline wall (BW1) built with $^3/_8$-in. (9.525 mm) OSB + stucco are shown in Fig. 6. This figure shows the 99%, 95%, 90% and 84% non-exceedence curves for the life safety (LS, 10/50) limit state. Also shown is the FEMA 356 drift limit of 2% for life safety. Considering Fig. 6, this particular shearwall built with OSB + stucco can sustain about 1520 lbs/ft (22.18 kN/m) with 95% confidence.

Design charts are constructed using the information in the performance curves. Specifically, one design chart (i.e., set of selection curves) is developed a particular performance limit state and 95^{th}-percentile value (non-exceedence probability). Using the performance curves in Fig. 6, the design chart shown in Fig. 7 can be constructed for the baseline wall BWl (8 ft. × 8 ft. solid wall). This type of figure can be used to estimate the capacity (maximum seismic weight able to be sustained) of a shearwall built with different sheathing material combinations ensuring that it will perform within the specified drift limit (e.g., 2%) with a certain confidence (e.g., 95%).

While seismic weight is used as a design parameter to construct performance curves and design charts, the value of spectral acceleration (used to scale the ground motion records with a response

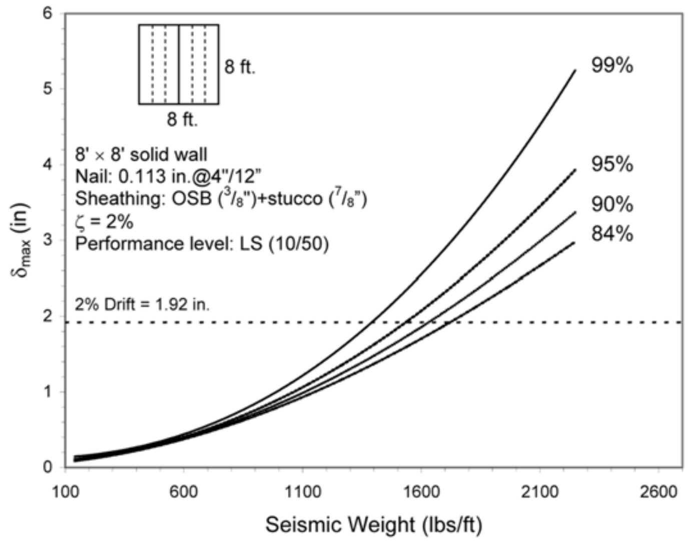

Fig. 6 Performance curves for wall built with OSB + stucco [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

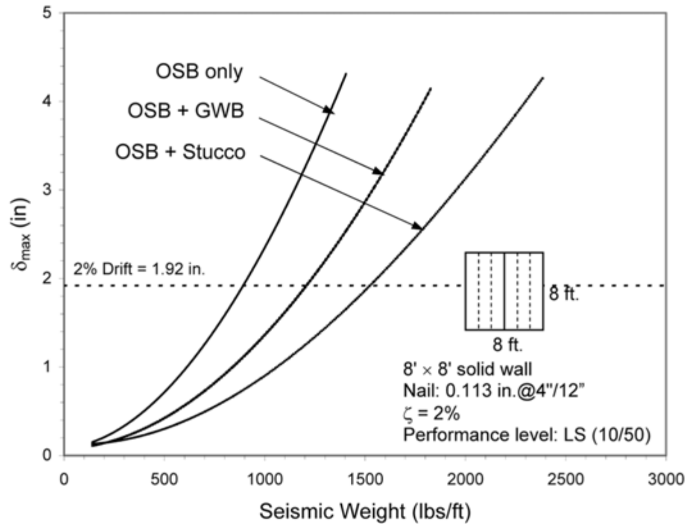

Fig. 7 95th-percentile design charts considering nonstructural finish material effects [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

spectrum procedure) is used to develop fragility curves. Fragility curves can be defined as the failure probability, conditioned on a given value of spectral acceleration, S_a. Their application to woodframe structures is described in (Ellingwood *et al.* 2004, Rosowsky and Ellingwood 2002). By varying the spectral acceleration used to scale the suite of 20 earthquake records, a peak displacement CDF can be developed for each level of scaling. The probability of failure can then be determined parametrically by first fitting the peak displacements using a specific distribution (e.g., lognormal) and considering specified drift limits such as those provided by the NEHRP guidelines

(FEMA 2000a,b). This method is appropriate when the number of ground motion records is small. Alternatively, the probability of failure can be determined non-parametrically as the relative frequency of the peak displacement exceeding the specified drift limits. This has the advantage of not requiring that a particular distribution be fit to the peak displacements. Therefore, this method is suitable when the number of ground motion records is large. Using either approach, since this probability of failure is conditioned on a given value of spectral acceleration, it becomes one point on the fragility curve. For more information on the specific procedure used to scale the ground motion records (response spectrum approach), see (Kim 2001, 2003).

Ground motion records were scaled to increasing spectral acceleration levels using a response spectrum approach. Records were scaled so that the average 5% damped spectral value over a period range typical for woodframe structures (about 0.1-0.6 secs) matched the design response spectrum constructed using the procedures in FEMA 356[1]. This period range generally corresponds to the plateau region of the design response spectrum. The target spectral acceleration was increased by intervals of 0.1 g and the records were re-scaled at each increment. This allowed for an accurate estimation of the shearwall failure probability at given levels of target spectral acceleration. This target spectral acceleration was increased up to 3.0 g in order to construct the fragility curves.

Fig. 8 shows the 1%, 2% and 3% peak drift fragility curves for the baseline shearwall built with OSB + gypsum wallboard with an assumed seismic weight of 11240 lb. (50 kN). Fig. 9 presents fragility curves for different sheathing material combinations (OSB only, OSB + GWB, and OSB + stucco) considering the 2% drift limit. In the plateau region of the response spectrum constructed using the NEHRP guidelines (considering seismic zone IV and soil profile type S_D, typical of the

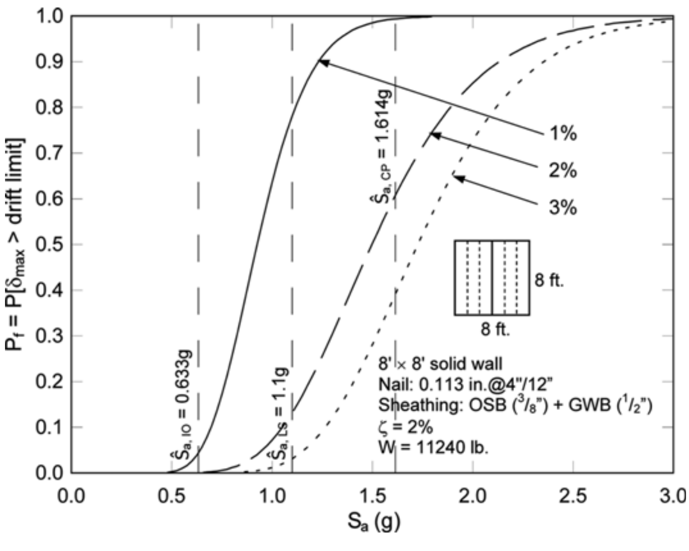

Fig. 8 Fragility curves for wall with OSB + GWB [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

[1]For the life safety (LS, 10/50) limit state in particular, each record was scaled such that its mean 5% damped spectral value between periods of 0.12 and 0.58 seconds matched the NEHRP design spectral value of 1.1 g for the same period range. For all other levels, design response spectra were constructed using the procedure recommended in the NEHRP Guidelines and the each record was scaled to match the target spectral acceleration over the plateau region of the response spectrum (FEMA 2000a,b).

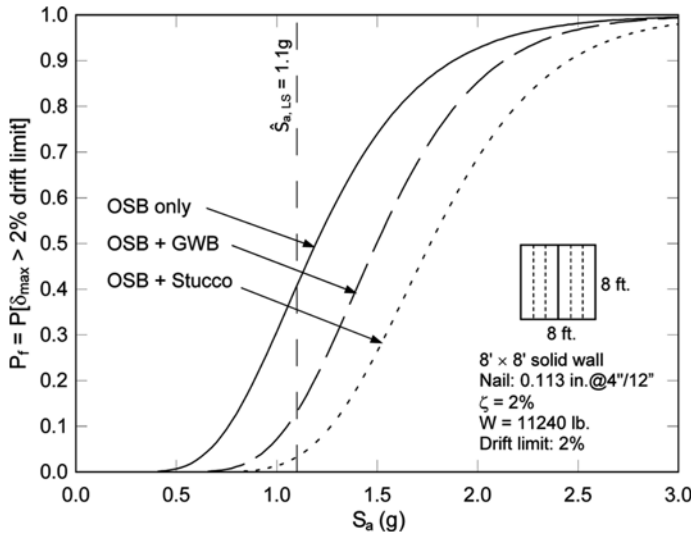

Fig. 9 Fragility curves considering nonstructural finish material effects, drift limit = 2% [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

region considered in this study), S_a = 1.1 g for life safety (LS, 10/50). In this example, if the seismic demand variable (S_a) is 1.1 g and the drift limit is 2%, the probability of failure for the shearwall built with OSB + stucco is about 3%. However, the failure probability for the same shearwall built with OSB only is much larger, about 40%.

3.2 Effects of construction quality

There are a number of construction quality issues which could significantly influence overall shearwall behavior. Among these are missing or misplaced fasteners and hold-downs, deterioration of structural and nonstructural finish materials, improper selection of fasteners, under-driven or over-driven fasteners, missing blocking, the use of smaller panel segments, cutouts in framing members (e.g., for installation of conduit), and so forth. Wood structures may deteriorate with time. In addition to natural aging, walls in woodframe structures may be subject to severe environmental conditions such as moisture absorption and fungus attack. A number of such durability issues could significantly impact the dynamic behavior of fasteners and woodframe assemblies (Kim *et al.* 2004).

As a preliminary investigation into the effect of construction tolerances (errors) on wood shearwall performance, the effect of missing fasteners was investigated by Rosowsky and Kim (2002) using the CASHEW modeling procedure (Folz and Filiatrault 2000, 2001). Using one of the baseline walls, the effect of missing fasteners or fastener lines in several locations was investigated. Selected results are shown in Fig. 10 for the life safety (LS, 10/50) limit state. Notice that while the nailing along the sole plate has a significant effect on performance, the fact that overturning anchors (hold-downs) are present reduces the effect of missing nails since the sole plate nails only resist wall racking forces. While certainly not a comprehensive investigation, this study provided some indication of the relative importance of ensuring the design fastener schedule and ensuring proper hold-down installation.

It is generally understood that a well-applied (i.e., uncracked and undamaged by moisture and

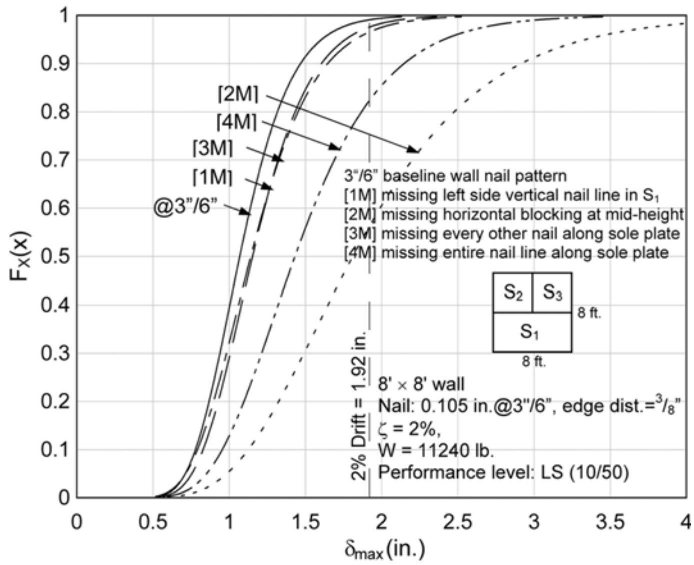

Fig. 10 Effect of missing fasteners on peak displacement (LS, 10/50) [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

other environmental attacks) stucco layer can significantly alter the performance (stiffness, displacement, energy dissipation) of a woodframe shearwall. Less clear, however, is the degree to which quality control during the application of the stucco, time in-service, or cracking in the finish

Table 1 Construction quality classifications and definitions (from: Isoda *et al*. 2002)

Superior Quality	Typical Quality	Poor Quality
Good nailing of diaphragms. 100% of stiffness and strength from high-quality laboratory tests.	Good nailing of diaphragms. 90% of stiffness and strength from high-quality laboratory tests.	Poor nailing of diaphragms. 80% of stiffness and strength from high-quality laboratory tests.
Good nailing of shearwalls. 100% of stiffness and strength from high-quality laboratory tests.	Average nailing of shearwalls. 5% greater of nail spacing.	Poor nailing of shearwalls. 20% greater of nail spacing. 5% reduction stiffness and strength due to water damage.
Good connections between structural elements. 100% of stiffness and strength from high-quality laboratory tests.	Typical connections between structural elements. 10% reduction of stiffness and strength in shearwalls from high-quality laboratory tests.	Poor connections between structural elements. 20% reduction of stiffness and strength in shearwalls from high-quality laboratory tests.
Good quality stucco. 100% of stiffness and strength from high-quality laboratory tests.	Average quality stucco. 90% of stiffness and strength from high-quality laboratory tests.	Poor quality stucco. 70% of stiffness and strength from high-quality laboratory tests.
Superior nailing of interior gypsum wallboard. 100% of stiffness and strength from high-quality laboratory tests.	Good nailing of interior gypsum wallboard. 85% of stiffness and strength from high-quality laboratory tests.	Poor nailing of interior gypsum wallboard. 75% of stiffness and strength from high-quality laboratory tests.

Table 2 Developed deterministic modification factor for construction quality (Note: NSF stands for "non structural finish")

Quality	Sheathing	K_0	r_1	r_2	r_3	F_o	F_u	F_I	α	β
TYP.	OSB	0.86	0.99	0.99	1.00	0.85	0.85	0.85	1.00	1.00
	OSB + NSF	0.87	0.99	0.99	1.00	0.86	0.86	0.87	1.00	1.00
POOR	OSB	0.61	0.99	0.97	1.01	0.63	0.63	0.61	1.00	1.00
	OSB + NSF	0.66	1.00	0.98	1.00	0.67	0.66	0.66	1.00	1.00
	OSB + NSF (GWB)	0.69	1.00	0.98	1.00	0.69	0.69	0.69	1.00	1.00

(1) r_4 is assumed to be 1.00 for all cases.

itself caused by moisture, exposure, or low-amplitude cycling may influence this benefit. Isoda *et al.* (2002) attempted to quantify the issue of construction quality by considering four index buildings and selected full-scale wall test data from two large-scale testing programs (Filiatrault *et al.* 2000, SEAOSC 2001). The walls in the four woodframe buildings included nonstructural finish materials such as gypsum wallboard and stucco. Three categories of construction quality (superior, typical, and poor) and the corresponding definitions are shown in Table 1. Through analysis, Isoda *et al.* (2002) provided sets of hysteretic parameters for the shearwalls in the four index buildings for each level of construction quality. Using this information, and selected results from Task 1.3.1 of the CUREE-Caltech Woodframe Project (Gatto and Uang 2001), Kim (2003) developed modification factors for use with selected baseline wall configurations for each of the three construction quality levels described in Table 1. These modification factors are shown in Table 2. These factors modify the original global hysteretic parameters (assumed to correspond to walls of superior quality) to obtain the global hysteretic parameters for shearwalls of typical and poor quality construction.

The peak displacement curves were constructed using the modified global hysteretic parameters

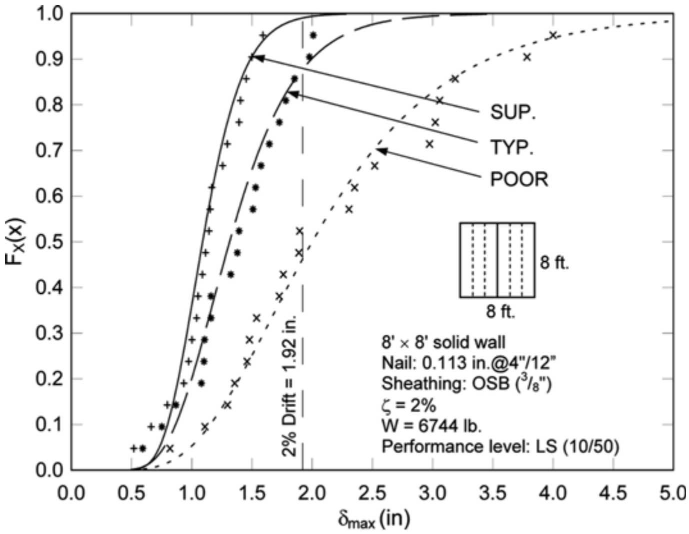

Fig. 11 Peak displacement distribution considering effect of construction quality [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

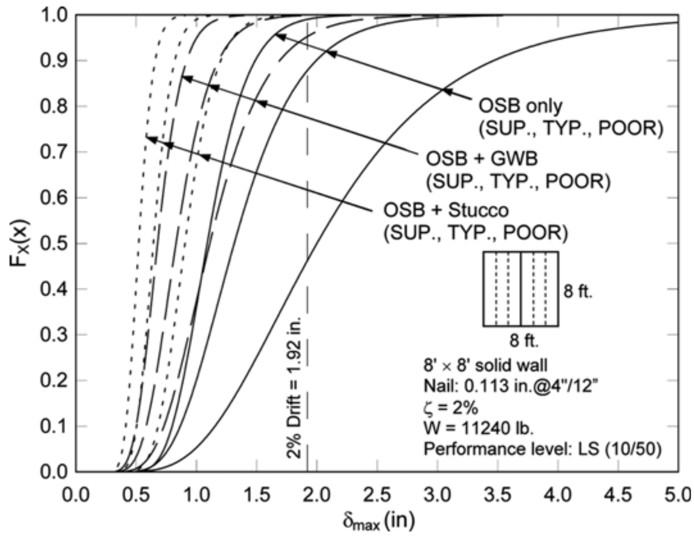

Fig. 12 Peak displacement distribution considering effect of construction quality (OSB + NSF) [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

(i.e., global hysteretic parameters obtained from experimental test × modification factor) and the CUREE-Caltech Woodframe Project Task 1.3.1 shearwall test results to investigate the effects of construction quality. A range of seismic weights was considered and peak displacement distributions were constructed for shearwalls having different sheathing combinations (i.e., OSB only, OSB + GWB, and OSB + stucco). These results are used to develop performance curves and design charts (discussed later) considering different construction quality levels. Fig. 11 illustrates the effect of construction quality on peak shearwall displacement for a solid 8 ft. × 8ft. (2.44 m × 2.44 m) wall with OSB only, for the LS (10/50) limit state. The seismic weights in this figure and Fig. 12 were selected such that the majority of peak displacements of the poor quality shearwalls were below a practical drift limit of four inches. Fig. 12 shows the peak displacement distribution for different combinations of sheathing materials and different construction quality levels. The results in this figure suggest that NSF materials significantly improve the shearwall performance at all quality levels.

The previous comparison of peak displacement distributions for the three different construction qualities revealed significant differences between walls built with different construction quality levels. Performance curves and design charts can be constructed (as described earlier) for the walls with and without nonstructural finish materials and considering different levels of construction quality. Fig. 13 presents the performance curves for baseline wall BW1 considering the three different levels of construction quality defined in Table 1. This figure shows the 95% and 84% non-exceedence curves, for the life safety (LS, 10/50) limit state. Also shown is the corresponding FEMA 356 drift limit of 2%. Assuming a target peak drift non-exceedence probability of 95% and a drift limit of 2% for LS (10/50), the wall built with superior quality and having parameters shown in Fig. 13 can sustain about 900 lbs/ft (13.13 kN/m), the wall built with typical quality can sustain about 760 lbs/ft (11.09 kN/m), and the wall built with poor quality can sustain about 560 lbs/ft (8.17 kN/m).

Fig. 14 presents the design chart (developed using the 95th-percentile performance curves, see

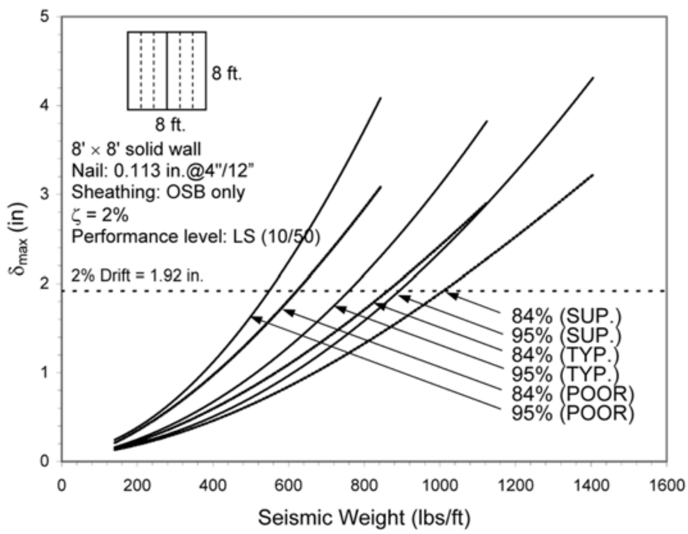

Fig. 13 Performance curves considering construction quality (OSB only) [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

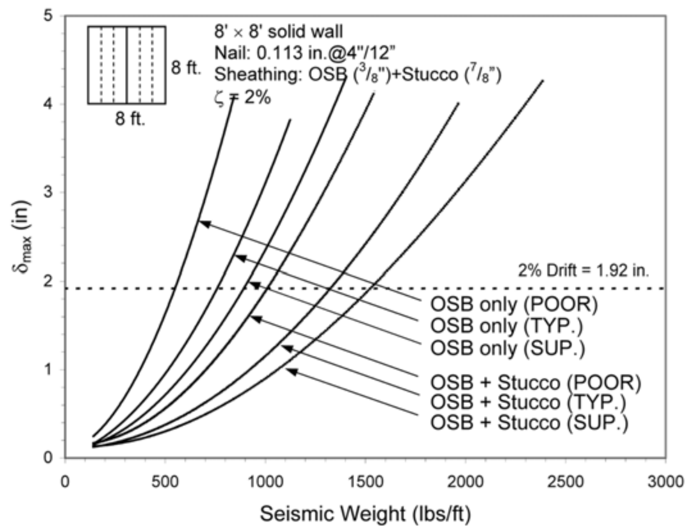

Fig. 14 95th-percentile design charts considering construction quality [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

Fig. 13) for a wall with and without consideration of nonstructural finish materials and considering the three different construction quality levels. This figure illustrates how construction quality issues can influence shearwall performance and hence the selection of wall parameters to meet specific performance requirements. Note that model uncertainty and sheathing-to-framing hysteretic parameter variability (as treated elsewhere by the authors (Rosowsky and Kim 2002)) are not explicitly considered in the development of the performance curves and design charts in this paper since the shearwall global hysteretic parameters were obtained directly from full-scale shearwall tests.

Using the procedure described previously, fragility curves for shearwalls considering different construction quality levels were developed. Fig. 15 shows the fragility curves for the baseline shearwall built with OSB + stucco, considering three performance levels (IO, LS, and CP) and the corresponding drift limits (1%, 2%, and 3%). Using life safety (LS) as an example, if the seismic demand variable S_a = 1.1 g and the drift limit is 2%, the probability of failure for the shearwall built with superior quality is about 3%. However, the probabilities of failure for the walls built with lower quality levels are notably higher, about 10% (typical) and 35% (poor), respectively.

Fig. 16 presents the fragility curves for life safety (LS) for the baseline wall with and without

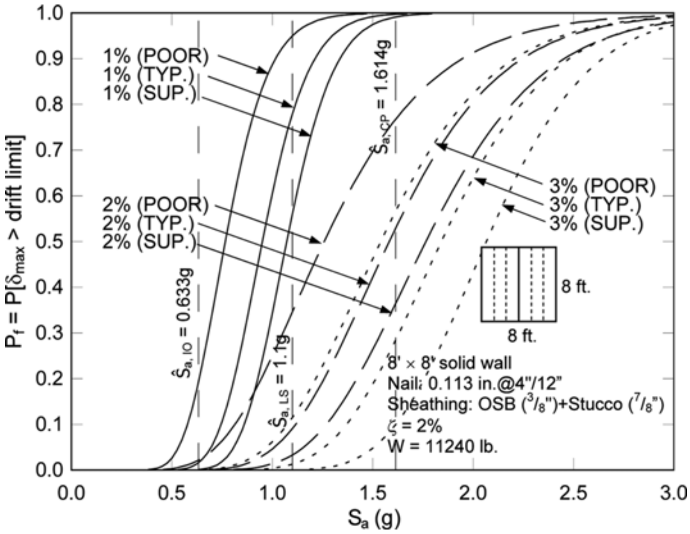

Fig. 15 Fragility curve considering construction quality (OSB + Stucco) [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

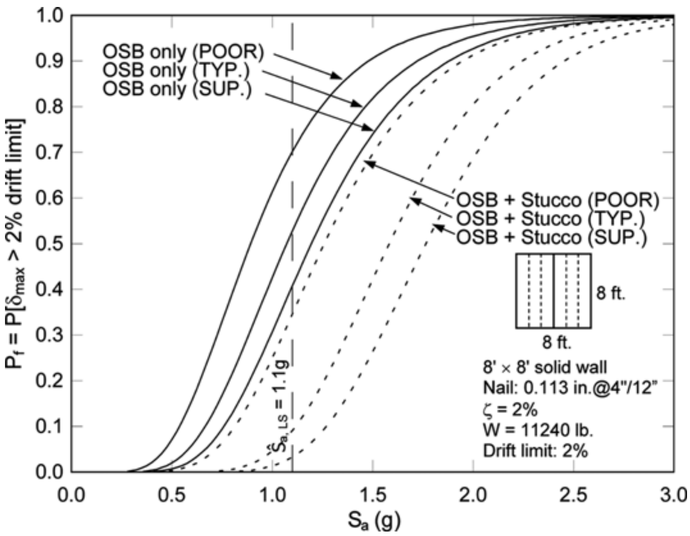

Fig. 16 Fragility curve considering construction quality (drift limit = 2%) [Note: 1 in. = 25.4 mm, 1 ft. = 0.305 m, 1 lb. = 4.448 N]

nonstructural finish materials and considering different construction quality levels. The LS seismic demand level (S_a = 1.1 g) is also shown. As expected, the shearwall failure probability increases as the construction quality level decreases for both sheathing combinations (OSB only, OSB + stucco). Also, the application of stucco to shearwall serves to significantly reduce the failure probability. For example, the failure probability of a shearwall built with stucco (even assuming poor construction quality) is lower than the failure probability of the same shearwall built with OSB only and assuming superior construction quality.

4. Conclusions

This paper presented one approach for taking into account the contributions of nonstructural finish materials and the effects of construction quality in a performance-based seismic design procedure for woodframe shearwalls. Shearwalls were assumed to be constructed using wood materials, framing techniques, fasteners, and anchorage typical of North American light-frame construction. The finish materials considered in this study were likely toward the lower bound of stiffness since continuity at wall boundaries and around the corners of the structure were not considered. Further, the individual wall approach taken in this study likely overestimates drift (since structural system effects are not taken into account) and thus is expected to be conservative. Using a suite of ordinary ground motion records to characterize the (non-near fault) seismic hazard in southern California, nonlinear dynamic time history analyses were performed to predict the behavior (peak displacement) of shearwalls under seismic loadings. Procedures were described for developing performance curves and design charts which can be used to determine limits on the seismic weight to ensure target non-exceedence probabilities for different performance levels (drift limits). Examples of performance curves and design charts were presented for shearwalls built with and without nonstructural finish materials and considering different levels of construction quality. Several examples of displacement-based fragility curves also were presented.

The contributions of nonstructural finish materials (gypsum wallboard and stucco) to the seismic performance of woodframe shearwalls may be significant, and therefore should be considered when developing performance-based design guidelines. In particular, the application of stucco serves to greatly reduce peak shearwall displacements. Construction quality (tolerance) issues such as missing or misplaced fasteners and quality of nonstructural finish material application also can significantly influence shearwall performance under earthquake loading. While it is recognized that the presence of nonstructural finish materials can significantly improve shearwall performance, and evidence of this has been documented in a number of recent studies, the degree to which this benefit can be (1) counted upon for the design life of the structure, and (2) quantified for design purposes, remains a topic for further study. Fragility curves such as those developed in this paper can be used to help guide the selection of materials and sub-assemblies (i.e., prescriptive design) to meet specific performance requirements when coupled with specified hazard levels.

Acknowledgements

Support for this project was provided by the CUREE-Caltech Woodframe Project (Task 1.5.3 – Reliability Studies) and the U.S. National Science Foundation (Grant No. CMS-0049038). This support is gratefully acknowledged. The findings reported in this paper represent the views of the writers and do not necessarily reflect the opinions of the sponsoring agencies.

References

AISC (2001), "Description of performance-based design procedures and related issues", White Paper Distributed at AISC Workshop on Performance-Based Design of Steel Buildings for Seismic Loads, June 2001.

Cobeen, K.E., Russell, J. and Dolan, D. (2004), "Recommendations for earthquake resistance in the design and construction of woodframe buildings, Part I and II - Code, standards and guidelines design and analysis procedures and construction practice", CUREE Publication No. W-30(a and b), Consortium of Universities for Research in Earthquake Engineering, Richmond, CA.

Deam, B.L. (1997), "Seismic ratings for residential timber buildings", BRANZ Study Report 73, Judgeford, New Zealand.

Deam, B.L. (2000), "Rating seismic bracing elements for timber buildings", *12th World Conf. on Earthquake Engineering*, Auckland, New Zealand.

Dolan, J.D. and Madsen, B. (1992), "Monotonic and cyclic tests of timber shear walls", *Canadian J. Civil Eng.*, **19**(3), 115-122.

Durham, J.P. (1998), "Seismic response of wood shearwalls with oversized oriented strand board panels", MASc Thesis, University of British Columbia, Vancouver, Canada.

Ellingwood, B.R., Rosowsky, D.V., Li, Y. and Kim, J.H. (2004), "Fragility assessment of light - frame wood construction subjected to wind and earthquake hazards", *J. Struct. Eng.*, ASCE, **130**(12), 1921-1930.

FEMA (2000a), Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, DC.

FEMA (2000b), Global Topics Reports on the Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, DC.

Filiatrault, A., Fischer, D., Folz, B., Uang, C.M. and Seible, F. (2000), "Shake table test of a two-story woodframe house", Report No. SSRP-2000/15, Struc. Sys. Res. Proj., Dept. of Structural Engineering, University of California San Diego, La Jolla, CA.

Folz, B. and Filiatrault, A. (2000), "CASHEW – Version 1.0: A computer program for cyclic analysis of wood shear walls", Report No. SSRP-2000/10, Struc. Sys. Res. Proj., Dept. of Structural Engineering, University of California San Diego, La Jolla, CA.

Folz, B. and Filiatrault, A. (2001), "Cyclic analysis of wood shear walls", *J. Struct. Eng.*, ASCE, **127**(4), 433-441.

Fonseca, F.S., Rose, S.K. and Campbell, S.H. (2002), "Nail, screw and staple fastener connections", CUREE Publication No. W-16, Consortium of Universities for Research in Earthquake Engineering, Richmond, CA.

Gatto, K. and Uang, C.M. (2001), "Cyclic response of woodframe shearwalls: Loading protocol and rate of loading effects", Report No. SSRP-2001/06, Struc. Sys. Res. Proj., Department of Structural Engineering, UC-San Diego, La Jolla, CA.

ICBO (1997), *Uniform Building Code*, International Conference of Building Officials, Whittier, CA.

Isoda, H., Folz, B. and Filiatrault, A. (2002), "Seismic modeling of index woodframe buildings", CUREE Publication No. W-12, Consortium of Universities for Research In Earthquake Engineering, Richmond, CA.

Kim, J.H. (2001), "Performance-based design of light-frame structures subjected to natural hazard", Ph.D. Proposal, Department of Civil, Construction, and Environmental Engineering, Oregon State University, Corvallis, OR.

Kim, J.H. (2003), "Performance-based seismic design of light-frame shearwalls", Ph.D. Dissertation, Department of Civil, Construction, and Environmental Engineering, Oregon State University, Corvallis, OR.

Kim, J.H., Kent, S.M. and Rosowsky, D.V. (2005), "The effect of biological deterioration on the dynamic performance of woodframe shearwalls", *Computer-Aided Civil and Infrastructure Engineering* (in press).

Krawinkler, H., Parisi, F., Ibarra, L., Ayoub, A. and Medina, R. (2000), "Development of a testing protocol for wood frame structures", CUREE Publication No. W-02, Consortium of Universities for Research in

Earthquake Engineering, Richmond, CA.
Pardoen, G.C., Kazanjy, R.P., Hamilton, C.H., Waltman, A. and Freund, E. (2002), "Testing and analysis of one-story and two-story shearwalls under cyclic loading", Task 1.4.4. Draft Report, CUREE-Caltech Woodframe Project, UC-Irvine, Irvine, CA.
Rosowsky, D. (2002), "Reliability-based seismic design of wood shear walls", *J. Struct. Eng.*, ASCE, **128**(11), 1439-1453.
Rosowsky, D. and Ellingwood, B. (2002), "Performance-based engineering of wood frame housing: A fragility analysis methodology", *J. Struct. Eng.*, ASCE, **128**(1), 32-38.
Rosowsky, D.V. and Kim, J.H. (2002), "Reliability studies", CUREE Publication No. W-10, CUREE-Caltech Woodframe Project, Richmond, CA.
Rosowsky, D.V. and Kim, J.H. (2004), "Incorporating nonstructural finish effects and construction quality issues into a performance-based framework for wood shearwall selection", 2004 ASCE/SEI Structures Congress, Nashville, TN.
SEAOSC (2001), *Report of a Testing Program of Light-Framed Walls with Wood-Sheathed Shear Panels*, Final Report to the City of Los Angeles Department of Building and Safety, Structural Engineers Association of Southern California and Department of Civil and Environmental Engineering, UC-Irvine, December.