Loss Estimates for Large Soft-Story Woodframe Buildings in San Francisco

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ABSTRACT

As part of a study for the City of San Francisco entitled *Community Action Plan for Seismic Safety* (CAPSS), we estimated the consequences of 4 large Bay Area earthquakes for large soft-story woodframe dwellings in San Francisco to inform City risk-management policy. A survey by the City's Department of Building Inspection found 2,800 residential woodframe buildings with 3 or more stories and 5 of more housing units with soft-story conditions. As proxies for these 2,800 unique buildings, four index buildings and three retrofits for each building were designed and analyzed. We developed nonlinear pseudostatic structural models for each building and retrofit. From lab tests and other observations, we created fragility functions for straight-sheathed woodframe walls, lath and plaster interior wall finish, brick veneer, and stucco veneer, and related the observed damage to HAZUS-MH damage states. We estimated the distribution of drift at which buildings would likely receive a red tag under ATC-20 post-earthquake safety inspection procedures. From observations of similar buildings in the San Francisco Marina District after the 1989 Loma Prieta earthquake, we estimated the transient drift associated with collapse. Using a procedure based on the HAZUS Advanced Engineering Building Module, we estimated the economic loss, ATC-20 safety tag color, and collapse probability for each of the 2,800 buildings in each of the 4 scenario earthquakes. Loss estimates were reviewed by an expert panel. We conclude that, in a M7.2 rupture of the San Andreas Fault, between 30 and 50% of the 2,800 buildings would be rendered unsafe to enter or occupy (i.e., they would be red-tagged), and an additional 10 to 30% would collapse. Seismic retrofit costing \$11,000 to \$17,000 per dwelling unit could reduce red tags to between 6 and 12% of the subject buildings, with 0.3 to 0.7% collapsing—substantial improvements on both counts—though repairs would still be costly.

INTRODUCTION

As part of a study by the Applied Technology Council (ATC) for the City of San Francisco, entitled *Community Action Plan for Seismic Safety* (CAPSS), the consequences of several moderate to large Bay Area earthquakes on large soft-story woodframe dwellings were studied, with the objective of informing San Francisco policy decisions on seismic retrofit for this type of highly vulnerable building. A companion work by Samant et al. (2009) addresses the overall project; the present work offers more technical detail of the damage and loss estimates.

Soft-story woodframe buildings have wood-framed ground stories in which the amount of wall, exterior and interior, is significantly less than at the stories above. According to a survey by the San Francisco Department of Building Inspections (DBI) and encoded in a DBI database, the City has approximately 4,000 woodframe buildings with three or more stories and five or more housing units, representing more than 10% of the City's housing stock. Of these buildings, 2,800 have soft story configurations, defined here as having one ground-level façade at least 80% open or two at least 50% open. These buildings tend to be more vulnerable than others to moderate and major earthquakes. They represent a potentially enormous seismic safety and post-earthquake housing problem for the City of San Francisco, since approximately 8% of City residents live in these buildings. As part of the CAPSS project, we studied the costs and consequences resulting from the retrofit of 4 index buildings, representative of San Francisco multi-family woodframe building stock. This paper presents the index buildings, retrofit designs, anticipated building behavior, scenario earthquakes, fragility functions, loss estimates, and results.

INDEX BUILDINGS

Plans and street front elevations for candidate representative buildings were compiled by San Francisco Bureau of Building Inspection for consideration, and a design charette was held to get input on selection of buildings, retrofit approaches, and performance expectations. Using this information, four buildings were selected for study. See Figure 1 for illustrations of the buildings. It was decided to use two corner buildings and two mid-block buildings. Variables chosen included the number of stories, the size of the building footprint, and the construction materials. Each of the index buildings has a soft story ground story, accommodating parking and storage areas, while the upper stories have residential units with a significant amount of interior partition wall. The soft story ground story could alternatively have housed commercial space; the use of the ground floor does not have a great impact for the buildings and retrofits chosen. Table 1 summarizes the four index buildings.

Figure 1. Representative index buildings 1-4 (from left to right) studied as part of the CAPSS project.

Table 1. Description of index buildings selected for study.

Building	Location	Stories	Sq ft/floor	Replacement cost, $1000	Housing units	Interior finish
1	Corner	3	3,610	$3,600	6	Plaster & wood lath
2	Corner	4	5,800	$7,700	8	Plaster & wood lath
3	Mid-block	4	2,270	$3,000	6	Plaster & wood lath
4	Mid-block	3	1,750	$1,700	4	Gypsum Wallboard

Replacement costs were estimated by five local specialists: an architect, a developer, two insurance experts, and a commercial risk modeler. Their estimates ranged from $190 to $350 per square foot. After rejecting the two lowest values, which came from insurance-company computer models, the average the remaining three estimates, $330 per square foot, was used to estimate post-earthquake repair costs. For each of the index buildings a series of capacity curves were developed to describe the building load-deflection behavior. From these curves, yield and peak capacity forces and displacements were identified, and converted to spectral accelerations and displacements.

RETROFIT DESIGNS

Consideration of performance expectations was framed around comparison to building code force levels, ASCE 41 (2006) performance objectives, and related to four levels of performance approximately based on SPUR (2008) objectives for community resilience: Level A – safe and operational, Level B – safe and usable during repair, Level C – safe and usable after repair, and Level D – Safe but not repairable. Three levels of retrofit design were selected as being reasonable to pursue, keeping in mind that a targeted performance description does not translate into certainty that the performance description will be met for each individual building. Retrofits are summarized in Table 2.

Table 2. Summary of retrofits considered

Retrofit	Performance level target	Average cost per		Total cost, $M
		Building	Housing unit	
1. Structural sheathing at ground-story walls with inadequate bracing length	D	$65,000	$11,000	$180
2. Structural sheathing at ground-story walls and steel moment frames at garage openings	C	$105,000	$17,000	$290
3. Structural sheathing at ground-story walls and steel cantilever columns with lower R factor and same seismic force level as 2	C to B	$93,000	$16,000	$260

Vertical elements of the retrofits were proportioned using 75% of the equivalent lateral seismic force required by the San Francisco Building Code (SFBC) (City and County of San Francisco 2007). They were also checked against SFBC drift requirements. The design of the retrofits was carried out on a line-by-line basis, allowing the R factor to vary by wall line. Ground and upper floor plans and one retrofit plan for Index Building 1 are shown in Figures 2 and 3.

PUSHOVER CURVES AND OBSERVATIONS

For development of the capacity curves, load-deflection behavior from a variety of sources was reviewed. Resources included: Forest Products Laboratory (1956), Ben Schmid (1984), ASCE 41 (2006), and AF&PA Special Design Provisions for Wind and Seismic (SPDWS) (AF&PA 2005). From these references, best estimate capacities and deflections were identified. The values used are shown in Table 2.

These material values were used as bilinear load-deflection curves, and from these the yield and peak capacities for each story and building and bi-linear capacity curves were developed. The pushover curve for Building 1 is shown in Figure 4.

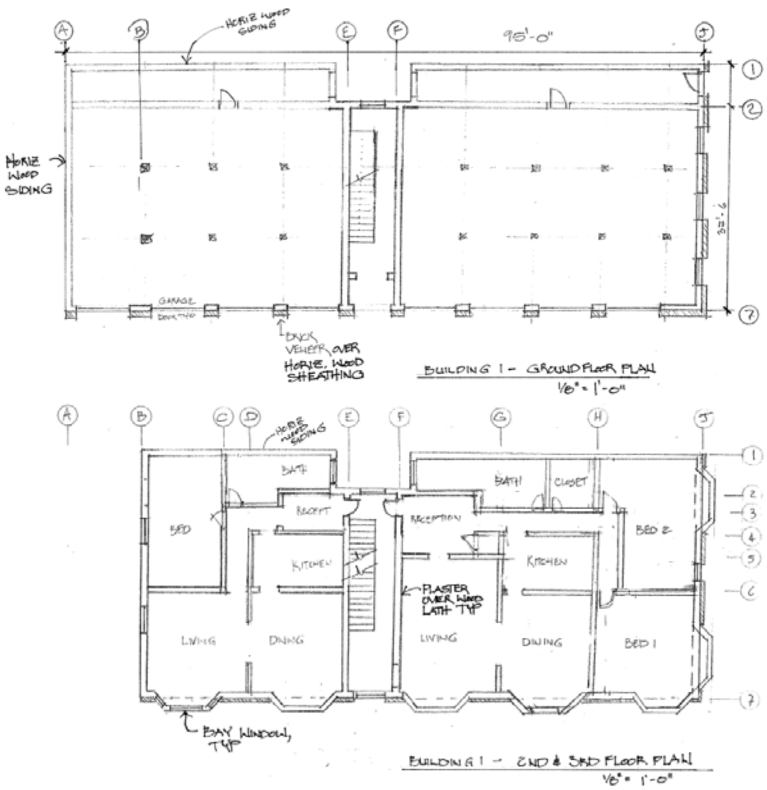


Figure 2. Index Building 1 ground story and upper story plans prior to seismic retrofit.

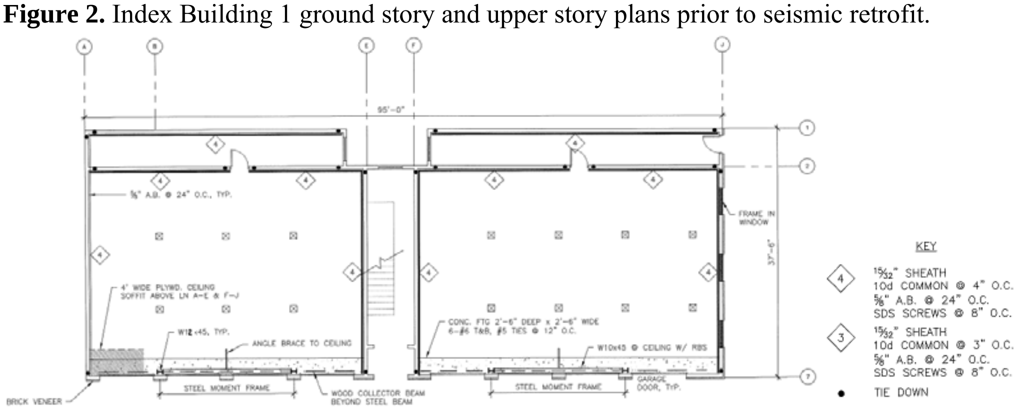


Figure 3. Index Building 1, ground story Retrofit 2 plan. Retrofit is limited to the ground story.

As part of the calculation of the pushover curves it was possible to compare the seismic capacities of the ground floor to upper floors. Upper story yield and peak capacities used in developing the capacity curves were dependant exclusively on the interior finish material of plaster over wood lath for three buildings, and gypsum wallboard for the fourth building. The contribution of other materials was minimal due to high flexibility and low capacity. Using this approach, up to peak capacity of the ground floor retrofit, the upper stories of the buildings with plaster interiors remained at or below yield capacity for the plaster. This suggested that, given the construction materials and the retrofit approaches used in this study, it is very reasonable to limit the retrofit work to the ground story without concern that damage causing life-safety concerns will occur at upper floors. In the gypsum board sheathed building, upper story capacities did control the building peak capacity for some of the retrofits. This may have been a function of conservative properties chosen for the gypsum wallboard. The history of performance of residential light-frame construction suggests that this is not as big a concern as suggested by the analysis, however, further study in this area is recommended.

Table 3. Shear wall capacities and deflections at yield and peak loads

Sheathing	Yield capacity	Yield deflection	Peak capacity	Deflection at peak
Plaster over wood lath	350 plf	0.5"	400 plf	0.7"
Straight horiz. sheath.	160 plf	0.8"	200 plf	3.0"
Gypsum wallboard	67 plf	0.1"	100 plf	0.5"
OSB sheathing	67% peak capacity per ASCE 41	Per SDPWS deflection equation	Per SDPWS tabulated nominal capacity	Per SDPWS deflection equation

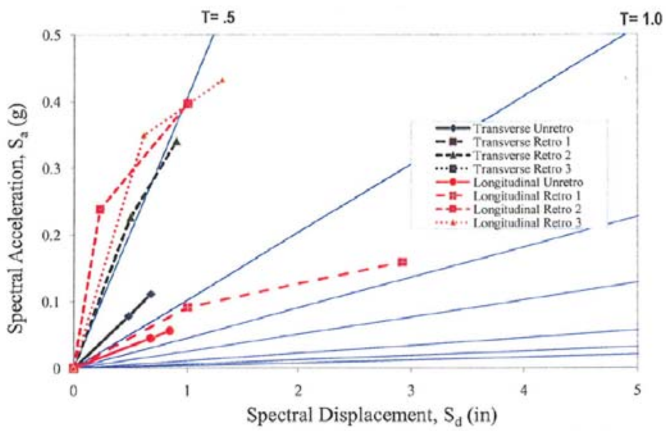


Figure 4. Pushover curves for Building 1.

Building 1 displayed significant torsional response under longitudinal loading in both the original building configuration and Retrofit 1. In the original building configuration the center of rigidity was at the rear longitudinal wall, while in Retrofit 1 the center of rigidity moved very close to the street front at the added moment frames. The effect of the torsion is to put high demands on the end transverse walls. For a ground motion at an angle to the primary axes, the combination of direct and torsional load on these walls is significant and of concern. Significant damage to end transverse walls was identified in CUREE-Caltech Woodframe Project testing of an open wall building on the Berkeley shake table (Mosalam et al. 2002). The particular vulnerability of this geometry of building should be taken into consideration.

SCENARIO EARTHQUAKE GROUND MOTIONS

As part of the original CAPSS work by ATC, the engineering firm Treadwell and Rollo estimated the shaking intensities across San Francisco from each of four scenario earthquakes: a M7.9 rupture of the San Andreas Fault, representing a repeat of the 1906 earthquake with a return period of approximately T = 360 yr; a M7.2 event rupturing the 60-km long Peninsula segment of the San Andreas Fault; a M6.5 event on an 18-km portion of the San Andreas Fault nearest to San Francisco (T ≈ 130 yr); and a M6.9 event rupturing 36 km of the Hayward Fault (T ≈ 520 yr). The study predated the Next Generation Attenuation (NGA) relationships. The authors used then-current attenuation relationships. The resulting maps of shaking intensity in terms of peak ground acceleration are shown in Figure 5. Maps of 5%-damped, 0.3-sec and 1.0-sec spectral acceleration response were also generated.

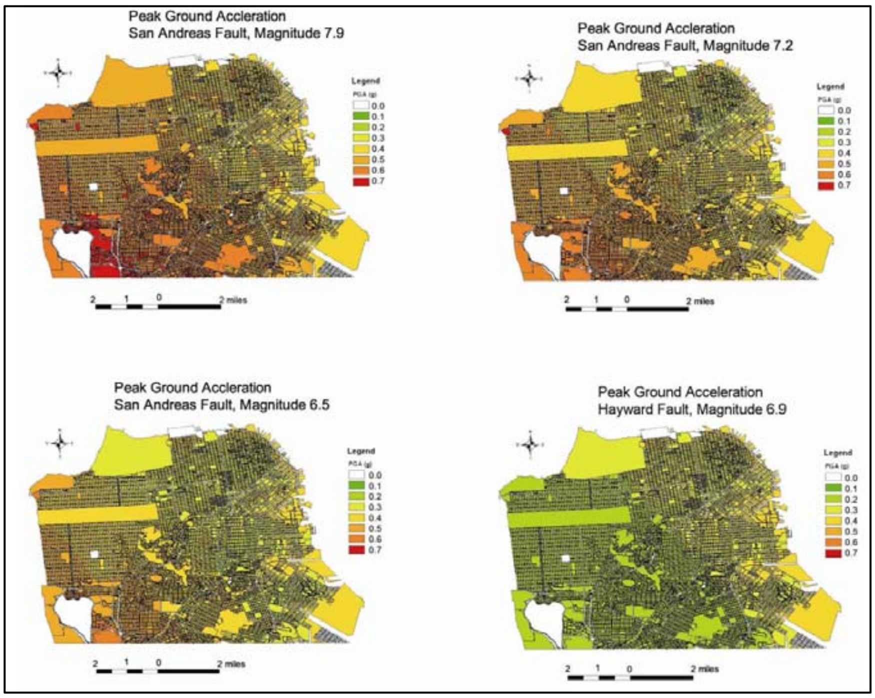


Figure 5. Shaking intensity in 4 scenario earthquakes.

Treadwell and Rollo's estimates suggest average shaking on the order of S_a(1.0 sec, 5%) = 0.30g in the M6.9 Hayward Fault event, 0.35g in the M6.5 San Andreas Fault event, 0.50g in the M7.2 San Andreas Fault event, and 0.67g in the M7.9 San Andreas Fault event. For reference, the USGS ShakeMap for the 1989 Loma Prieta earthquake estimates that sites in the San Francisco Marina District on soft soil (NEHRP categories D or E) experienced roughly S_a(1.0 sec, 5%) = 0.17g, i.e., 1/4 to 1/2 the average citywide shaking of any of these events.

For reference, of the 111 corner, soft-story woodframe buildings of 3 or more stories and 5 or more dwelling units in the Marina District shown in the DBI database, approximately 33 were red-tagged (i.e., 30%), including 6 collapses (5%), per Seekins et al.'s (1990) map. Harris and Egan (1992) studied 74 soft-story apartment buildings in the Marina District and report that 11 of the 74 (15%) experienced major damage or collapsed. Here, major damage refers to ATC-13 (1985) damage state 6, roughly equivalent to the HAZUS-MH complete damage state. Collapse here means that at least the 1st story gravity system failed to the point that the 2nd floor dropped to the ground, touching the 1st floor. Thus, the CAPSS scenarios produce 2-4x the 1989 shaking in the Marina District, which caused 30% of corner buildings to be red-tagged, 5% of them to collapse, and 15% to suffer total economic loss (repairs costing ≥ 60% of replacement cost).

FRAGILITY FUNCTIONS

Straight sheathing. Fragility functions were developed for straight-sheathed woodframe shearwalls using the results of laboratory tests by Trayer (1956). He reports racking tests of various contemporary woodframe wall specimens. Each specimen was approximately 9 ft high and 14 ft long, with 2x4 studs of No. 1 common well-seasoned southern yellow pine at 16 in centers with 2x4 top and bottom plates of the same material and a double stud at each end. Each plate was connected to each stud with 2 16d common nails through the plate into the ends of the studs. No vertical load was applied to the wall. To prevent overturning, holddown rods connected the top plate to the testing floor on either side of the test specimen 1 to 2 ft from the end of the specimen. The specimen was subjected to in-plane, pseudostatic loading by a force applied along the axis of the upper plate. Fifty specimens with and without openings were tested. Several specimens had horizontal straight sheathing on one side with square-edged 1x8, also of the same material, nailed with two 8d at each stud crossing. The author's only indication of a particular damage state is the point of ultimate strength, where connections begin to lose strength. This is reported to occur at a drift of approximately 3 in, or a drift ratio of 2.8%.

Also relevant to the fragility of straight-sheathed walls were tests by Elkhoraibi and Mosalam (2007a,b) who report on full-scale dynamic tests of a 2-story house and two woodframe wall specimens with straight sheathing. From sample force-deformation data, it appears that the combined specimens reach an ultimate-strength limit state near 3 inches of drift (3% drift), similar to the 2.8% drift at ultimate observed by Trayer (1956). They observed connection failure in the braces, and that deformation was accommodated by bending of the nail.

Other fragility functions. The fragility of wood lath and plaster was estimated based on laboratory tests by Trayer (1956) and in-situ cyclic racking tests of walls in an existing Los Angeles building by Schmid (1984). Fragility of masonry veneer was based in part on observations by Klingner (2004) in the 1994 Northridge and 2003 Mexico earthquakes, test results from ongoing research by Okail et al. (2008), racking tests by Thurston and Beattie (2008) of a single-story, full-scale prototype house with modern brick veneer over woodframe, and analysis by Jalil et al. (1992) of the performance of the building at 2 Alhambra St, San Francisco, in the 1989 Loma Prieta earthquake. In ongoing work for ATC-58, Deierlein (2007) proposes that 5 inches of transient drift produces an expected residual drift in woodframe buildings of 2 inches, sufficient to bring about a red tag under ATC-20 procedures. Based on Harris and Egan's (1992) estimates of spectral displacement and damage to corner apartment buildings in the Marina District in 1989, it can be estimated that at 1st-story transient drifts of approximately 13 inches, 50% of this type of building experiences collapse. See ATC (2009) for detail. From these and other observations omitted in this short work, we propose the median capacities presented in Table 4.

The equivalence between experimental observations and HAZUS damage states is ambiguous. We consulted with Sig Freeman, Chris Arnold, Bill Holmes, and Jack Moehle, and resolved on associating the HAZUS damage states with higher values of spectral displacement: slight at 0.4 to 0.5 inches, moderate at 1.5-1.9 in, extensive at 2.9-3.6 in, complete at 5-8 in, and collapse at 13 in, with the lower values being used for as-is conditions and the higher for retrofitted buildings. Logarithmic standard deviations for each damage state were copied from HAZUS-MH for W2 pre-code buildings: 1.0 for all damage states except collapse, where 0.6 was used based on examination of Harris and Egan's (1992) Marina District data. As a result of the soft story and resulting spectral shape, ground-story transient drift approximately equals the spectral displacement in the equivalent SDOF oscillator.

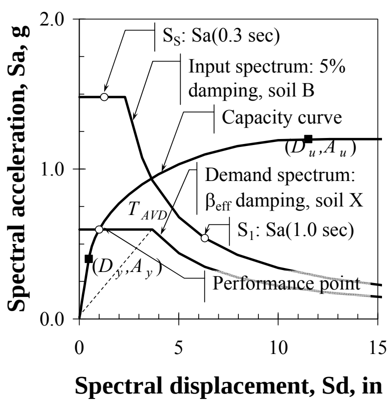
Table 4. Relationship between experimental observations and damage states

Experimental observations	*Median* S_d, in.
Small cracks appear near corners of openings in lath and plaster walls	0.06-0.07
Small cracks appear throughout the ground-floor lath and plaster walls; brick veneer begins to fall off	0.4-0.5
Ground-floor stucco delaminates from wood lath; ground-floor lath and plaster walls exhibit large cracks	1.5-1.9
Nails heavily deformed in exterior straight sheathing	2.9-3.6
Residual drift in the ground floor ≥ 2 inches; no collapse	5-8
Collapse occurs	13-?

HAZUS structural damage
Slight: Small plaster or gypsum-board cracks at corners of door and window openings and wall-ceiling intersections; small cracks in … masonry veneer. (Green tag under ATC-20.)
Moderate: Large plaster or gypsum-board cracks at corners of door and window openings; small diagonal cracks across shear wall panels exhibited by small cracks in stucco and gypsum wall panels…. (Green tag under ATC-20.)
Extensive: Permanent lateral movement of floors and roof… partial collapse of "room-over-garage" or other "soft-story" configurations…. (Yellow tag under ATC-20.)
Complete: Structure may have large permanent lateral displacement, may collapse, or be in imminent danger of collapse …. Approximately 3% of the total area of W1 buildings with Complete damage is expected to be collapsed. (Red tag under ATC-20.)

LOSS ESTIMATES AND EXPERT PANEL INTERPRETATION

We originally intended to use the HAZUS Advanced Engineering Building Module (AEBM, NIBS and FEMA 2003) to estimate structural response, damage, and loss for the soft story buildings. The methodology uses the capacity spectrum method to estimate structural response (CSM, ATC 1996). CSM is a pseudostatic nonlinear procedure that idealizes a building as a single-degree-of-freedom nonlinear damped harmonic oscillator with a 3-part pushover curve: linear up to a yield point (D_y, A_y), perfectly plastic past an ultimate point (D_u, A_u), and a portion of an ellipse between the two. The effective damping ratio is estimated as the sum of an elastic damping ratio and a fraction of the hysteretic damping, where the fraction, κ, is a function of earthquake magnitude (and indirectly, duration) to reflect pinching of the idealized hysteresis loop. Structural response is parameterized as the spectral displacement (S_d, in inches) and spectral acceleration (S_a, in g's) of the idealized oscillator. It is estimated as the point—called the performance point—where the pushover curve intersects the idealized demand spectrum with has the same effective damping ratio B_{eff}. The idealized demand spectrum has two parts: a constant-acceleration portion and a constant-velocity portion, both adjusted to account for site soil amplification (F_a in the constant-acceleration region and F_v in the constant-velocity region) and effective damping (R_A in the constant-acceleration region and R_V in the constant-velocity region). Figure 6 summarizes the methodology. Equations (1) through (7) give the equations for the demand spectrum. In Equation (6), *Area* is the area of one idealized full hysteresis loop whose upper right hand corner is the performance point, and B_E is the elastic damping ratio. See Porter (2009) for the pushover curve, especially for the ellipse portion between yield and ultimate.



$$S_a = S_S F_a / R_A \qquad 0 < T \le T_{AVD} \tag{1}$$

$$= S_1 F_v / (R_V T) \qquad T_{AVD} \le T \tag{2}$$

$$T = 0.32\sqrt{S_d / S_a} \tag{3}$$

$$R_A = 2.12 / \left(3.21 - 0.68 \ln\left[100 B_{eff}\right]\right) \tag{4}$$

$$R_V = 1.65 / \left(2.31 - 0.41 \ln\left[100 B_{eff}\right]\right) \tag{5}$$

$$B_{eff} = B_E + \kappa\left(Area / \left(2\pi S_d S_a\right)\right) \tag{6}$$

$$Area \approx 4 S_a \left(S_d - S_a / \left(A_y / D_y\right)\right) \tag{7}$$

Figure 6. Capacity spectrum method of structural analysis as intended for HAZUS

Damage is estimated by inputting structural response (S_d, S_a) into lognormal fragility functions for three generalized buildings components (structural, nonstructural drift-sensitive, and nonstructural acceleration-sensitive) and each of four qualitative damage states (slight, moderate, extensive and complete), as shown in Equation 7 for the uncertain structural damage state D_s. In the equation, Φ is the cumulative standard normal distribution, θ_i and β_i are parameters of the distribution for damage state *i*, and *d* is a particular value of D_s. Nonstructural drift-sensitive damage D_{nd} and nonstructural acceleration-sensitive damage D_{na} are estimated similarly. Repair cost for building *i* is then estimated as the sum of component damage state probabilities and mean repair cost given component damage, as shown in Equation (8). In the equation, $L_{s,d}$, $L_{nd,d}$, and $L_{na,d}$ represent the damage factor (repair cost as a fraction of total replacement cost *V*) for the structural, nonstructural drift-sensitive and nonstructural acceleration-sensitive components, respectively, in damage state *d*.

$$\begin{aligned} P\left[D_s = d \middle| S_d = x\right] &= \Phi\left(\frac{\ln\left[x/\theta_d\right]}{\beta_d}\right) - \Phi\left(\frac{\ln\left[x/\theta_{d+1}\right]}{\beta_{d+1}}\right) \quad 1 \le d \le 3 \\ &= \Phi\left(\frac{\ln\left[x/\theta_4\right]}{\beta_4}\right) \qquad d = 4 \end{aligned} \tag{7}$$

$$E\left[L_i\right] = V\left(\sum_{d=1}^{4} P\left[D_s = d \mid S_d = x\right] L_{s,d} + \sum_{d=1}^{4} P\left[D_{nd} = d \mid S_d = x\right] L_{nd,d} + \sum_{d=1}^{4} P\left[D_{na} = d \mid S_a = y\right] L_{na,d}\right) \quad (8)$$

For a portfolio of n buildings, the total number N_d of buildings in damage state d in a given scenario is calculated using Equation (9) and the total economic loss using Equation (10), which simply sum over the individual buildings i.

$$E\left[N_d\right] = \sum_{i=1}^{n} P_i\left[D_s = d \mid S_d = x_i\right] \quad (9)$$

$$E\left[L\right] = \sum_{i=1}^{n} E\left[L_i\right] \quad (10)$$

During the project it emerged that AEBM suffers from a programming flaw. In the calculation of the demand spectrum for the constant-velocity portion, the software was dividing by the larger of R_A and R_V, rather than by R_V, which is usually less than R_A. For instance, when effective damping $B_{eff} = 0.20$, S_d is low by 18%. Owing to the nonlinear nature of the damage estimate, the underestimate of loss is greater than the underestimate of response. At moderate to high excitation, most HAZUS structure types and these relatively long-period index buildings tend to have their performance point on the constant-velocity portion of the response spectrum, so the calculated spectral displacement tends to be substantially less than the one specified by the methodology, and the loss much less. Thus the programming flaw has fairly common and significant consequences. Equations 1-10 were therefore performed outside of AEBM, using a methodology described in Porter (2009), which honors all HAZUS-MH methodologies, and was independently verified by several people.

Damage and loss were evaluated for each of 4 scenarios and 4 retrofit conditions: all buildings as-is, and all buildings with retrofit 1, 2, and 3, respectively. Under as-is conditions, the scenario earthquakes are estimated to cause 50-90% of soft-story multifamily dwellings to be red-tagged or collapse, and cost \$2.5 to \$4.4 billion to repair; see bold figures in Table 5. The Project Engineering Panel focused on the collapse and red-tag rates, and felt that these figures represent an upper bound. Based on judgment, panelists estimated lower damage bounds shown in italics in Table 5.

Here, SPUR performance level E means that at least a portion of the building is likely to collapse during the hypothetical earthquake, most likely the ground story. SPUR performance level D (colored red in the table to indicate red-tagging) means that after the hypothetical earthquake, the ground story of the building would be leaning at least 2 inches, which would tend to cause building safety inspectors to post the building as unsafe to enter or occupy under the ATC-20 post-earthquake inspection procedures (ATC 1996). Under current City of San Francisco policy, these buildings would have to be repaired unless they actually collapse. SPUR performance level C (yellow tag) means that restricted use of the buildings would be allowed, such as temporary occupancy to remove personal property. SPUR performance levels A and B (green tag) means that the buildings would be labeled “Inspected” and it would therefore be lawful to occupy these buildings even if some repairs were required.

Table 5. Damage and loss to housing among 2,800 study buildings. Figures in **bold** are model results; figures in *italics* are the interpretation of the Project Engineering Panel

Scenario	Retrofit	Loss*		SPUR performance level, % of 2,800 buildings			
		$B	%	A-B	C	D	E
M6.9 Hayward Fault	As-is	$ 2.5	18%	33 – *49*	*19* – 27	*18* – 30	*6* – 18
	1	$ 1.2	9%	72 – *75*	*18* – 20	*4* – 8	*1* – 2
	2	$ 0.8	6%	84 – *86*	*10* – 11	*3* – 6	*0.2* – 0.3
	3	$ 0.6	5%	88 – *89*	*9* – 10	*2* – 3	*0.1* – 0.2
M6.5 San Andreas Fault	As-is	$ 3.0	21%	22 – *42*	*17* – 27	*23* – 39	*8* – 23
	1	$ 1.5	10%	61 – *66*	*23* – 26	*6* – 13	*2* – 4
	2	$ 1.1	8%	76 – *79*	*15* – 17	*4* – 9	*0.3* – 0.5
	3	$ 0.9	6%	79 – *81*	*15* – 16	*3* – 6	*0.2* – 0.3
M7.2 San Andreas Fault	As-is	$ 3.8	27%	6 – *35*	*9* – 23	*32* – 54	*11* – 31
	1	$ 2.3	16%	36 – *48*	*28* – 34	*14* – 28	*4* – 8
	2	$ 1.7	12%	57 – *64*	*24* – 27	*9* – 18	*0.5* – 1
	3	$ 1.3	9%	67 – *71*	*21* – 23	*6* – 12	*0.3* – 0.7
M7.9 San Andreas Fault	As-is	$ 4.4	31%	1 – *33*	*2* – 18	*37* – 62	*12* – 35
	1	$ 3.5	24%	13 – *35*	*21* – 32	*26* – 52	*7* – 14
	2	$ 7.5	53%	23 – *40*	*27* – 35	*24* – 47	*1* – 3
	3	$ 3.4	24%	28 – *44*	*26* – 33	*22* – 44	*1* – 3

* Total replacement cost ≈ $14B, excluding land

VALIDATION

Treadwell and Rollo estimated 2-4x greater shaking intensities compared with the USGS's ShakeMap for the San Francisco Marina District in the 1989 Loma Prieta earthquake. It seems reasonable that the scenarios examined here would cause greater damage on average to soft-story buildings than occurred in the Marina District in Loma Prieta. The same methodology presented above was applied to corner apartment buildings in the San Francisco Marina District in the 1989 Loma Prieta earthquake. The official USGS estimate of shaking in this areas was S_a(1.0 sec, 5%) ≈ 0.17g. According to the present study, index buildings 1 and 2 would have on average 39% probability of being in the "complete" structural damage state, including those buildings with some fraction of their floor area collapsed.

A comparison on an aggregated basis is shown in Table 6. The table shows damage experienced by corner apartment buildings on all soil profiles in the Marina District in the 1989 Loma Prieta earthquake, versus a hindcast considering only corner (index buildings 1 and 2). The 1989 "observed" figures are calculated from Harris and Egan's (1992) estimates of the damage state to 74 corner apartment buildings in the San Francisco Marina District after the 1989 Loma Prieta earthquake. Agreement is reasonable, with the figures generally agreeing within ±50%. The present study does not reflect ground failure, but the "observed" figures do not change substantially by excluding buildings in the region that had ground failure in the 1989 earthquake.

Table 6. Hindcasting damage to corner buildings in the Marina District in 1989.

HAZUS-MH structural damage	Approximate ATC-13 state	1989 estimate	1989 observed
None	None	5%	3%
Slight or moderate	Slight, light, moderate	56%	74%
Extensive or complete	Heavy, major, destroyed	39%	24%

CONCLUSIONS

A large San Francisco Bay Area earthquake is inevitable and fairly likely to occur in the next 30 years. It will produce costly damage to San Francisco's 2,800 large soft-story woodframe residential buildings. In a study for the City of San Francisco, we estimated that a M7.2 San Andreas Fault event would result in 30 to 50% of these buildings being rendered unsafe to enter or occupy, and another 10-30% collapsing. A repeat of the 1906 San Francisco earthquake would result in an estimated 40-60% being red-tagged and 10-35% collapsing. The economic losses would also be large, with repairs costing 20-30% of the buildings' total replacement cost. Seismic retrofit could substantially reduce these losses. Three retrofits were considered, from limited efforts primarily involving additional structural sheathing at ground-story interior walls, and intended to largely prevent collapse, to more ambitious efforts involving cantilever columns and more structural sheathing, intended to make the buildings occupiable during repair. The retrofits would cost between $200 and $300 million—roughly $11,000 to $17,000 per housing unit on average—but reduce economic losses by up to an estimated $2.5 billion and prevent up to 99% of collapses.

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