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APPLICATION OF EMPIRICAL FRAGILITY FUNCTIONS FROM RECENT EARTHQUAKES

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SUMMARY

This paper is a companion to the paper, "Empirical Fragility Functions from Recent Earthquakes," by Sarabandi et al. [1], which focuses on the model development, while this paper focuses on the application of the developed models. The models, empirically derived fragility functions, were developed from a study of the correlation of building performance with recorded ground motion. The dataset includes buildings within 1000 feet (300 meters) of recording stations that were surveyed by licensed engineers following the 1994 Northridge, California and the 1999 Chi-Chi, Taiwan earthquakes. The fragility functions, developed for four distinct building construction types and for several time- and frequency-dependent measured ground motion parameters, are used to estimate regional and site-specific earthquake damage and loss. The regional damage and loss estimates are computed using the HAZUS99 [2] software. The default fragility functions in the software are replaced by the empirically-derived functions. Results indicate that, on an aggregated regional basis, the total loss estimates are quite similar. The losses for individual building construction types differ, but when averaged over a distributed inventory, the differences cancel out. The site-specific damage and loss estimates are computed for a hypothetical wood frame dwelling located in southern California, a region of high seismicity. The estimates are used to compare the damage and loss produced using the various fragility functions that are conditional on different ground motion parameters, where the parameters are computed from the same simulated ground motion for the site. Results indicate that, for the majority of the ground motion parameters, the damage and loss estimates produced with the different fragility functions are relatively similar. This paper explores possible reasons for the damage and loss results, as well as the limitations in applications of the developed fragility functions.

INTRODUCTION

Relationships between building performance and ground motion form the core of earthquake loss estimation methodologies, and are also used for structural analysis studies and in the design code formulation process [3]. Current motion-damage relationships are based primarily on models developed

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from expert opinion, such as ATC-13 [4], or models that combine analytical results with expert opinion, such as HAZUS99 [2]. Attempts have been made to update the published motion-damage relationships with empirical data collected after damaging earthquakes [5,6,7]. Small improvements have been made, but in most cases, progress in model development has been hampered by the lack of useful empirical data on building performance.

The majority of the empirical building performance datasets that have been collected following significant earthquakes are deficient in some respect, which limits their use for developing relationships that correlate building performance to ground motion [3]. Some of the deficiencies in these datasets include:

- Relatively small number of data points;
- Bias towards damaged or noteworthy buildings;
- Focus only on a few select building types;
- Not collected in a consistent and complete manner by experienced engineers;
- Collected and held by private companies (not publicly available); and
- Buildings not located in the vicinity of free-field recording instruments.

In recent years, there have been selected efforts undertaken to remedy the lack of useful empirical building performance data [3]. For example, following the 1994 Northridge earthquake, an effort was made to systematically document the effects of earthquake shaking on structures adjacent to locations of strong ground motion recordings. The ATC-38 Project [8] involved the inspection of more than 500 buildings located near (within 1000 feet of) 30 strong motion recording stations. The resulting database of building characteristic and performance documentation, photos, and strong motion recordings provides a wealth of information for developing new motion-damage relationships based on non-proprietary empirical data. A similar dataset was also developed following the 1999 Chi-Chi, Taiwan earthquake by Degenkolb Engineers [9].

The availability of comprehensive empirical data on the performance of buildings in earthquakes is one essential part of developing motion-damage relationships [3]. A second essential part is a clear and systematic method for combining those empirical data with the associated recorded ground motion parameters to produce fragility curves and damage probability matrices that can be used in earthquake loss estimation methodologies, structural studies, and in design code formulation. Examples of published material on fragility curve development include Singhal and Kiremidjian [10], who present a method for developing fragility curved using simulated ground motion parameters with analytically-derived building performance data, and Başöz and Kiremidjian [11], who use documented bridge damage and repair cost data with recorded earthquake motions to develop empirical fragility curves for several classes of highway bridges.

The purpose of the project described in this paper is to develop motion-damage relationships based on the correlation of observed building performance with measured ground motion parameters and illustrate their use in regional and site-specific earthquake loss estimation applications. This recently-completed project was funded by the California Department of Conservation, Division of Mines and Geology, Strong Motion Instrumentation Program. Most of the information contained in this paper, as well as that in the companion paper by Sarabandi et al. [1], is taken from the final report of the project by King et al. [3,12]. The papers by Sarabandi et al. [1,13] focus on the data collection and development of the motion-damage relationships, thus these aspects of the project are only briefly summarized in the following two sections. The remainder of this paper focuses on the utilization of the developed relationships for damage and loss estimation applications.

DATA COLLECTION

The empirical building performance datasets collected for use in the project are:

- ATC-38 – Performance of Structures Near Strong-Motion Recordings in the 1994 Northridge, California Earthquake [8]
- LADiv88 – Performance of Rehabilitated Unreinforced Masonry Buildings (retrofitted according to Los Angeles Division 88 standards) in the 1994 Northridge, California Earthquake developed by Rutherford & Chekene Consulting Engineers [7]
- SAC – Performance of Steel Moment Frame Buildings in the 1994 Northridge, California Earthquake [6]
- Chi-Chi – Performance of Reinforced Concrete Buildings Near Strong-Motion Recordings in the 1999 Chi-Chi, Taiwan Earthquake developed by Degenkolb Engineers [9]

The ATC-38 and Chi-Chi datasets include the strong ground motion data recorded within 1000 feet of each building. Buildings from the SAC and LADiv88 datasets with strong ground motion data recorded within 1000 feet were extracted through GIS (geographic information system) analysis. Sources for strong ground motion recorded in the vicinity of the SAC and LADiv88 building sub-sets are:

- COSMOS – Consortium of Organizations for Strong-Motion Observation Systems Virtual Data Center, which contains links to strong-ground motion from the California Geological Survey, the U.S. Geological Survey, the U.S. Bureau of Reclamation, the U.S. Army Corps of Engineers, and others [14]
- PEER – Pacific Earthquake Engineering Research Center Strong Motion Database [15]
- NGDC – National Geophysical Data Center Earthquake Strong Motion Database [16]

MODEL DEVELOPMENT

Ground Motion Analysis

The parameters that were computed from each time history record (maximum of two horizontal components, average of two horizontal components, and vertical) are:

- Peak Ground Acceleration (PGA)
- Peak Ground Velocity (PGV)
- Peak Ground Displacement (PGD)
- ShakeMap Instrumental Intensity (I_{mm}) from Wald et al. [17]
- Duration
- Root Mean Square Acceleration (a_{RMS})
- Arias Intensity (A_I)

The parameters that were computed from each response spectrum (maximum of two horizontal components, average of two horizontal components, and vertical) are:

- Acceleration Spectrum Intensity (ASI) from Von Thum et al. [18]
- Effective Peak Acceleration (EPA) from ATC-3-06 [19]
- Effective Peak Velocity (EPV) from ATC-3-06 [19]
- Housner Intensity (S_I) [20]
- Spectral Acceleration (S_a) at several periods
- Spectral Velocity (S_v) at several periods
- Spectral Displacement (S_d) at several periods

Other parameters not computed, but assigned through GIS analysis include:

- Modified Mercalli Intensity (MMI)
- NEHRP Site Classification [21]

Building Response Analysis

Building response datasets were analyzed for two purposes – to group the buildings into similar structural classes and to interpret the damage survey information. The grouping of buildings by structural class follows the FEMA 310 [22] model building type classification. The damage survey information for each building is translated into four characterizations of performance: (1) ATC-13 damage states [4], (2) HAZUS99 damage states [2], (3) Vision 2000 performance levels [23], and (4) FEMA 273/356 performance levels [21]. In addition, for each building, the design code year, the fundamental period, the design base shear, the roof drift ration, and the maximum interstory drift ratio [24] were determined and added to the database attributes.

Motion-Damage Relationship Development

The first step in the model development is the identification of strong correlations between building performance and measured ground motion parameters. Empirical damage probability matrices were developed for all building performance descriptors and the corresponding ground motion or building demand parameters. Damage probability matrices (DPMs) show the conditional probability of being in a discrete damage state or performance level as a function of the input ground motion or building demand level, which can be a discrete value (e.g., MMI) or a range of values (e.g., PGA). For the areas of strong correlation, fragility curves were developed in the form of lognormal probability distributions following the method outlined in Singhal and Kiremidjian [10]. Fragility curves show the conditional probability of being equal to or exceeding a given damage state or performance level as a function of the ground motion or building demand parameter. Final DPMs were derived from the fragility functions by discretizing the continuous distributions. Figure 1 illustrates the relationship between DPMs, probability distributions, and fragility curves.

Motion-damage relationships were developed for wood frame, steel frame, and concrete frame buildings using data from the 1994 Northridge earthquake and the 1999 Chi-Chi, Taiwan earthquake. The final report for this project [3] includes the complete set of motion-damage relationships with lognormal fragility parameters and curves, and the papers by Sarabandi et al. [1,13] discuss the results for concrete frame and steel frame buildings, respectively. Fragility curve results presented here are limited to one building type for comparison with HAZUS99 fragility curves and ATC-13 damage probability matrices.

Fragility Curves

Figure 2 shows a comparison of the lognormal fragility curves conditional on spectral displacement for wood frame buildings published in HAZUS99 (Moderate Code W1) and as computed in this project. It can be seen in Figure 2 that for the fragility curves developed in the project, the differences between the various damage states are small, while the HAZUS99 curves for the various damage states are quite distinct. One possible explanation for this observation is that the HAZUS99 fragility curves were developed based on analysis of one model building of the same structural type. Hence the performance of the particular building population of the same class is not uniform and for the close values of spectral displacement there are buildings in several damage states. Another source of difference between the HAZUS99 fragility curves and those developed in the project is that the empirical data tend to be concentrated at the lower values of spectral displacement and in the lower damage states. For the curves representing higher levels of damage, only a small number of data points were used in the analysis, thus the parameters should be used with caution. Note also that the fragility curves in Figure 2a actually cross at a spectral acceleration value of about 0.9 inches, thus they should not be used beyond this level of displacement demand.

DAMAGE PROBABILITY MATRIX					
	Probability of being in a given damage state as a function of range in PGA for a given building type				
Damage State	PGA 0-0.2g	PGA 0.2-0.4g	PGA 0.4-0.6g	PGA 0.6-0.8g	PGA 0.8-1.0g
1, <2% loss	90	80	60	25	20
2, 2-10% loss	10	15	20	40	30
3, 10-30% loss	0	5	15	20	30
4, 30-60% loss	0	0	5	10	15
5, >60% loss	0	0	0	5	5

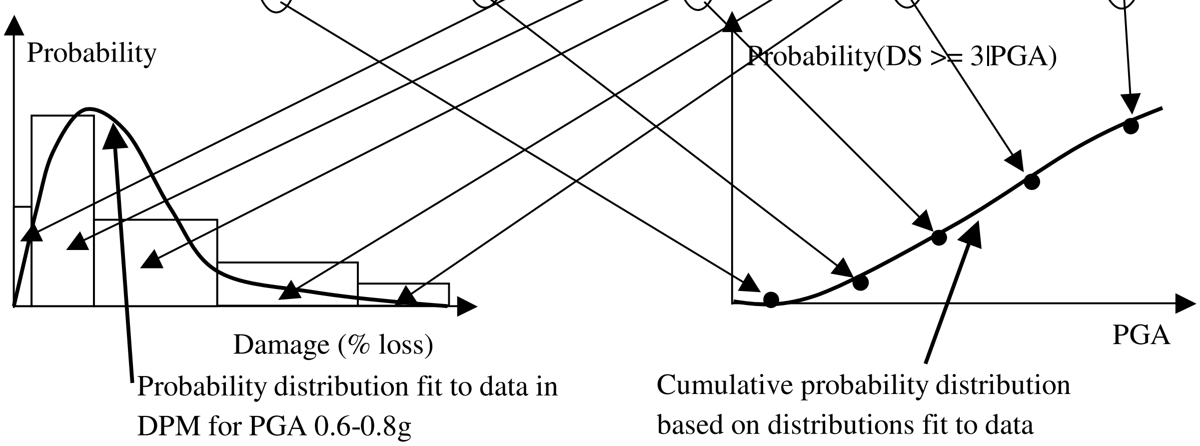
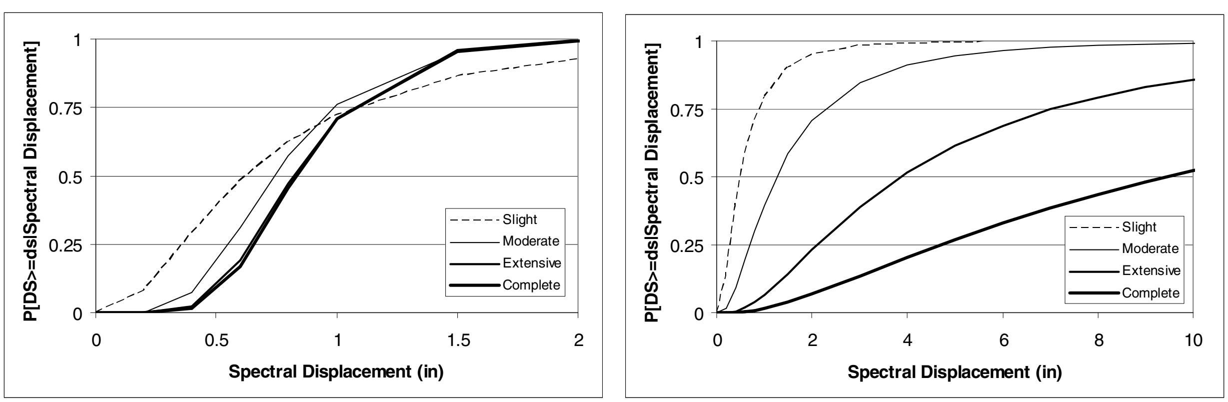


Figure 1 Illustration of relationship among damage probability matrix, probability distribution fit, and fragility curve for hypothetical data.



(a) **(b)**

Figure 2 Fragility curves for wood frame (W1) buildings, (a) computed in the project and (b) from HAZUS99 [2].

Damage Probability Matrices

Figure 3 shows the comparison between the damage probability matrix (DPM) conditional on MMI developed in the project for wood frame buildings with that published in ATC-13 for low-rise wood frame buildings (class 1). As shown in Figure 3, the two DPMs are quite different. The ATC-13 DPM, developed by fitting Beta distributions to expert opinion data, shows a significant increase in the probabilities of being in higher damage states for higher levels of MMI. Although the empirically-derived

DPM (derived from the lognormal fragility curves developed in the project) also shows as increase, it is very gradual. Most of the data points are at MMI levels of IX or lower, thus the probabilities associated with MMI X or XI should be used with caution. Note also that the ATC-13 DPM reflects a much narrower probability distribution on damage at each MMI level.

Damage State	**Modified Mercalli Intensity**					
	VI	**VII**	**VIII**	**IX**	**X**	**XI**
1-None	0.817	0.787	0.760	0.734	0.709	0.687
2-Slight	0.134	0.148	0.159	0.168	0.175	0.180
3-Light	0.030	0.037	0.043	0.048	0.053	0.057
4-Moderate	0.010	0.013	0.016	0.019	0.022	0.024
5-Heavy	0.004	0.006	0.008	0.009	0.011	0.013
6-Major	0.001	0.002	0.002	0.003	0.004	0.004
7-Destroyed	0.004	0.008	0.013	0.019	0.027	0.036

(a)

Damage State	**Modified Mercalli Intensity**					
	VI	**VII**	**VIII**	**IX**	**X**	**XI**
1-None	0.037	~ 0	~ 0	~ 0	~ 0	~ 0
2-Slight	0.685	0.268	0.016	~ 0	~ 0	~ 0
3-Light	0.278	0.732	0.949	0.624	0.115	0.018
4-Moderate	~ 0	~ 0	0.035	0.376	0.760	0.751
5-Heavy	~ 0	~ 0	~ 0	~ 0	0.125	0.231
6-Major	~ 0	~ 0	~ 0	~ 0	~ 0	~ 0
7-Destroyed	~ 0	~ 0	~ 0	~ 0	~ 0	~ 0

(b)

Figure 3 Damage Probability Matrix for wood frame (W1) buildings, (a) computed in the project and (b) from ATC-13 [4].

MODEL APPLICATION

Relationships between building performance and strong ground motion are most commonly used for regional and site-specific earthquake damage and loss estimation, with the resulting estimates providing information for purposes such as emergency response planning, probabilistic risk assessment, and performance-based design. A few of the relationships developed in this project are discussed above and in Sarabandi et al. [1,13]; however, based on this information alone, it is not possible to assess the quality and potential use of the motion damage-relationships. A more meaningful assessment is based on the results of the application of the relationships, i.e., the resulting regional and site-specific damage and loss estimates.

Regional Loss Estimation

The HAZUS99 [2] software is used to assess the motion-damage relationships developed in the project. The study region is Los Angeles County, California. The analysis is run using the ShakeMap [25] developed for the M 6.7 1994 Northridge, California earthquake. The default HAZUS99 fragility parameters are first used, then the fragility parameters developed in this project for the W1, W2, S1, C1, and C2 building classes are used to replace the default values for the corresponding building classes in the HAZUS99 software. The replacement procedure follows that outlined in Porter et al. [26]. The results of the HAZUS99 analysis using the details and replaced fragility parameters with the 1994 Northridge ShakeMap are given in Table 1, which compares the number of buildings in each damage state by general structural class. In general, the number of buildings in the damage states of *None* and *Complete* increase significantly, while the number of buildings in the *Slight*, *Moderate*, and *Extensive* damage states decrease. The wood frame buildings show results that are similar to the total building inventory, as would be expected since they make up approximately 92% of the inventory. For concrete frame buildings, the numbers in the *None* and *Slight* damage states change very little, but there is a significant shift in the number of buildings from the *Extensive* and *Complete* damage states to the *Moderate* damage state. For the steel frame buildings, the number of buildings in the *None* damage state increases while in the other damage states the number of buildings decreases.

HAZUS99-generated structural, nonstructural, and total building losses are compared in Table 2 by general structural class. For the three building classes with default fragility parameters replaced with those developed in the project, the losses decrease by more than 10% for the structural loss. This is consistent with the increase in the number of building in the *None* damage state. Nonstructural loss does not change because nonstructural fragility parameters were not developed in the project. As shown in Table 2, the decrease in total building loss is almost insignificant (from \$16.93B to \$16.52B, or 2.4%) due to the fact that the nonstructural loss (which remains constant) comprises more than 80% of the total building loss. In the HAZUS99 software, replacement values for nonstructural components are typically 70 to 80% of the total replacement value of the building.

Site-Specific Loss Estimation

The use of the fragility curves for other ground motion parameters and other damage or performance characterization is illustrated through site-specific damage and loss estimation. Motion-damage relationships, regardless of the method used to develop them, are typically intended to represent the average behavior, with uncertainty, of a group of buildings of similar type that are subjected to the same ground motion. The user needs to be aware of the limitations in applying these relationships to a single building, where the uncertainty on the performance of an individual facility can be greater than the uncertainty on the performance of a group of similar facilities. Further discussion of uncertainties is beyond the scope of the project and this paper; thus results are presented as expected values.

The motion-damage relationships are used to estimate damage and loss to a hypothetical single-story wood frame building (class W1) located in southern California. The purpose of this application is not only to illustrate the use of the motion-damage relationships for site-specific loss estimation, but also to compare and assess the reasonableness of the damage and loss results obtained using the various parameters from a single ground motion record. The ground motion parameters are based on the probabilistic hazard for the site, obtained from the U.S. Geological Survey National Seismic Hazard Mapping Program website [27]. The time-dependent and frequency-dependent ground motion parameters are computed following the same procedure as for the recorded ground motion used in the project and described in [1,3,12,13]. These parameters are listed in Table 3 for two seismic hazard levels – 10% probability exceedance in 50 years (475-year return period) and 2% probability of occurrence in 50 years (2475-year return period). Table 4 lists the expected damage, in terms of percent loss, for a W1 building for each characterization of performance (i.e., ATC-13, HAZUS99, FEMA 273/356, and Vision 2000) for each 10% in 50-year hazard ground motion parameter for which reasonable lognormal fragility curves could be developed.

Table 1 HAZUS99 Results: Number of Buildings in Each Damage State by General Structural Class for Los Angeles County and 1994 Northridge Earthquake ShakeMap Using (a) Default Fragility Parameters and (b) Fragility Parameters Developed in the Project

(a)

General Structural Class	Number of Buildings by HAZUS99 Damage State					TOTAL
	None	Slight	Moderate	Extensive	Complete	
Concrete	12,763	2,987	2,048	613	105	**18,516**
Mobile Home	32,814	8,802	8,394	3,814	1,566	**55,390**
Precast	11,193	2,216	2,440	745	162	**16,756**
Reinforced Masonry	26,664	4,837	4,850	1,801	303	**38,455**
Steel	13,542	1,918	2,324	747	113	**18,644**
URM	3,309	1,181	1,059	409	209	**6,167**
Wood	1,216,291	410,652	153,587	16,945	4,946	**1,802,421**
TOTAL	**1,316,576**	**432,593**	**174,702**	**25,074**	**7,404**	**1,956,349**

(b)

General Structural Class	Number of Buildings by HAZUS99 Damage State (% Change from Results Using Default Fragility Parameters)					TOTAL[1]
	None	Slight	Moderate	Extensive	Complete	
Concrete	12,732 (-0.2)	2,922 (-2.2)	2,832 (38.3)	43 (-93.0)	11 (-89.5)	**18,540 (0.1)**
Mobile Home	32,814 (0.0)	8,802 (0.0)	8,394 (0.0)	3,814 (0.0)	1,566 (0.0)	**55,390 (0.0)**
Precast	11,193 (0.0)	2,216 (0.0)	2,440 (0.0)	745 (0.0)	162 (0.0)	**16,756 (0.0)**
Reinforced Masonry	26,664 (0.0)	4,837 (0.0)	4,850 (0.0)	1,801 (0.0)	303 (0.0)	**38,455 (0.0)**
Steel	15,166 (12.0)	1,195 (-37.7)	1,635 (-29.6)	552 (-26.1)	112 (-0.9)	**18,660 (0.1)**
URM	3,309 (0.0)	1,181 (0.0)	1,059 (0.0)	409 (0.0)	209 (0.0)	**6,167 (0.0)**
Wood	1,696,471 (39.5)	75,427 (-81.6)	12,269 (-92.0)	2,551 (-84.9)	16,904 (241.8)	**1,803,622 (0.1)**
TOTAL	**1,798,349 (36.6)**	**96,580 (-77.7)**	**33,479 (-80.8)**	**9,915 (-60.5)**	**19,267 (160.2)**	**1,957,590 (0.1)**

[1] Changes in total number of buildings are due to round-off error in HAZUS99 software

The results in Table 4 show that, for the most part, the expected damage using the ATC-13 damage state characterization is slightly higher than for the other characterizations. The expected damage is in the range of 2-3% for the ATC-13 damage state characterization, in the range of 1-2% for the HAZUS99 and Vision 2000 characterizations, and less than 1% for the FEMA 273/356 characterization. One possible explanation for this variation is that ATC-13 uses seven damage states, HAZUS99 and Vision 2000 use five, while FEMA 273/356 uses four. Each damage state has an associated mean percent loss, thus the higher damage states in ATC-13 (with mean percent loss values of 45%, 80%, and 100%) will contribute to higher expected values.

Table 2 HAZUS99 Results: Building Loss by General Structural Class for Los Angeles County and 1994 Northridge Earthquake ShakeMap Using (a) Default Fragility Parameters and (b) Fragility Parameters Developed in the Project

(a)

General Structural Class	Loss in $×1000		
	Structural	Nonstructural	Total Building
Concrete	321,441	1,185,176	**1,506,617**
Mobile Home	51,753	124,631	**176,384**
Precast	344,032	870,492	**1,214,524**
Reinforced Masonry	354,523	1,271,606	**1,626,129**
Steel	331,943	1,070,631	**1,402,574**
URM	152,077	456,155	**608,232**
Wood	1,419,668	8,974,569	**10,394,237**
TOTAL	**2,975,437**	**13,953,260**	**16,928,697**

(b)

General Structural Class	Loss in $×1000 (% Change from Results Using Default Fragility Parameters)		
	Structural	Nonstructural	Total Building
Concrete	141,978 (-55.8)	1,185,176 (0.0)	**1,327,154 (-11.9)**
Mobile Home	51,822 (0.0)	124,631 (0.0)	**176,453 (0.0)**
Precast	344,031 (0.0)	870,492 (0.0)	**1,214,523 (0.0)**
Reinforced Masonry	354,527 (0.0)	1,271,606 (0.0)	**1,626,133 (0.0)**
Steel	258,899 (-22.0)	1,070,631 (0.0)	**1,329,530 (-5.2)**
URM	152,227 (0.0)	456,155 (0.0)	**608,382 (0.0)**
Wood	1,265,557 (-10.9)	8,974,569 (0.0)	**10,240,126 (-1.5)**
TOTAL	**2,569,041 (-13.7)**	**13,953,260 (0.0)**	**16,522,301 (-2.4)**

A comparison of the results in Table 4 across ground motion parameters (rather than across the four performance characterizations as in the previous paragraph) shows that similar damage estimates are produced using the relationships developed for nearly all of the ground motion parameters except PGA and MMI. The motion-damage relationships based on PGA and MMI produce higher estimates of damage. For reference, for this building and the 10% in 50-year hazard level, the expected damage based on the damage probability matrix published in ATC-13 is 9.2% and that computed using the HAZUS99 method and data is 8.3%, both of which are higher than most of the values reported in Table 4.

Table 3 Ground Motion Parameters Computed from Site-Specific Acceleration Data

Parameter	10% in 50-year Hazard Level	2% in 50-year Hazard Level
Peak Ground Acceleration (g)	0.74	1.18
Peak Ground Velocity (cm/sec)	43.1	127.3
Peak Ground Displacement (cm)	10.3	59.2
Total Record Duration (sec)	64	64
90% Cumulative Duration (sec)	7.0	6.0
Bracketed Duration (sec)	12.8	13.2
Root Mean Acceleration for Total Duration (g)	0.06	0.11
Root Mean Acceleration for 90% Duration (g)	0.18	0.33
Root Mean Acceleration for Bracketed Duration (g)	0.14	0.24
Arias Intensity (cm/sec)	409.3	1140.7
Acceleration Spectrum Intensity (g)	0.50	0.84
Effective Peak Acceleration (g)	0.50	0.83
Effective Peak Velocity (cm/sec)	40.8	73.4
Response Spectrum or Housner Intensity (cm/sec)	227.6	405.7
Modified Mercalli Intensity[1]	IX	X
ShakeMap Instrumental Intensity	8.0	9.7
Roof Drift Ratio (%)	0.17	0.29
Maximum Interstory Drift Ratio (%)	0.21	0.35
Spectral Displacement at Predominant Period[2] (cm)	0.63	1.05
Spectral Velocity at Predominant Period[2] (cm/sec)	27.9	46.3
Spectral Acceleration at Predominant Period[2] (g)	1.26	2.09

[1] Computed using formula from Trifunac and Brady [28], with rounding to nearest integer
[2] Predominant period for one-story wood frame building estimated as 0.14 sec.

Damage estimates for the 2% in 50-year hazard level are not shown here due to space limitations. The results for the 2% in 50-year hazard level are much less consistent than those for the 10% in 50-year hazard level shown in Table 4, both across the four performance characterizations and across the various ground motion parameters. Consistency in results shows that the motion-damage relationships produce similar expected values. Consistency does not directly indicate whether or not these expected values are a valid representation of reality, but it does increase the user's degree of confidence in their validity. A possible explanation for the lack in consistency in damage estimates for the 2% in 50-year hazard is that the motion-damage relationships developed in the project are based primarily on data from the 1994 Northridge earthquake, a relatively moderate event with shaking similar to the 10% in 50-year hazard level rather than the more rare 2% in 50-year hazard level. This emphasizes the point that the motion-damage relationships developed in this project are limited in application to moderate levels of ground motion; extrapolation beyond that point should be done with caution.

It should be noted that it would be interesting to compare the correlations among damage estimates based on the various ground motion parameters (e.g., Table 4) with the correlations among the actual ground motion parameters themselves. Several researchers have looked at correlations among ground motion parameters, for instance Naeim and Anderson [29]. Such a comparison is beyond the scope of this project, but it is a good example of future research that can be conducted with the extensive building performance and ground motion database that has been compiled over the course of this effort.

Table 4 Expected Damage for Example Site-Specific Analysis of Single-Story W1 Building for 10% in 50-year Ground Motion Hazard

Parameter	Expected Damage in Percent Loss by Damage or Performance Characterization Type			
	ATC-13	HAZUS99	FEMA 273/356	Vision 2000
Peak Ground Acceleration (g)	9.5	8.6	0.6	81.4
Peak Ground Velocity (cm/sec)	2.8	1.0	0.5	1.1
Peak Ground Displacement (cm)	5.7	1.0	0.5	1.0
Total Record Duration (sec)	NA	NA	NA	NA
90% Cumulative Duration (sec)	NA	NA	NA	NA
Bracketed Duration (sec)	NA	1.3	0.6	1.3
Root Mean Acceleration for Total Duration (g)	3.9	NA	0.8	1.2
Root Mean Acceleration for 90% Duration (g)	NA	NA	NA	NA
Root Mean Acceleration for Bracketed Duration (g)	NA	NA	NA	NA
Arias Intensity (cm/sec)	NA	NA	NA	NA
Acceleration Spectrum Intensity (g)	NA	NA	NA	NA
Effective Peak Acceleration (g)	NA	NA	NA	NA
Effective Peak Velocity (cm/sec)	2.0	NA	NA	NA
Response Spectrum or Housner Intensity (cm/sec)	NA	NA	1.2	1.8
Modified Mercalli Intensity	13.4	NA	NA	NA
ShakeMap Instrumental Intensity	2.4	1.0	0.5	NA
Roof Drift Ratio (%)	3.7	2.4	1.7	2.0
Maximum Interstory Drift Ratio (%)	2.4	1.0	0.5	1.0
Spectral Displacement at Predominant Period (cm)	3.2	1.6	0.8	1.6
Spectral Velocity at Predominant Period (cm/sec)	3.0	1.5	0.8	1.5
Spectral Acceleration at Predominant Period (g)	2.5	2.0	1.3	2.0

Note: NA indicates that reasonable fragility curve parameters could not be found.

CONCLUSIONS

Motion-damage relationships in the form of lognormal fragility curves and corresponding damage probability matrices have been developed from observed building performance data and recorded ground motion within 1000 feet of the buildings. The relationships are for wood frame, steel frame, and concrete frame buildings, for damage characterized by ATC-13 and HAZUS99 damage states and FEMA 273/356 and Vision 2000 performance levels, and for several ground motion and building demand parameters. A comparison to the ATC-13 and HAZUS99 published damage models shows that the models developed in the project are quite different. The difference is due primarily to the characteristics of the data used in the model development – there is a bias towards lower levels of ground motion and lower levels of damage. Despite the differences in the models themselves, when applied to regional loss estimation via the HAZUS99 software, the total losses for the study region are similar to those computed with the default fragility curve data. For site-specific applications, the results show that similar losses are produced using different ground motion parameters, and that damage or performance characterization has an influence on the loss values.

The project discussed in this paper and in other publications by the same authors [1,3,12,13] utilized a systematic and rigorous method for developing motion-damage relationships from databases of observed building performance and nearby recorded strong ground motion. Although several relationships were developed in the project, the number of building types for which relationships could be developed was limited due to the lack of useful building performance datasets for several types of buildings. In addition, the range of strong ground motion and building demand parameters over which the relationships should be used is limited due to the lack of datasets corresponding to high levels of ground motion. It is hoped that these problems will be remedied by accurate and complete collection of performance data following future seismic events. Utilizing the methods outlined in this project, the developed motion-damage relationships can be updated when new data become available, and additional relationships can be developed for other model building types.

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