Seismic Vulnerability of Four Soft-Story Woodframe Index Buildings and their Retrofits

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Revision history

Ver.	Date	Comments
1.0	12 Dec 2008	Initial release
1.1	12 Jan 2009	Formatting changes
1.2	10 Apr 2009	Adopt changes from fragility paper, wording changes for clarity, show red-tag and collapse probability

EXECUTIVE SUMMARY

A companion work offers fragility functions for major components common to multifamily, soft-story, woodframe dwellings: straight sheathed walls, lath and plaster interior finish, brick veneer, and stucco exterior sheathing. It also proposes fragility functions for red-tagging and collapse. Another work offers pushover curves for 4 particular soft-story, woodframe multifamily dwellings 3 or more stories in height and with 5 or more housing units. Each "index building" is modeled under as-is conditions and with 3 seismic retrofit alternatives, for a total of 16 buildings. A third offers a non-iterative approach to the capacity spectrum method of structural analysis and other principles to create seismic vulnerability functions: relationships giving various measures of damage, life safety, and economic loss versus 5%-damped spectral acceleration response at 0.3 sec and 1.0 sec periods. The purpose of this report is to derive from these prior works motion-damage relationships for the 16 index buildings. The relationships give the mean damage factor (repair cost as a fraction of replacement cost) and damage-state probabilities for each building as functions of spectral acceleration response, earthquake magnitude, fault rupture distance, and NEHRP site soil class. The results, illustrated in Figures ES-1 through 3, are proposed for studying seismic risk mitigation options for the City of San Francisco's Community Action Plan for Seismic Safety (CAPSS). They may be useful for other purposes as well.

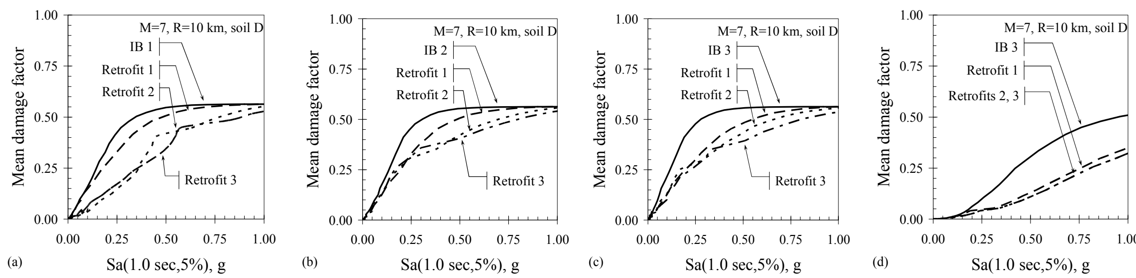


Figure ES-1. Vulnerability functions for the index buildings as a function of Sa(1.0 sec, 5%)

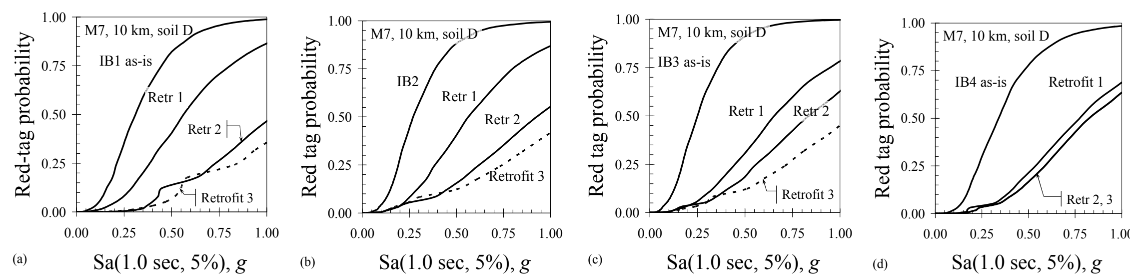


Figure ES-2. Red-tag probability as a function of Sa(1.0 sec, 5%) for the index buildings

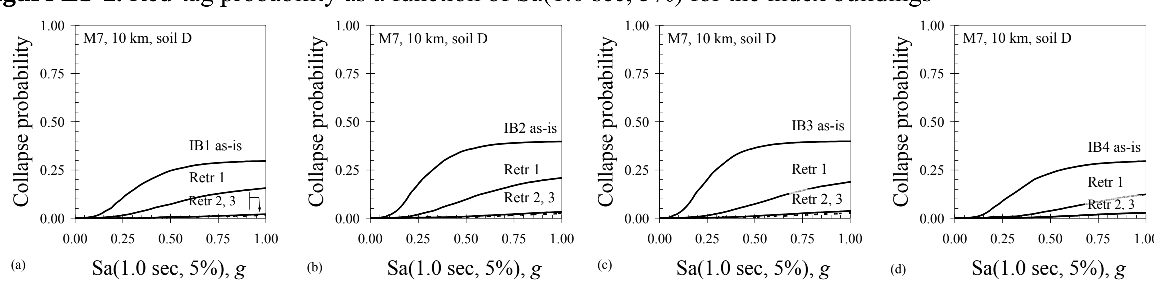


Figure ES-3. Collapse probability as a function of Sa(1.0 sec, 5%) for the index buildings

TABLE OF CONTENTS

INDEX OF FIGURES

INDEX OF TABLES

OBJECTIVES AND SCOPE OF WORK

The City of San Francisco's Community Action Plan for Seismic Safety (CAPSS) project seeks among other things to assess the impacts of various realistic earthquake scenarios on the City's housing stock, with emphasis on one of its more widespread and vulnerable classes of housing: older, woodframe, multifamily dwellings (with 5 or more housing units) with soft-story conditions. It is desired to estimate losses to these buildings under several earthquake scenarios, which requires models of their seismic vulnerability. The relevant housing stock is idealized via four "index buildings" shown in Figure 1: (1) a 3,600-sf, 3-story pre-WWII, woodframe with garage door openings along one entire side of the building, with few transverse walls at ground level; (2) a 5,800-sf, 4-story pre-WWII, woodframe corner building with garage door openings along two partial sides of the building and internal walls at ground level between several parking spaces; (3) a 2,300-sf, midblock, pre-WWII, 4-story building with a neighbor on both sides; and (4) a 1,800-sf, midblock, 1950s, > 80% open on ground-floor façade, 3-story building with neighbors on both sides. SPA Risk LLC undertook several tasks in this regard for the Applied Technology Council:

(1) To identify common building components of buildings in this era of construction, and to develop fragility functions for them, i.e., relationships between component forces or deformation and the probability of various levels of damage to those components.

(2) To develop seismic vulnerability relationships for these buildings, i.e., relating overall repair cost as a fraction of replacement cost new to spectral acceleration response.

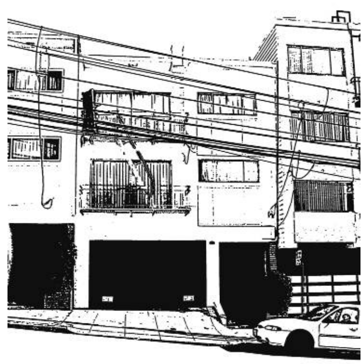
(3) To select or design a loss-estimation methodology, perform the loss calculations, and present the methodology and results in meetings and in a written report.

Tasks 1 and 3 are addressed in other reports. The present work addresses Task 2.

(1)

(2)

(3)



(4)

Figure 1. Index buildings. Four older, soft-story, woodframe, multifamily dwellings representing a realistic range of performance of buildings of this class in San Francisco: (1) corner, 3 story, no interior walls at garage level, one street facade ≥ 80% open at ground floor; (2) corner, 4 story, both street facades ≥ 50% open at ground floor; (3) mid-block, 4 story, pre-WWII, neighbors on both sides; (4) mid-block, 3 story, post-1950, neighbors on both sides. Square footage is 3,600, 5,800, 2,300, and 1,800 sf, respectively.

AVAILABLE VULNERABILITY METHODOLOGIES

The PEER and HAZUS-MH methodologies of performance-based earthquake engineering (e.g., Porter et al. 2002 and NIBS and FEMA 2003a, respectively) both offer the means to relate probabilistic damage, economic, or life-safety losses to ground motion measures such as 5%-damped elastic spectral acceleration response. The PEER approach models a building at the level of detail of structural design, uses multiple 2-D or 3-D nonlinear dynamic structural analyses, and applies fragility functions at level of individual wall segments.

The HAZUS-MH approach by contrast idealizes a building as a single-degree-of-freedom nonlinear oscillator and employs the capacity spectrum method of structural analysis. It simplifies a building as comprising only three aggregate components: structural, nonstructural drift-sensitive, and nonstructural acceleration sensitive, each with 5 or 6 somewhat qualitative damage states. When both approaches are applied carefully, the PEER approach offers far greater resolution, but it is far more labor intensive, largely because of the effort creating a probabilistic structural model, and it can be computationally expensive for large buildings modeled in 3 dimensions. The HAZUS-MH approach offers less fidelity to the behavior of real buildings, but has been used to hindcast societal losses with ± 50% accuracy in the 1989 Loma Prieta and 1994 Northridge earthquakes (NIBS and FEMA 2001), and has produced what are deemed to be realistic estimates of losses in a future repeat of the 1906 San Francisco earthquake (Kircher et al. 2006) and in a possible future M7.8, 300-km-long rupture of the southern San Andreas Fault (Jones et al. 2008). For purposes of the CAPSS study, the HAZUS-MH approach is practical and the PEER approach is not. Furthermore since CAPSS aims at societal-level risk assessment, the fidelity offered by a HAZUS-MH approach is deemed adequate.

HAZUS-MH reflects old, large woodframe buildings with the seismic vulnerability model W2 pre-code. HAZUS-MH also contains another building type for woodframe construction (W1), but this type is generally smaller than the index buildings, both in terms of height (W1 is idealized with a 1-story buildings) and area (W1 has an area of less than 5,000 sf). The HAZUS-MH W2 type alone cannot distinguish the effects of building configuration or details such as soft story and the detailed seismic retrofits examined here. The HAZUS-MH Advanced Engineering Building Module (AEBM; NIBS and FEMA 2003b) provides the means to calculate the seismic performance of particular buildings. However, the AEBM was found to have a programming flaw in calculating the performance point when it lies on the constant-velocity portion of the idealized response spectrum with effective damping greater than 5%. It is unclear how frequently and severely the flaw impacts results. A patch was not available at the time of this work.

Because of these challenges to using HAZUS-MH or the AEBM, an alternative approach developed by Porter (2009b) is used here. The alternative honors all HAZUS-MH modeling assumptions while avoiding the use of AEBM and the requirement for iterative calculation of the performance point. It has been peer reviewed and its results independently duplicated by several researchers.

SELECTED METHODOLOGY

The present work concerns the effort of relating 5%-damped spectral acceleration response at 0.3 sec or 1.0-sec periods to damage and loss, not the calculation of loss given some scenario shaking. The interested reader is referred to Porter (2009b,c) for details on the methodology. In brief, one must create a pushover curve, referred to as a capacity curve in the HAZUS-MH methodology, which relates the peak acceleration of the equivalent SDOF nonlinear oscillator to its displacement, i.e., in the space of spectral displacement (S_d) and spectral acceleration (S_a) response. As shown in Figure 2, the capacity curve has a linear portion between the origin and a yield point denoted by (D_y, A_y), a perfectly plastic portion when displacement exceeds an ultimate point denoted by (D_u, A_u), and a portion of an ellipse connecting the two segments. It is discretized into a number of points; Porter (2009b) uses 51 equally logarithmically spaced values of S_d between 0.01 in and 1000 in, though for low- and midrise woodframe buildings a useful upper limit is more like 10 in to perhaps 100 inches. At each S_d value, one calculates the corresponding S_a value and the effective damping ratio, denoted by B_{eff} and calculated as

$$B_{eff} = B_E + \kappa\left(\frac{Area}{2\pi S_d S_a}\right) \tag{1}$$

where B_E is the elastic damping of the model building type, κ is a degradation factor, and *Area* is the area enclosed by the hysteresis loop as in Figure 2. Ignoring the rounded corners,

$$Area \approx 4S_a\left(S_d - \frac{S_a}{A_y/D_y}\right) \tag{2}$$

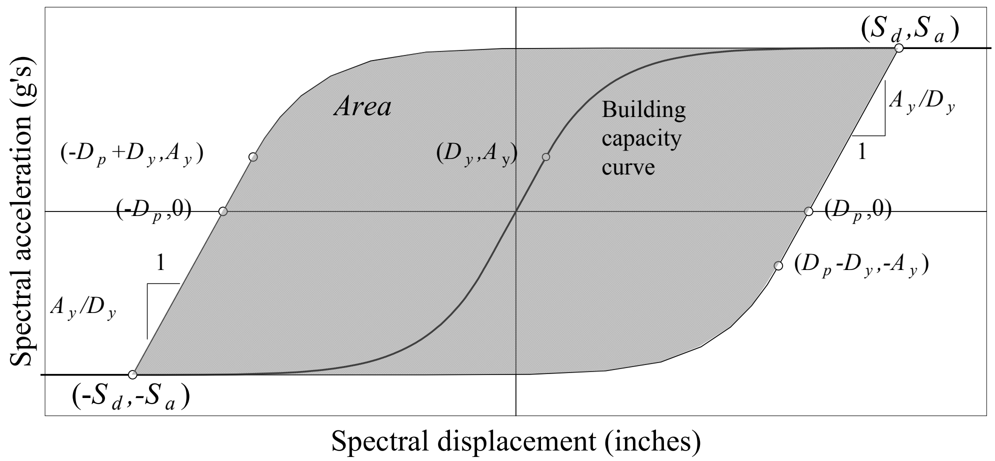


Figure 2. Establishing effective damping ratio at HAZUS-MH performance point

Omitting details, the performance point (S_d, S_a, B_{eff}) lies on an idealized, site-soil-adjusted response spectrum with the same effective damping ratio and called the *demand spectrum*. Considering the effect of damping and site soil amplification, the demand spectrum is then related to a 5%-damped site-soil-adjusted response spectrum referred to as the *index spectrum*. The index spectrum has a constant-acceleration portion parameterized via its 5%-damped spectral acceleration response at 0.3 sec period denoted by S_a(0.3 sec, 5%), and a constant-velocity portion parameterized via S_a(1.0 sec, 5%). The relationships between the spectral acceleration response at 0.3 and 1.0 sec on both demand and index spectra, and whether the performance point lies on the constant-acceleration or constant-velocity portions of the demand spectrum, depend on several parameters: the earthquake magnitude M, fault distance R, mean shearwave velocity in the top 30m of soil Vs30, and whether the site is near a plate boundary or in a continental interior.

Probabilistic structural damage at the performance point is then calculated using fragility functions of the form:

$$
\begin{aligned}
P\left[D=d\,\middle|\,S_d=x\right] &= 1-\Phi\left(\frac{\ln\left[x/\theta_1\right]}{\beta_1}\right) && d=0\\
&= \Phi\left(\frac{\ln\left[x/\theta_d\right]}{\beta_d}\right)-\Phi\left(\frac{\ln\left[x/\theta_{d+1}\right]}{\beta_{d+1}}\right) && 1\le d\le 3\\
&= \left(1-P_c\right)\Phi\left(\frac{\ln\left[x/\theta_4\right]}{\beta_4}\right) && d=4\\
&= P_c\Phi\left(\frac{\ln\left[x/\theta_4\right]}{\beta_4}\right) && d=5
\end{aligned}
\tag{3}
$$

where $P[D = d|S_d = x]$ denotes the probability of structural damage state d given that S_d takes on some particular value x, and Φ denotes the cumulative standard normal distribution. The parameters θ_i, β_i, and P_c denote, respectively, the median and logarithmic standard deviation values of the component capacity to resist damage state i, and the fraction of buildings in the complete damage state that are expected to be collapsed. Damage to the nonstructural drift-sensitive component is similar except that only 4 damage states are considered. Damage to the nonstructural acceleration-sensitive component is also similar, again with only 4 damage states and instead of conditioning on S_d, it uses S_a as the input to the fragility functions.

An alternative to Equation (3) can be formulated wherein collapse fragility is explicitly modeled, as in

$$
\begin{aligned}
P\left[D=d\,\middle|\,S_d=x\right] &= 1-\Phi\left(\frac{\ln\left[x/\theta_1\right]}{\beta_1}\right) && d=0\\
&= \Phi\left(\frac{\ln\left[x/\theta_d\right]}{\beta_d}\right)-\Phi\left(\frac{\ln\left[x/\theta_{d+1}\right]}{\beta_{d+1}}\right) && 1\le d\le 4\\
&= \Phi\left(\frac{\ln\left[x/\theta_5\right]}{\beta_5}\right) && d=5
\end{aligned}
\tag{4}
$$

where θ_5 and β_5 represent median and logarithmic standard deviation of spectral displacement associated with collapse (a damage state that applies only to the structural component). In either case, given structural response S_d and S_a, Equations (3) and (4) each produces a probability mass function for the damage state of the structural, nonstructural drift-sensitive, and nonstructural acceleration-sensitive building components. These uncertain damage states are denoted here by D_1, D_2, and D_3, respectively. With these probability mass functions available, the expected value of repair cost is calculated as

$$E\left[L\right]=\sum_{d=1}^{5}P\left[D_1=d\left|S_d=x\right.\right]L_{1d}+\sum_{d=1}^{4}P\left[D_2=d\left|S_d=x\right.\right]L_{2d}+\sum_{d=1}^{4}P\left[D_3=d\left|S_a=x\right.\right]L_{3d} \quad (5)$$

where L_{1d}, L_{2d}, and L_{3d} represent the repair cost to the same three components, given that the component is in damage state d.

PARAMETER VALUES FOR CAPSS INDEX BUILDINGS

Pushover curve parameters D_y, A_y, D_u, and A_u are taken from analysis by Cobeen (2008), and are recapped in Table 1, along with calculated elastic period T_E and ductility capacity μ ($=D_u/D_y$). Units of D, A, T_E, and μ are inches, gravities, seconds, and unitless, respectively. Cobeen (2008) provides pushover parameter values in each direction; their geometric means are shown in Table 1. Several checks imply internal consistency and reasonable comparison with HAZUS-MH, as follows:

1. Ultimate strengths (Au) increase with retrofit level; okay.
2. Periods generally decrease with retrofit, though 1r3 has slightly longer period than 1r2, as does 2r3 vs. 2r2, which at first glance appears questionable given the greater number of columns with roughly same member size (W10x45) as columns in the bents (W10x45 and W12x45 in 1r3 and 2r3, respectively), and the presumably nearly fixed base of the W10 cantilever columns. This issue is under examination now.
3. Ductility capacity Du/Dy generally increases, from 1.3 to 2.8, though Du/Dy drops from 1r2 to 1r3. Okay.
4. Ultimate strengths for 1 and 2 are 25-33% that of HAZUS-MH values for W2 pre-code (Au = 0.25g), which seems reasonable.
5. Ultimate strengths for 1r3 and 2r3 are between W2 low-code and moderate-code (Au = 0.25g and 0.5g, respectively), which seems reasonable.
6. Periods for 1, 1r1, and all variants of 2 are longer than HAZUS-MH W2, which has TE = 0.4, despite comparable height. However, HAZUS-MH W2 assumes 2 12-ft stories, so perhaps half the mass per sq ft above 1st story, ~1-2x stiffness per sq ft at 1st story, and no soft story, so expect one would expect periods of 1.4 to 2.8 times longer than HAZUS-MH W2 for the as-is buildings, which is what is observed, so okay.
7. Ductility capacities are small compared with all HAZUS W2 even pre-code, which has Du/Dy = 15. However the HAZUS-MH figure seems unrealistic, so okay.

Median capacities for the drift-sensitive components are taken from Porter (2009a) and are recapped in Table 2. Since the mapping from observable damage states to HAZUS-MH damage states was shown to be uncertain, a second alternative for median capacities is shown in Table 3. Other values are taken as the HAZUS-MH defaults, as recapped in Table 2.

Table 1. Capacity curve parameters, after Cobeen (2008)

Index building	D_y	A_y	D_u	A_u	T_E	μ
1. 3-story corner building one side > 80% open no walls between garages	0.57	0.06	0.76	0.08	1.00	1.32
1r1. New wood shearwalls at garage end walls, steel frames at garage doors	0.70	0.08	1.41	0.13	0.92	2.02
1r2. Ditto but new wood shearwalls at all ground-floor walls	0.34	0.23	0.95	0.37	0.39	2.79
1r3. Ditto but cantilever columns at garage openings instead of moment frames	0.55	0.28	1.08	0.38	0.45	1.96
2. 4-story corner building both sides > 50% open, walls between garages	0.51	0.05	0.71	0.06	1.00	1.38
2r1. Ditto with new steel frames both facades	0.68	0.11	1.03	0.13	0.80	1.51
2r2. Ditto but wood shearwalls all interior garage walls	0.84	0.20	2.09	0.31	0.65	2.49
2r3. Ditto but cantilever columns at garage openings instead of moment frames	0.99	0.23	2.62	0.37	0.67	2.64
3. 4-story midblock building pre-WWII, front façade > 80% open	0.65	0.04	1.42	0.05	1.31	2.18
3r1. Ditto, plus transverse woodframe shearwalls	0.81	0.13	1.57	0.18	0.81	1.93
3r2. Ditto, transverse steel frames and longitudinal shearwalls	0.84	0.19	1.60	0.25	0.68	1.90
3r3. Ditto, cantilever columns not steel frames, more longitudinal shearwalls	0.84	0.23	1.69	0.30	0.61	2.01
4. 3-story midblock 1950s building front façade > 80% open	0.11	0.02	0.84	0.06	0.75	7.75
4r1. Ditto, plus woodframe shearwalls on longitudinal end walls and back wall	0.29	0.11	0.76	0.17	0.53	2.65
4r2. Ditto, plus steel moment frame at front	0.23	0.11	0.65	0.17	0.47	2.83
4r3. Ditto, but cantilever columns inboard instead of steel moment frame at front	0.23	0.11	0.65	0.17	0.47	2.83

Table 2. Parameters employed in the present study, alternative 1

Parameter		Source
κ_S	degradation factor short-duration event (M ≤ 5.5)	IB1-3 as-is: 0.40 = HAZUS-MH woodframe > 5000 sf pre-code Retrofits 1-3: 0.60, 0.80, 0.90 resp. = low-, mod-, high-code IB4 as is: 0.80 = HAZUS-MH woodframe > 5000 sf mod code IB4 retrofit 1: 0.85, retrofits 2-3: 0.90 = ditto, high code
κ_M	ditto, medium duration (5.5 < M < 7.5)	IB1-3 as-is: 0.2, retrofit 1: 0.3, retrofit 2: 0.4, retrofit 3: 0.6 IB4 as-is: 0.4, retrofits 1: 0.5, retrofits 2-3: 0.6
κ_L	ditto, long duration (M ≥ 7.5)	IB1-3 as-is: 0.0, retrofit 1: 0.1, retrofit 2: 0.2, retrofit 3: 0.4 IB4 as-is: 0.2, retrofits 1: 0.3, retrofits 2-3: 0.4
B_E	Elastic damping ratio	Porter et al. (2002): 10%, from system identification of strong-motion records from several California woodframe buildings
θ_{11}	Median S_d where structural component reaches or exceeds “slight” damage	IB1-3: Porter (2009a): 0.06 in; from lab tests of older materials IB4: 0.86 in. = HAZUS-MH for mod-code woodframe > 5000 sf
θ_{12}	Ditto, moderate	IB1-3: Ditto, 0.4 in. for buildings with brick veneer, else 1.2 in. IB4: 2.14 in. = HAZUS-MH for mod-code woodframe > 5000 sf
θ_{13}	Ditto, extensive	IB1-3: Ditto, 1.2 in. IB4: 6.62 in. = HAZUS-MH for mod-code woodframe > 5000 sf
θ_{14}	Ditto, complete	IB1-3: Ditto, 2.5 in. IB4: 16.2 in. = HAZUS-MH for mod-code woodframe > 5000 sf
P_c	Fraction of building area collapsed, given complete damage	IB1-2 as-is: 14%, from SF Marina District 1989 IB1-2 retrofits 1-3: 6%, 1.5%, 1.5%, respectively IB3-4 as-is: 3%, retrofits 1-3: 2%, 1.5%, 1% respectively
θ_{21}	Median S_d for slight nonstruct. drift-sens. damage	Taken same as θ_{11}
θ_{22}	Ditto, moderate	Taken same as θ_{12}
θ_{23}	Ditto, extensive	Taken same as θ_{13}
θ_{24}	Ditto, complete	Taken same as θ_{14}
θ_{31}	Ditto, S_a, nonstructural accel-sensitive, slight	HAZUS-MH default for woodframe > 5000 sf: 0.2g
θ_{32}	Ditto, moderate	Ditto: 0.4g
θ_{33}	Ditto, extensive	Ditto: 0.8g
θ_{34}	Ditto, complete	Ditto: 1.6g
β_{11}	Log standard deviation of S_d where structural component reaches or exceeds “slight” damage	Ditto: 1.0
β_{12}	Ditto, moderate	Ditto: 1.0
β_{13}	Ditto, extensive	Ditto: 1.0
β_{14}	Ditto, complete	Ditto: 1.0
β_{21}	Ditto, nonstructural drift-sensitive, slight	Ditto: 1.0
β_{22}	Ditto, moderate	Ditto: 1.0
β_{23}	Ditto, extensive	Ditto: 1.0
β_{24}	Ditto, complete	Ditto: 1.0
β_{31}	Ditto, S_a, nonstructural accel-sensitive, slight	Ditto: 0.7
β_{32}	Ditto, moderate	Ditto: 0.7
β_{33}	Ditto, extensive	Ditto: 0.7
β_{34}	Ditto, complete	Ditto: 0.7
L_{11}	Repair cost, struct., slight, fraction of repl. cost	HAZUS-MH default for multifamily dwelling: 0.003
L_{12}	Ditto, moderate	Ditto: 0.014
L_{13}	Ditto, extensive	Ditto: 0.069
L_{14}	Ditto, complete	Ditto: 0.138
L_{15}	Ditto, collapse	Ditto: 0.138
L_{21}	Ditto, nonstructural drift-sensitive, slight	Ditto: 0.009
L_{22}	Ditto, moderate	Ditto: 0.043
L_{23}	Ditto, extensive	Ditto: 0.213
L_{24}	Ditto, complete	Ditto: 0.425
L_{31}	Ditto, nonstructural acceleration-sensitive, slight	Ditto: 0.008

Parameter		Source
L_{32}	Ditto, moderate	Ditto: 0.043
L_{33}	Ditto, extensive	Ditto: 0.131
L_{34}	Ditto, complete	Ditto: 0.437

Table 3. Fragility parameters employed in the present study, alternative 2

Parameter		Source
θ_{11}	Median S_d where structural component reaches or exceeds "slight" damage	IB1-3: Porter (2009a): 0.4 in; from lab tests of older materials IB4: 0.86 in. = HAZUS-MH for mod-code woodframe > 5000 sf
θ_{12}	Ditto, moderate	IB1-3: Ditto, 1.5 in. for buildings with brick veneer, else 1.2 in. IB4: 2.14 in. = HAZUS-MH for mod-code woodframe > 5000 sf
θ_{13}	Ditto, extensive	IB1-3: Ditto, 3.0 in. IB4: 6.62 in. = HAZUS-MH for mod-code woodframe > 5000 sf
θ_{14}	Ditto, complete	IB1-3: Ditto, 5.0 in. as-is, 8 in retrofit IB4: 16.2 in. = HAZUS-MH for mod-code woodframe > 5000 sf
θ_{15}	Ditto, collapse	IB1-3: 13 in. IB4: 24 in. = 1.5 x θ_{14}
θ_{21}	Median S_d for slight nonstruct. drift-sens. damage	Taken same as θ_{11}
θ_{22}	Ditto, moderate	Taken same as θ_{12}
θ_{23}	Ditto, extensive	Taken same as θ_{13}
θ_{24}	Ditto, complete	Taken same as θ_{14}
θ_{31}	Ditto, S_a, nonstructural accel-sensitive, slight	HAZUS-MH default for woodframe > 5000 sf: 0.2g
θ_{32}	Ditto, moderate	Ditto: 0.4g
θ_{33}	Ditto, extensive	Ditto: 0.8g
θ_{34}	Ditto, complete	Ditto: 1.6g

RESULTS

The methodology presented in Porter (2009b,c) and summarized above was employed to develop a vulnerability function for each of the index buildings listed in Table 1. The calculations were performed for every combination of 16 index buildings, 1 occupancy type (multifamily residential, denoted by RES3 in HAZUS-MH), 5 NEHRP site soil classes (A, B, C, D, and E), 4 earthquake magnitudes (5, 6, 7, and 8), 4 site distances (10, 20, 40, and 80 km), and 1 seismic region: western US. The mean damage factor results are compiled in a database table laid out as shown in Table 4. The fragility results are compiled in a database table laid out as shown in Table 5.

Table 4. Sample layout of vulnerability-function table

IB	Occ	Domain	M	R	Siteclass	IM	S_SF_a	S_1F_v	L
CAPSS1	RES3	WUS	7	80	E	Sa10	0.25	0.16	0.28
CAPSS1	RES3	WUS	7	80	E	Sa10	0.28	0.19	0.33

Table 5. Sample layout of fragility-function table

IB	Occ	Domain	M	R	Siteclass	IM	S_SF_a	S_1F_v	P_{11}	P_{12}	P_{13}	P_{14}	P_{15}
CAPSS1	RES3	WUS	7	80	E	Sa10	0.25	0.16	1.00	0.92	0.61	0.32	0.03
CAPSS1	RES3	WUS	7	80	E	Sa10	0.28	0.19	1.00	0.95	0.69	0.41	0.04

In Table 4, “IB” refers to the index building (e.g., CAPSS1r1 means CAPSS index building 1, retrofit 1). “Occ” refers to the HAZUS occupancy class (e.g., RES3, meaning multi-family dwelling). “Domain” refers to whether the function is appropriate for western US (“WUS”) or central and eastern US (“CEUS”)—only WUS is used here. M refers to the approximate magnitude and R to the approximate distance (used here only for spectral shape and duration effects). “Siteclass” refers to the NEHRP site soil classification (A, B, C, D, or E). “IM” indicates whether the performance point corresponds to a point on the constant-acceleration portion of the index spectrum (“Sa03”) or on the constant-velocity portion of the index spectrum (“Sa10”), and therefore which of the two subsequent intensity measures is probably more appropriate to use to estimate loss: “S_SF_a,” which refers to the 5%-damped, site-soil-adjusted spectral acceleration response at 0.3 sec period, or “S_1F_v,” the 5%-damped, site-soil-adjusted spectral acceleration response at 1.0 sec period, both expressed in units of gravity. “L” refers to mean damage factor, which here gives the expected value of repair cost as a fraction of replacement cost new. The record is interpreted this way: if a CAPSS index building 1 were standing on site class E in the western US, and it were shaken at intensity S_a(1.0 sec, 5%) = 0.16g, then on average the repairs would cost 28% of the replacement cost of the building. The record is only valid for an earthquake of magnitude between 6.5 and 7.5, at a distance of roughly 80 km. The record also says that the same loss could be expected for S_a(0.3 sec, 5%) = 0.25g, but that one should probably use S_a(1.0 sec, 5%) as the intensity measure instead.

In Table 5, P11 through P15 refer to the probability that the structural component reaches or exceeds damage states 1 through 5: slight, moderate, extensive, complete, and collapse, respectively. The 1st record means that the building is almost certainly damaged at least slightly when shaken at S_a(1.0 sec, 5%) = 0.16g, that with 92% probability the damage is at least moderate, 61% probability of extensive damage, 32% probability of at least complete damage, and that 3% of building area at this level of shaking would be collapsed.

The resulting data tables are too voluminous to present here, but some sample, summary charts are given in Figure 3. The vulnerability functions show mean damage factor on the y-axis as a function of 1.0-sec, 5%-damped spectral acceleration response on the x-axis. “Mean damage factor” refers to the average repair cost as a fraction of the replacement cost (new, not depreciated) of the building. The x-axis is limited to 1.0g (roughly 6x the shaking in the Marina District in the 1989 Loma Prieta earthquake) because this is the largest shaking estimated by the CAPSS hazard modeler Treadwell and Rollo (Golesorkhi and Gouchon 2002) anywhere in San Francisco in the earthquake scenarios they examined. The vulnerability functions show that the retrofits generally reduce damage by up to 30-70% depending on shaking intensity, but that the benefit is limited when S_a(1.0 sec, 5%) exceeds about 0.5g (roughly 3x the shaking experienced in the Marina District in the 1989 Loma Prieta earthquake).

The vulnerability functions shown here are labeled M = 7, R = 10 km, soil = D, because the HAZUS-MH methodology holds that these parameters affect the shape of the response spectrum and effective damping, and thus they affect structural response, damage, and loss. The vulnerability functions shown in Figure 3 are limited to $5.5 < M < 7.5$, $R \leq 15$ km, and the average shearwave velocity in the top 30 m of soil is limited to the range $600 \leq Vs30 \leq 1200$ ft/sec.

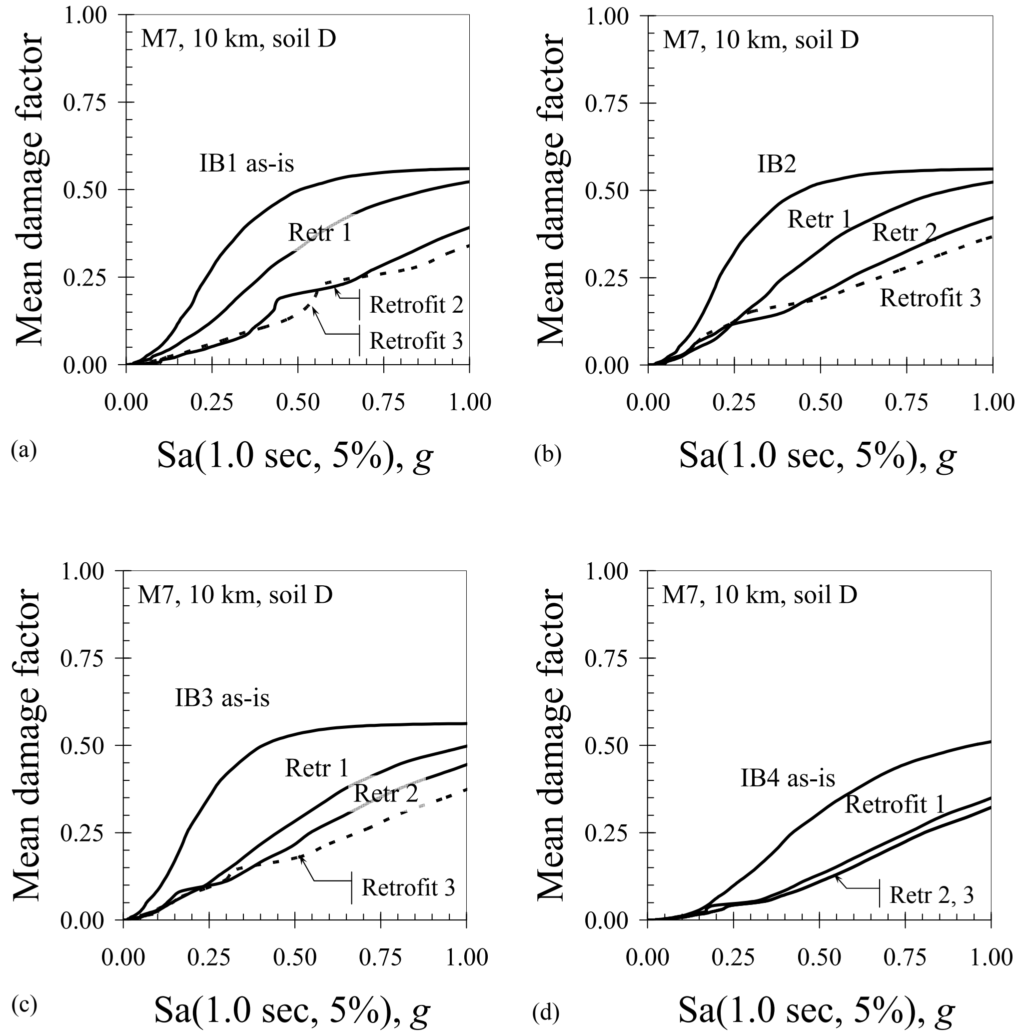


Figure 3. Vulnerability functions for index buildings 1-4 under the more-conservative (lower-loss) fragility alternatives: (a) index building 1, (b) index building 2, (c) index building 3, and (d) index building 4. The x-axis shows 1-sec, 5% damped spectral acceleration response on NEHRP site class D. The y-axis measures mean damage factor, which means the average repair cost as a fraction of the replacement cost of the building.

The vulnerability functions asymptote to a mean damage factor near 0.6. The reason stems from the fact that the index buildings have low values for A_u, as low as 0.05g for one index building and in no case higher than 0.4g. Under the capacity spectrum method of structural analysis, the building cannot experience acceleration greater than A_u. The building's acceleration-sensitive nonstructural components, which represent 44% of building value, cannot experience acceleration greater than A_u. The HAZUS-MH default fragility functions for these components have median capacities of 0.2g for slight damage, 0.4g for moderate damage, and 0.8 and 1.6g for extensive and complete damage. Consequently, there is a low probability that this 44% of building value ever experiences greater than slight or moderate damage, associated with loss of 0.8% and 4.3% of building value, respectively. As a result, there is a low probability that the total repair cost ever exceeds 60% of building replacement cost. Presumably the acceleration-sensitive building components in the ground story are destroyed if the ground story collapses, but this fact is not addressed in NIBS and FEMA (2003).

Another notable feature of the vulnerability functions is that they are sometimes not smooth. The wiggles in these curves are artifacts that the estimated loss is actually a function of two measures of ground motion: S_a(0.3 sec, 5%) and S_a(1.0 sec, 5%). At low levels of ground motion, S_a(0.3) controls the structural response and is used as the intensity measure for purposes of calculating damage and loss; at higher levels, S_a(1.0) controls and is used. The wiggles in the plots of loss versus S_a(1.0) occur at the transition between the two domains. In practice, and as applied here, the proper intensity measure is used.

Not shown in the sample vulnerability functions but apparent in the tables, is that magnitude has a significant impact on loss given S_a, largely because of its modeled impact on effective damping. Distance and site soil class make little difference given M and S_a, largely because at fixed values of M and S_a, distance and site class primarily affect the period at the intersection between the constant-acceleration and constant-velocity portions of the response spectra, which rarely matters. S_a(1.0 sec, 5%) is usually the preferred intensity measure, i.e., the performance point is usually on the constant-velocity portion of the demand spectrum.

Figure 4 shows the red-tag fragility functions for these 16 buildings. It suggests for instance that when index buildings 1 or 2 are subjected to $S_a(1.0) \approx 0.17$g, as estimated from the USGS *ShakeMap* of the Marina District in the 1989 Loma Prieta earthquake, approximately 20% to 25% would be red-tagged. A San Francisco Department of Building Inspection (DBI) database lists 111 soft-story corner buildings in the Marina District. One would therefore estimate approximately 22 to 28 red tags in corner soft-story buildings in the Marina in 1989. There appear to have been about 33, as shown in Figure 5, suggesting an underestimate but perhaps reasonable agreement. Figure 6 shows collapse probability for the index buildings. At $S_a(1.0) \approx 0.17$g, one would expect a 5 to 10% collapse rate among the approximately 111 corner soft-story buildings in the Marina. There were 6, again suggesting reasonable agreement.

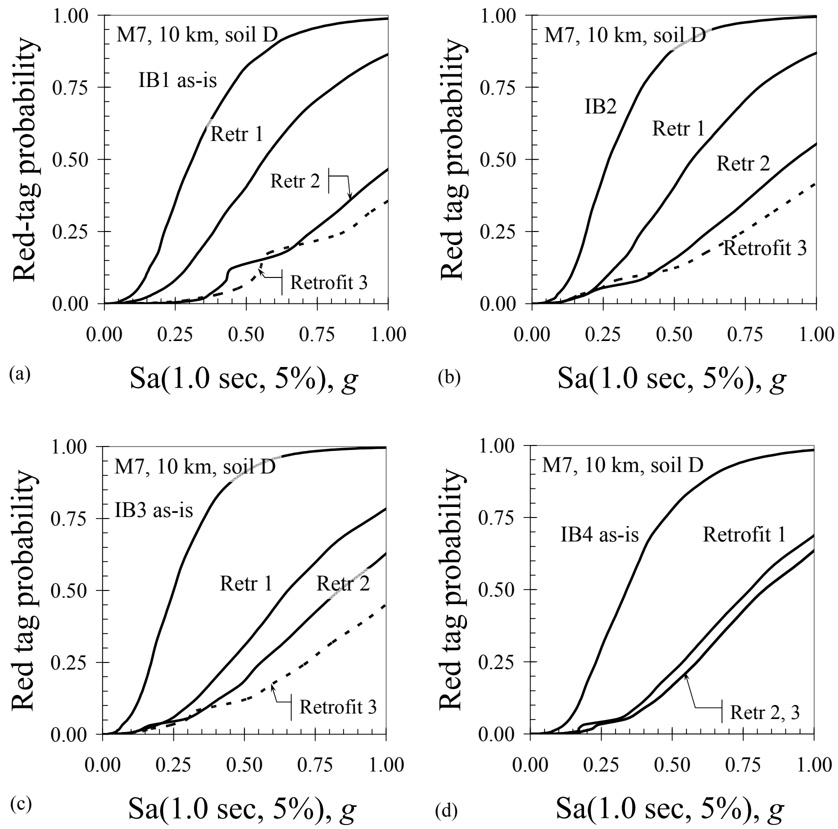


Figure 4. Red-tag probability as a function of Sa(1.0 sec, 5%) for (a) index building 1, (b) index building 2, (c) index building 3, and (d) index building 4

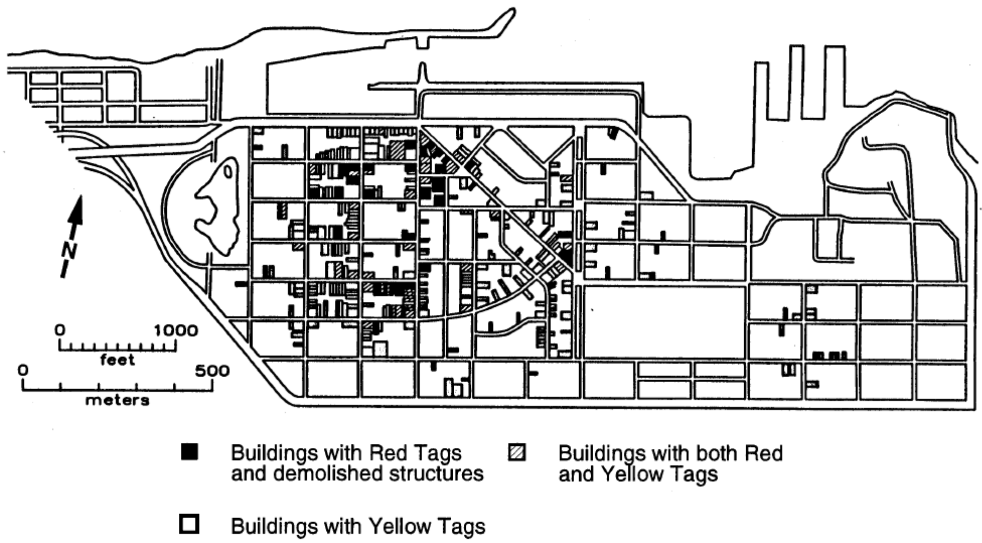


Figure 5. Safety tags in the SF Marina District after the 1989 Loma Prieta earthquake (Seekins et al. 1990 via Scawthorn et al. 1992)

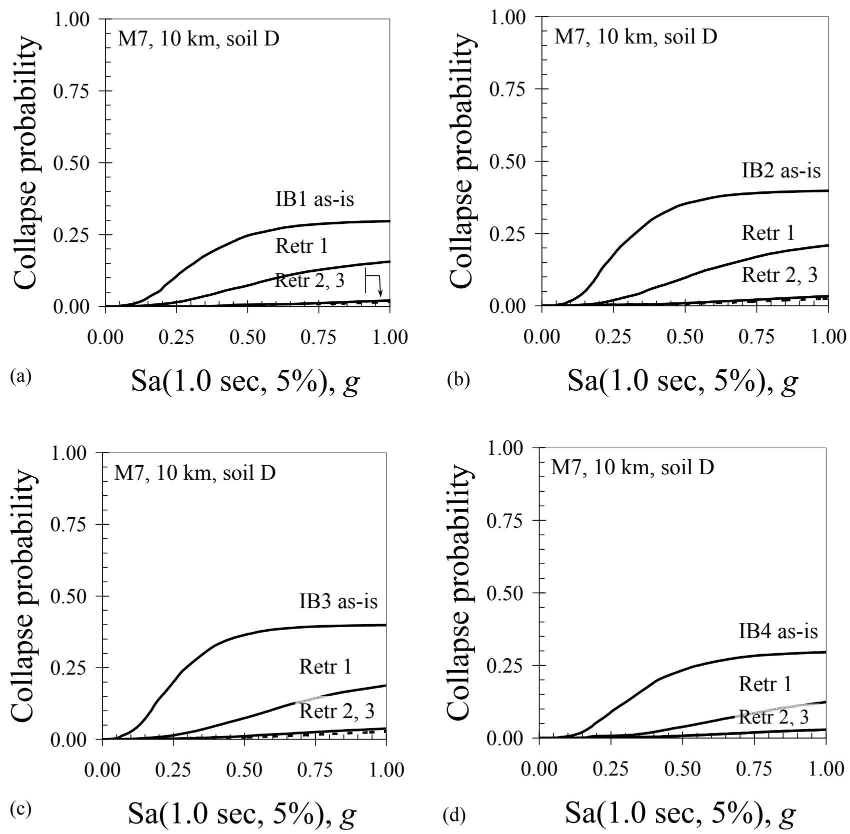


Figure 6. Collapse probability as a function of Sa(1.0 sec, 5%) for (a) index building 1, (b) index building 2, (c) index building 3, and (d) index building 4

CONCLUSIONS

A set of relationships between shaking intensity, damage, and loss were created for 4 buildings that are characteristic of soft-story multifamily woodframe dwellings in San Francisco. Vulnerability and fragility functions were also created for 3 retrofits of each building, for a total of 16 buildings. The relationships were calculated using the HAZUS-MH framework (NIBS and FEMA 2003) developed by Kircher and others for the Federal Emergency Management Agency. While simpler and offering less fidelity than a second-generation performance-based earthquake engineering model (PBEE-2) the HAZUS-MH methodology has been shown in several instances to produce realistic aggregate results. (A PBEE-2 approach would employ a multi-degree-of-freedom structural model examined using nonlinear dynamic structural analysis, along with disaggregated fragility functions, damage, and loss estimates, but would have exceeded the available resources for the present project.)

The buildings and retrofits were selected and designed in consultation with several DBI and consulting structural engineers. Their structural models were created by a reputed local structural engineer (Cobeen) with extensive experience in the performance of woodframe buildings, having led studies for the CUREE-Caltech Woodframe Project and various efforts related to woodframe buildings for the Federal Emergency Management Agency.

The building damage functions were developed considering the fragility of detailed components in them—lath and plaster walls, straight sheathing, brick veneer, and stucco exterior finish—and their observed damageability in laboratory experiments and past earthquake experience, rather than relying on default values offered by the HAZUS-MH developers for a much broader range of building types. A companion document details this effort; it was reviewed by a reputed local SE (Freeman).

A vulnerability-calculation procedure was applied that honors all HAZUS-MH methodologies while avoiding the iteration of the capacity spectrum method and a programming error recently discovered in the HAZUS-MH Advanced Engineering Building Module. The results appear to reasonably hindcast experience of corner soft-story apartment buildings in the Marina District in the 1989 Loma Prieta earthquake.

REFERENCES

Cobeen, K., 2008. *Design and Structural Response Characteristics of Some Large, Soft-story Woodframe Buildings*. Wiss Janney Elstner, Inc., Emeryville, CA.

Golesorkhi R., and J. Gouchon, 2002. *CAPSS Seismic Hazard (Earthquake Scenario).* Treadwell and Rollo, San Francisco, CA, 7 pp.

Jones, L.M., R. Bernknopf, D. Cox, J. Goltz, K. Hudnut, D. Mileti, S. Perry, D. Ponti, K. Porter, M. Reichle, H. Seligson, K. Shoaf, J. Treiman, and A. Wein, 2008. *The ShakeOut Scenario*. U.S. Geological Survey Open-File Report 2008-1150 and California Geological Survey Preliminary Report 25, http://pubs.usgs.gov/of/2008/1150/ [22 May 2008]

Kircher, C.A., H.A. Seligson, J. Bouabid, and G. Morrow, 2006. When the big one strikes again—estimated losses due to a repeat of the 1906 San Francisco earthquake. *Earthquake Spectra*, 22 (S2), S297-S339

(NIBS and FEMA) National Institute of Building Sciences and Federal Emergency Management Agency, 2001. *HAZUS-99 SR1 Validation Study,* Federal Emergency Management Agency, Washington, DC, 149 pp.

(NIBS and FEMA) National Institute of Building Sciences and Federal Emergency Management Agency, 2003a. *Multi-hazard Loss Estimation Methodology, Earthquake Model, HAZUS®MH Technical Manual,* Federal Emergency Management Agency, Washington, DC, 690 pp

(NIBS and FEMA) National Institute of Building Sciences and Federal Emergency Management Agency, 2003b. *HAZUS-MH MR1 Advanced Engineering Building Module Technical and User's Manual,* Federal Emergency Management Agency, Washington, DC, 119 pp.

Porter, K.A., J.L. Beck, H.A. Seligson, C.R. Scawthorn, L.T. Tobin, and T. Boyd, 2002. *Improving Loss Estimation for Woodframe Buildings,* Consortium of Universities for Research in Earthquake Engineering, Richmond, CA, 136 pp., http://resolver.caltech.edu/caltechEERL:2002.EERL-2002-01 (main report) and http://resolver.caltech.edu/caltechEERL:2002.EERL-2002-02 (appendices)

Porter, K.A., 2009a. *Component Fragility Functions for Older Woodframe Construction*. SPA Risk LLC, Denver CO, 32 pp.

Porter, K.A., 2009b (expected). Cracking an open safe: more HAZUS vulnerability functions in terms of structure-independent spectral acceleration. Expected for publication in *Earthquake Spectra* May 2009.

Porter, K.A., 2009c (expected). Cracking an open safe: HAZUS vulnerability functions in terms of structure-independent spectral acceleration. Expected for publication in *Earthquake Spectra* August 2009.

Scawthorn, C.R., K.A. Porter, and F.T. Blackburn, 1992. Loma Prieta, California Earthquake of October 17, 1989, strong ground motion and ground failure, marina district: performance of emergency-response services after the earthquake. *Loma Prieta, California Earthquake of October 17, 1989: Marina District, USGS Professional Paper 1551-F.* O'Rourke, Thomas D, ed., US Government Printing Office, Washington DC, F195-F215.

Seekins, L.C., F. Lew, and L. Kornfield, 1990. Areal distribution of damage to surface structures. *Effects of the Lorna Prieta earthquake on the Marina District, San Francisco, California*. U.S. Geological Survey Open-File Report 90-253, C1-C5.