A Methodology for Evaluating Component-Level Loss Predictions of the FEMA P-58 Seismic Performance Assessment Procedure

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As performance-based earthquake engineering (FEMA P-58) becomes more widely adopted in design and risk analysis practice, it is important to understand the degree to which the calculations reflect reality. This paper proposes a methodology for evaluating P-58 component-level loss predictions across buildings subjected to given seismic events, which involves ranking P-58 loss predictions according to categorical component damage information recorded on post-earthquake damage surveys. The methodology explicitly incorporates uncertainties in predictions, and utilizes a ground shaking benchmark to determine whether P-58 analyses provide more insight into damage than variations in ground shaking between buildings. Two example applications of the methodology are provided, involving non-structural component data from the 2011 M_w 6.1 Christchurch Earthquake, for which there is negligible variation in shaking between buildings, and the 1994 M_w 6.7 Northridge Earthquake, for which there is notable variation in shaking between buildings. We find that P-58 non-structural component-level loss predictions perform better overall than the ground shaking benchmark in both cases. The methodology offers an understanding of how P-58 component-level loss predictions align with actual observed damage.

INTRODUCTION

MOTIVATION

FEMA P-58 is a seismic performance assessment methodology for individual buildings (FEMA, 2012a) that follows the performance-based earthquake engineering philosophy (Moehle and Deierlein, 2004). It has been in development for many years (e.g. Cornell and Krawinkler,

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2000; Porter and Kiremidjian, 2000), and has in many ways revolutionized the thinking about acceptable performance of buildings in earthquakes. The methodology combines ground motion hazard and structural response to make predictions of component-level damage and its associated consequences, which are defined in terms of repair costs, repair time, casualties, and building tagging. Monte Carlo sampling is employed at each stage in the analysis, reflecting the substantial uncertainty associated with seismic performance prediction. The consequences predicted can be compared with the performance objectives set by building stakeholders to determine if they are acceptable. As FEMA P-58 becomes more prevalent in the seismic design and evaluation of buildings worldwide, it is important to understand the degree to which the calculations reflect reality. To date, however, few studies have evaluated how well P-58 loss or damage predictions align with observations.

The purpose of this study is to develop a methodology for evaluating FEMA P-58 component-level loss predictions across a group of buildings subjected to a given seismic event, using component-level damage information collected during post-earthquake reconnaissance efforts. While post-earthquake reconnaissance data are categorical in nature and cannot be used to directly assess quantitative loss predictions, they are typically far more accessible than information on actual dollar losses in the aftermath of seismic events, which makes our methodology broadly applicable. The proposed evaluation involves ranking P-58 component-level loss predictions according to the severity of component damage recorded in post-earthquake surveys. Our methodology has several notable features:

1. It uses a novel loss ratio metric that enables comparisons of P-58 numerical component-level predicted losses with categorical component damage reconnaissance data.
2. It uses rank-order statistical tests to evaluate these comparisons. These statistical tests are designed to handle both numerical and categorical data, and allow consideration of the uncertainties in P-58 loss predictions.
3. It utilizes ground shaking intensity as a benchmark to determine if P-58 loss predictions, obtained using knowledge of building properties, provide more insight into damage than variations in shaking intensity from building to building (which are obviously important contributors to damage).

Two example applications of the methodology will be carried out, to evaluate non-structural component-level P-58 loss predictions for two seismic events with different levels of variation in ground shaking between buildings.

PREVIOUS LOSS EVALUATION EFFORTS

Numerous studies have been undertaken in the past to evaluate the typical features of a loss assessment framework on a regional basis, using observations from different locations and seismic events. For example, Ordaz and Reyes (1999) compared predicted hazard curves with empirical estimates for Mexico City, Booth et al. (2011) validated assessments of damage made from remote sensing following the 2010 Haitian earthquake, and Spence et al. (2003) compared predicted and observed regional losses for the 1999 Kocaeli earthquake in Turkey. We now focus our discussion on such evaluation studies that used data from either the 1994 M_W 6.7 Northridge earthquake or the 2010-2011 Canterbury earthquake sequence, as data from these events will be used in the example applications of our methodology.

As part of the development of the HAZUS methodology (Whitman et al., 1997; Kircher et al., 1997b,a), loss functions were calibrated by comparing predicted loss with observed loss due to previous earthquakes, including the Northridge earthquake. For the Northridge earthquake, predictions of damage and loss were based on response spectra of ground shaking records, with representative ground response spectra being developed for each of the five regions of Modified Mercalli Intensity (MMI) shaking levels that comprised the Los Angeles study area. Observed loss was estimated based on a sample of insurance coverage and claims paid. The comparisons made for each earthquake either verified that the methodology's building loss functions could reasonably replicate observed impacts, or in certain cases, loss functions were revised to achieve better correlation between predicted and observed losses.

Olshansky (1997) examined the effectiveness of previously published seismic hazard maps in predicting the damage caused by the Northridge earthquake. Observed data used comprised of damage measure data (red tags, yellow tags, and pipe breaks), mapped geologic data, and census data. The study found that seismic hazard maps at the quality level of the 1985 USGS maps for LA can improve the prediction of both the amount and location of future damages.

Lin et al. (2012) assessed the reliability of the loss estimation platform MAEviz, using data from the Canterbury earthquake sequence. Strong motion data from the earthquakes were used to create a hazard map, which was input to MAEviz to benchmark the method's fragility curves against observed damage of reinforced concrete buildings in the form of building tagging levels (i.e. green, yellow, or red). It was found that MAEviz overestimated the number of yellow-tagged buildings, and significantly underestimated the number of red-tagged buildings.

Our work is substantially different from the aforementioned studies, as we attempt to benchmark loss predictions for individual buildings rather than regions. Some efforts have already been made to compare predicted and observed losses at the individual building level (Baker et al., 2016; Del Vecchio et al., 2018). However, these studies used actual repair cost data, while we evaluate component-level loss predictions using more widely available categorical post-earthquake reconnaissance data in this study.

OVERVIEW OF THE PROPOSED METHODOLOGY

The methodology is intended to evaluate component-level loss predictions of the FEMA P-58 assessment procedure across a group of individual buildings subjected to a given seismic event, using post-earthquake survey damage data. Damage for similar components is usually grouped and reported collectively in such surveys. The extent of damage for component groupings is typically expressed in terms of three or four possible categories. The loss metrics predicted by FEMA P-58 are typically not compatible with these data, which makes direct comparisons between observations and predictions challenging. In this study, we use the categorical component damage information recorded to instead evaluate whether the P-58 component-level loss predictions can order the buildings according to the component damage severity. The probabilistic nature of P-58 predictions complicates this evaluation.

A FEMA P-58 analysis for the seismic event of interest is first conducted for every building in the group. Component-level P-58 loss predictions are then compared with post-earthquake rapid assessment damage data collected in the event, using rank-order statistical tests designed to handle both numeric and categorical data. These comparisons are benchmarked against the comparison of ground shaking intensity with the observed damage data. A summary of the methodology is presented in Figure 1.

APPLICATIONS

We apply the methodology to groups of buildings that were surveyed following two seismic events. The first event is the 2011 M_w 6.1 Christchurch earthquake in New Zealand, for which there is negligible variation in ground shaking between examined buildings. The second event is the 1994 M_w 6.7 Northridge earthquake in California, for which there is notable variation in ground shaking between examined buildings.

We examine 95 buildings for the Christchurch event in the Christchurch Central Business

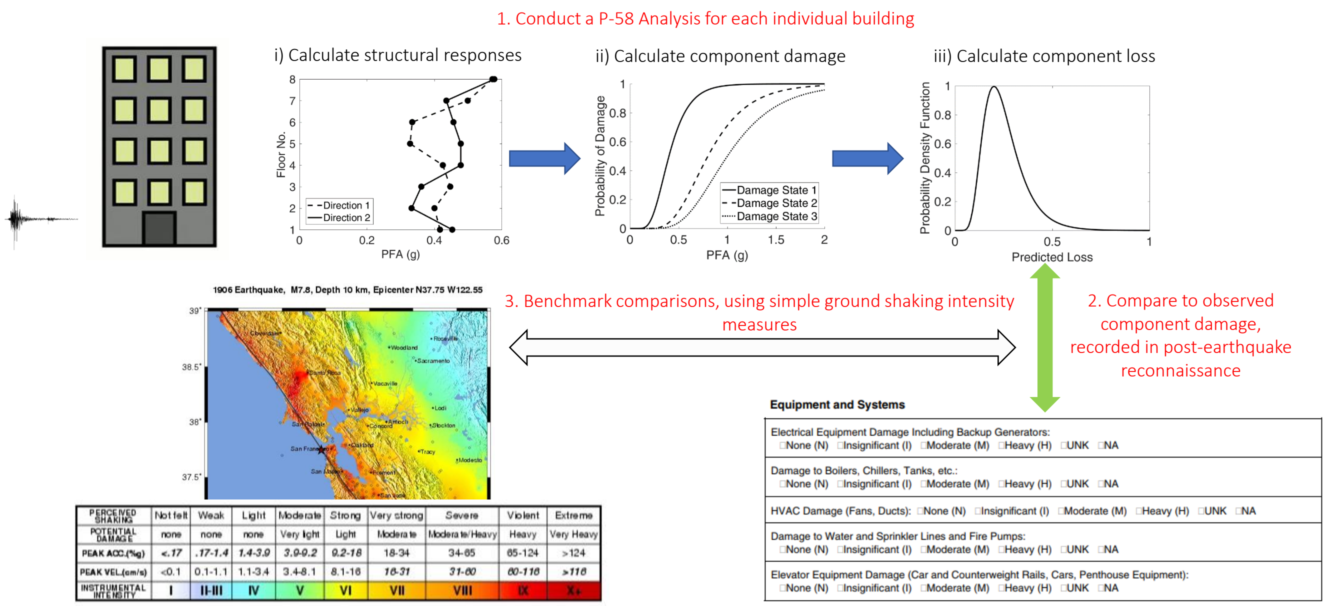


Figure 1. Overview of the benchmarking methodology. A FEMA P-58 analysis for the seismic event of interested is first conducted for every building, in which structural response inputs are translated to component-level damage predictions via component-level fragility functions, which are then used to compute component-level loss predictions. These component-level loss predictions are then compared with post-earthquake rapid assessment damage data. Finally, these comparisons are benchmarked against the comparison of ground shaking intensity with the observed damage data. Lower left figure from ShakeMap (Worden and Wald, 2016).

District (CBD), using data from a research database gathered after the Christchurch earthquake by Kim (2015), in collaboration with Christchurch City Council, the Canterbury Earthquake Recovery Authority, GNS Science, and from personal interviews. The database contains details of building characteristics as well as post-earthquake damage observations, and is limited to reinforced concrete shear walls and moment frames. The buildings range from non-seismic designed (pre-1965) to modern code (post-2003), and the number of stories range from 3 to 20. More data on these buildings can be found in Appendix B.

We examine 11 buildings for the Northridge event throughout the greater Los Angeles area, using post-earthquake inspection data from the Strong Motion Instrumentation Program (SMIP) Information System (Naeim, 1997; Naeim and Lobo, 1998). The buildings are a mixture of concrete and steel structures. Lateral systems in the concrete structures include shear walls and moment frames. Lateral systems in the steel structures include moment frames, concentrically braced frames, and chevron braced frames. The buildings range in age from 4 to 30 years (at the time of the earthquake), and range from 6 to 57 storeys in height. Further data on these buildings can be found in Appendix C.

STEP 1: RUNNING THE P-58 ANALYSES

Structural response inputs to the P-58 analyses for the Christchurch event are calculated using the FEMA P-58 simplified analysis procedure, since no instrumented data is available for the buildings (see Appendix A for more information). The types of components included in each model depend on the built era of the building (see Appendix B for more information).

For the Northridge event, structural response inputs to the P-58 analyses are derived from building instrument response data recorded at each building during the earthquake (see Appendix A for more information). Electrical equipment, HVAC, and piping components included in each building model are seismically rated, based on photographic evidence (Naeim, 1997) and personal communication with the author. All other building components included in the models are those autopopulated by the SP3 software tool (see Appendix C for more information). The SP3 software tool is used to run FEMA P-58 analyses in both cases.

The structural response inputs are then used in component-level fragility functions to calculate damage predictions for every component in a building. Depending on the component of interest, the structural response input to a given fragility function is either peak floor acceleration or story drift ratio. Component-level damage predictions are finally translated to component-level loss predictions using loss curves.

STEP 2: COMPARING OBSERVATIONS AND PREDICTIONS

Observed Component Damage Data

For the Christchurch event, we use non-structural component damage data recorded on Christchurch Earthquake Level 2 rapid assessment forms, which are provided in the New Zealand research database. This form was developed as part of the Guidelines for Building Safety Evaluation prepared by the New Zealand Society for Earthquake Engineering (Marquis, 2015), and is based on ATC-20-2 (Rojahn, 1995). Structural, nonstructural, and geotechnical damage is broken down to component groupings on the second page of the assessment form. Component damage is classified into three levels: 'Minor/None', 'Moderate' and 'Severe'. The 'Minor/None' and 'Severe' categories are hereafter referred to as 'None/Insignificant' and 'Heavy', to be consistent with the damage descriptions used for the Northridge event.

For the Northridge event, we use non-structural component damage data reported on ATC-38 post-earthquake building assessment forms (Rojahn, 2000), which are provided as part of the SMIP Information System. Component damage is recorded under both the 'Nonstructural Damage' and 'Detailed Damage Description' headings of these forms, and is classified into four levels: 'None', 'Insignificant', 'Moderate', and 'Heavy'. We treat damage reported as 'None' and 'Insignificant' under one category ('None/Insignificant'), to be consistent with the format of observed damage reporting used in the Christchurch event.

P-58 Predicted Losses

The predicted losses for P-58 components are grouped together, in accordance with the damage categories used in the post-earthquake survey. Depending on the nature of these damage categories, there might only be one P-58 component type represented in a given group of predicted losses. The predicted loss associated with each P-58 component group is calculated using the following predicted loss ratio (LR):

$$LR = \frac{\sum_i RC_i}{\sum_i ReplC_i} \leq 1 \tag{1}$$

where RC_i is the repair cost of the ith component included in the group, and $ReplC_i$ is its replacement cost.

Component repair costs are obtained by summing the costs across all units of the component in the building. The assumed replacement cost per unit of a component depends on the logical relationship that exists between damage states. For sequential damage states, it is taken to be the mean repair cost associated with the highest possible damage state. In the case of mutually exclusive damage states, it is assumed to be the largest mean repair cost of any damage state. For simultaneous damage states, it is the sum of the mean repair costs associated with each damage state. The total replacement cost for a component is found by scaling the replacement cost per unit by the total number of units present in the building. Note that we ignore FEMA P-58 volume discounting in our analyses.

Linking Observed Damage and Predicted Losses

We use the following three component groupings for the Christchurch case:

1. **Elevator Group:** We use the predicted losses associated with the 'Traction Elevator' component, to correspond with the 'Elevators' category of the Christchurch Earthquake Level 2 rapid assessment form. The fragility of the 'Traction Elevator' component is a function of peak floor acceleration in FEMA P-58.
2. **Cladding Group:** We use the predicted losses associated with the 'Curtain Walls' and 'Precast Concrete Panels' components, to correspond with the data recorded in the 'Cladding, glazing' category of the form. The fragilities of both components are a function of story drift ratio in FEMA P-58.
3. **Stairs Group:** We use the predicted losses associated with the 'Concrete Stairs' component, to correspond with the data recorded in the 'Stairs/Exits' category of the form. The fragility of the 'Concrete Stairs' component is a function of story drift ratio in FEMA P-58.

We use the following three component groupings for the Northridge case:

1. **Chiller Group:** We use the predicted losses associated with the 'Chiller' and 'Cooling Tower' components, to correspond with the data recorded in the 'Damage to Boilers, Chillers, Tanks, etc.' category of the ATC-38 post-earthquake damage assessment form. This comparison is carried out for only 7 of the 11 building models, since HVAC components do not appear by default in P-58 building models for either hotels or residential structures. The fragilities of both components are a function of peak floor acceleration in FEMA P-58.

2. **Elevator Group:** We use the predicted losses associated with the 'Traction Elevator' component, to correspond with the data recorded in the 'Elevator Equipment Damage' category of the form. The fragility of the 'Traction Elevator' component is a function of peak floor acceleration in FEMA P-58.
3. **Sprinkler Group:** We use the predicted losses associated with the 'Fire Sprinkler Water Piping' component, to correspond with the data recorded in the 'Damage to Water and Sprinkler Lines and Fire Pumps' category of the form. The fragility of the 'Fire Sprinkler Water Piping' component is a function of peak floor acceleration in FEMA P-58.

The component groupings used for each case are the only ones for which sufficient observed damage data is available for the buildings of interest.

Comparing Observations and Predictions

Comparing post-earthquake rapid assessment damage data and P-58 component-level loss predictions requires the use of tools that can relate categorical and numerical data. We use two rank-order statistical tests in our methodology that have this specific ability. We use the visual tool shown in Figure 2 to supplement the findings of each test. Component-level predicted loss ratios are grouped along the y-axis in accordance with the corresponding level of observed damage, to highlight ordinal differences between predictions associated with different damage observations. The data plotted in Figure 2 consist of the mean predicted loss data and the observed damage data for the Elevator grouping of components in Northridge buildings.

Statistical Test 1: Wilcoxon Rank-Sum Test

The Wilcoxon rank-sum test (Wilcoxon, 1945) is used to provide an understanding of the ability of the FEMA P-58 loss predictions to distinguish between buildings in which a given component grouping was reported as having 'None/Insignificant' damage, and buildings in which the component grouping was reported as having worse damage (i.e. 'Moderate' or 'Heavy'), by ranking the predicted loss ratios for the component grouping across the set of buildings of interest.

The alternative hypothesis for this test states that the sum of the ranks of the predicted loss ratios associated with 'Moderate' or 'Heavy' observed damage for the component grouping of interest is sufficient such that their distribution is shifted to the right of the predicted loss ratio distribution associated with 'None/Insignificant' damage. The null hypothesis states that the two populations of predicted loss ratios have identical distributions.

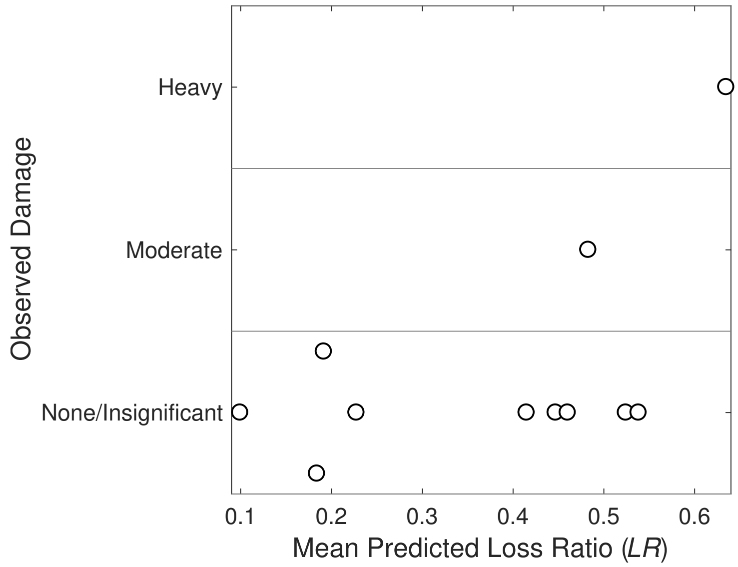


Figure 2. Supplementary visual tool for comparisons between observations and predictions, demonstrated with data for the Elevator grouping of components in Northridge buildings. Data points within each observed damage group with identical/similar x coordinates are offset vertically to aid visualization.

There is strong correspondence between predictions and observations if the alternative hypothesis is true. We can measure the statistical incompatibility of the data with the null hypothesis using the p-value: the probability of obtaining the given (or more extreme) comparison between observations and predictions if the null hypothesis is true. The smaller the p-value, the greater the statistical incompatibility of the data with the null hypothesis.

Assume that there are N buildings in the set of interest, and the component grouping of interest is observed to have either 'Moderate' or 'Heavy' damage in n of these buildings and observed to have 'None/Insignificant' damage in the remaining m of the buildings. The N predicted loss ratios for the component grouping are ranked in magnitude, and w is the sum of the ranks of the n predicted loss ratios associated with 'Moderate' or 'Heavy' damage.

Let W_s represent the set of all possible sums of n ranks from the available N. The p-value is the probability that W_s is at least as large as w. It is computed as follows:

$$\text{p-value} = p(W_s \geq w) = \frac{\#(w; n, m)}{\binom{N}{n}} \tag{2}$$

where $\#(w; n, m)$ denotes the number of all divisions of ranks 1,.,N into n and m ranks for which the sum of the n ranks is at least equal to w, and $\binom{N}{n}$ is the probability of any such division. For the data of Figure 2, $N = 11$, $n = 2$, $m = 9$, $w = 19$, $\#(w; n, m) = 4$, $\binom{N}{n} = 55$, and the resulting p-value = 0.07.

Statistical Test 2: Jonckheere-Terpstra Test

The Jonckheere-Terpstra test (Lunneborg, 2005) is used to provide insight on the ability of FEMA P-58 loss predictions to distinguish between buildings based on the observed damage levels of the component groupings of interest and be ordered in magnitude in accordance with the observed damage levels. Note that this statistic is only valid when there are at least three observed damage levels.

The alternative hypothesis for this test states that the medians of the predicted loss ratios associated with each of the observed damage levels (i.e. 'None/Insignificant', 'Moderate', and 'Heavy') for the component grouping of interest are different and have an a priori ordering in line with the observed damage levels, while the null hypothesis states that the medians of the three populations of predicted loss ratios are identical. Again, we use the p-value to calculate the probability of obtaining the given (or more extreme) comparison between observations and predictions if the null hypothesis is true.

Predicted loss ratios for the component grouping of interest are divided into three groups, based on the corresponding observed damage level. T is the sum of counts of predicted loss ratios across different groups that are correctly ordered in magnitude in line with their respective observed damage levels.

$$T = \sum_{k1=1}^{2} \sum_{k2=2}^{3} U_{k1k2} \tag{3}$$

where U_{k1k2} is the number of predicted loss ratios of observed damage level $k1$ that are less than each predicted loss ratio of more severe observed damage level $k2$ (with equal values in each group counted as 0.5). Let $f(T_s)$ represent the distribution of all possible values of T, which is assumed to be normal. The p-value is the probability that T_s is at least as large as T. It is computed as follows:

$$\text{p-value} = p(T_s \geq T) = (1 - \frac{T - E[T_s]}{\sigma_{T_s}}) \tag{4}$$

$$E[T_s] = \frac{N^2 - \sum_k n_k^2}{4} \tag{5}$$

$$\sigma_{T_s} = \sqrt{\frac{N^2(2N+3) - \sum_k n_k^2(2n_k+3)}{72}} \tag{6}$$

where N is the total number of buildings and n_k is the number of buildings associated with observed damage level k. For the data of Figure 2, $T = 17$, $E[T_s] = 9.5$, $\sigma_{T_s} = 4.3$, and the resulting p-value = 0.04.

We acknowledge that p-values are often misused and misinterpreted as the size of an effect or the importance of a result (Wasserstein and Lazar, 2016). It should be emphasized that there is no correct p-value for either test and we do not attempt to use p-values to make claims about the truth of the alternative hypotheses. We use them only as a means of providing evidence against the null hypotheses. In addition, the visual tool shown in Figure 2 can act as a sanity check on the findings of each test; strong correspondence between predictions and observations will result in a progressive increase in the height of each group of data across the x-axis.

STEP 3: BENCHMARKING THE COMPARISONS

We use comparisons between ground shaking intensity at each building and observed component damage to benchmark the evaluation of P-58 loss predictions. The relevant acceleration for each building is assumed to be the one-second peak spectral acceleration value, $S_a(1s)$, reported at the nearest grid point on the USGS ShakeMap (Worden and Wald, 2016) for the event of interest. We use ShakeMap values as they are developed in a standardized manner, globally available, and do not require strong motion instrumentation in a building.

While many previous studies (e.g. Cordova et al., 2000; Vamvatsikos and Cornell, 2002; Bojorquez and Iervolino, 2011) have concluded that a single value of spectral acceleration cannot accurately describe building response, single ground shaking intensity measures have been related to damage in rapid post-earthquake loss assessment procedures (e.g. Earle et al., 2009; Wald et al., 2008; Porter et al., 2008). $S_a(1s)$ values are used in this study simply to investigate if P-58 component-level loss predictions can better determine the correct ordering of observed component damage than a ground shaking intensity measure that can be obtained without any knowledge of building properties.

RESULTS

Figures 3-5 show comparisons of mean predicted loss ratios and observed damage, as well as the benchmark ground shaking comparison, for each of the three component groupings for the Christchurch event. The statistical tests indicate that the mean predicted loss ratios provide stronger evidence than the ground shaking benchmark against an inability to order based on observed damage for all component groupings examined. The plots also suggest that the ability of the mean predicted loss ratios to order based on observed damage severity is significantly better than that of the ground shaking benchmark. It is not surprising that the negligible variation in ground shaking is not predictive of damage in this case. It is, however, reassuring that P-58 predicted losses have superior ability, given the similarity of construction between the considered buildings and the fact that component fragility and repair cost data in FEMA P-58 are based primarily on United States rather than New Zealand construction data.

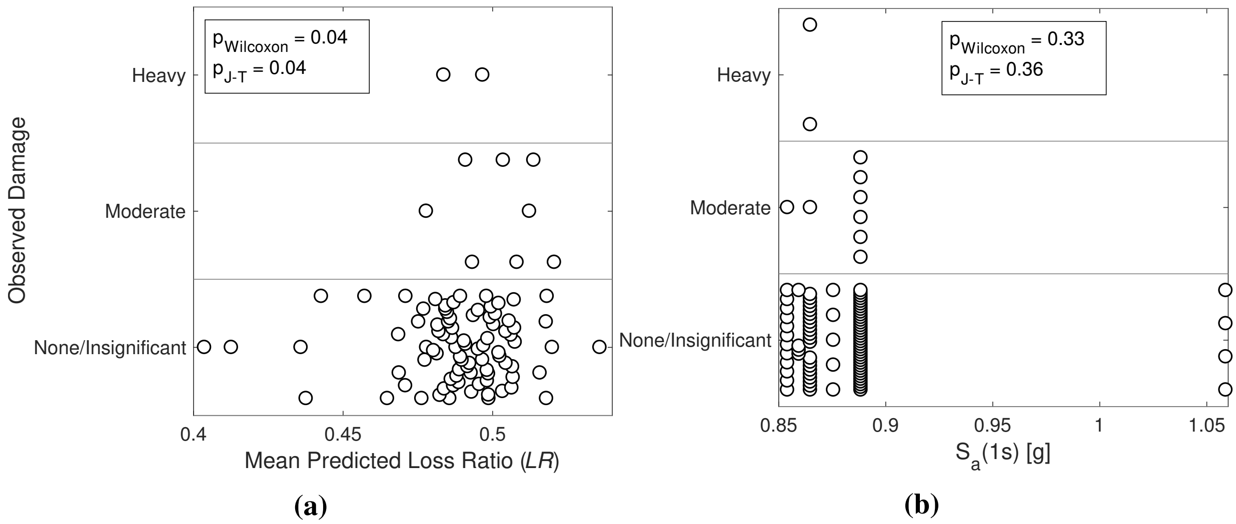
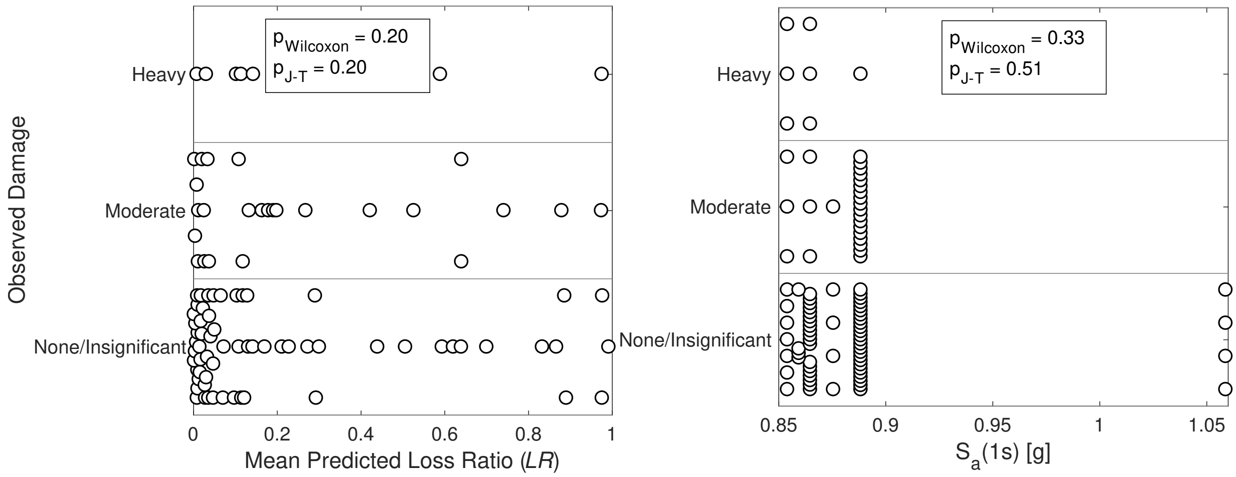


Figure 3. Observed damage to the Elevator grouping of components in Christchurch buildings, plotted versus (a) FEMA P-58 loss predictions and (b) $S_a(1s)$ amplitudes. (Note the narrow range of $S_a(1s)$ values). Data points within each observed damage group with identical/similar x coordinates are offset vertically to aid visualization.

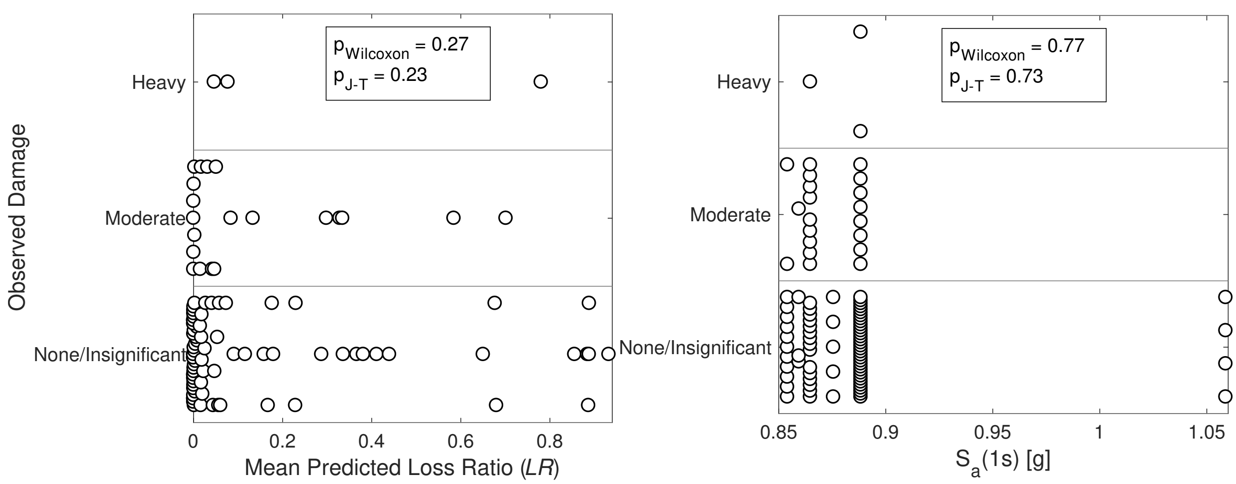
If we take account of uncertainty in predicted loss ratios, and perform the statistical tests for each set of Monte Carlo samples of predicted loss ratios, we find that a large proportion (73% on average across the six statistical tests) of these sets of predicted loss ratios result in smaller p-values than those of the benchmark (Figure 6), and the proportion is always greater than that expected by chance. Therefore, we conclude that P-58 predicted losses provide significant benefit over simply using ground shaking intensity as a predictor of damage in this case.

Figures 7-9 show comparisons of mean predicted loss ratios and observed damage, as well as the benchmark ground shaking comparison, for each of the three component groupings for the Northridge event. The statistical tests indicate that the mean predicted loss ratios provide stronger evidence than the ground shaking benchmark against an inability to order based on observed damage for the Elevator component grouping, but provide identical evidence for the other two component groupings. The plots support the findings of the tests, and suggest that the ability of mean predicted loss ratios to order according to observed damage severity is superior to that of the ground shaking benchmark for the Elevator component grouping, but identical to that of the benchmark for the other two component groupings. It is not surprising that ground



(a) **(b)**

Figure 4. Observed damage to the Cladding grouping of components in Christchurch buildings, plotted versus (a) FEMA P-58 loss predictions and (b) $S_a(1s)$ amplitudes. (Note the narrow range of $S_a(1s)$ values). Data points within each observed damage group with identical/similar x coordinates are offset vertically to aid visualization.



(a) **(b)**

Figure 5. Observed damage to the Stairs grouping of components in Christchurch buildings, plotted versus (a) FEMA P-58 loss predictions and (b) $S_a(1s)$ amplitudes. (Note the narrow range of $S_a(1s)$ values). Data points within each observed damage group with identical/similar x coordinates are offset vertically to aid visualization.

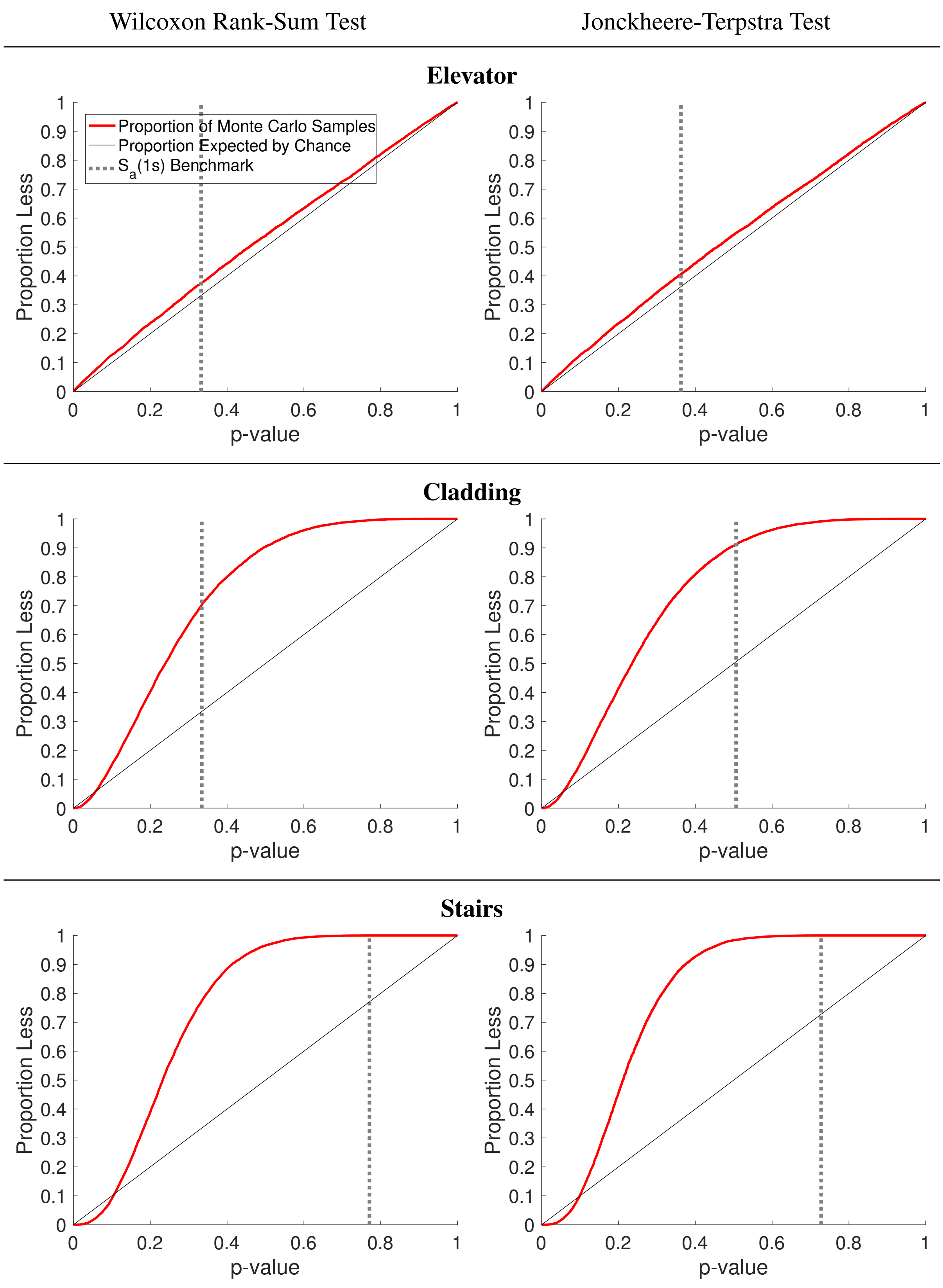


Figure 6. Proportion of predicted loss ratio Monte Carlo samples with p-values less than that on the x-axis, plotted with the proportion expected by chance and the benchmark $S_a(1s)$ p-values, for both statistical tests and all component groupings in Christchurch buildings.

shaking has comparable ability to order damage for these data, given the notable variation in shaking intensity between buildings.

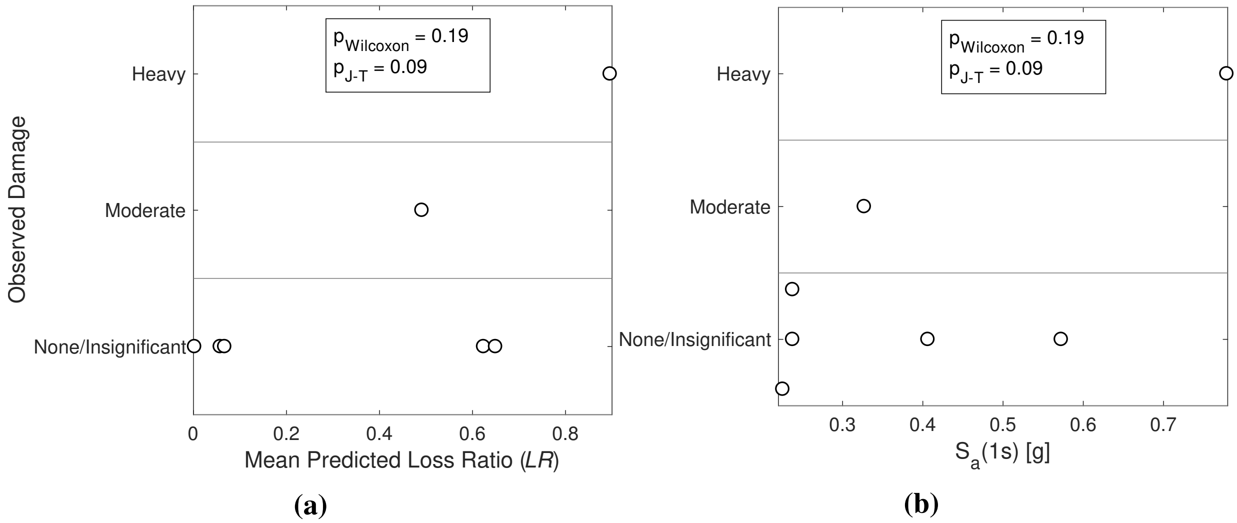


Figure 7. Observed damage to the Chiller grouping of components in Northridge buildings, plotted versus (a) FEMA P-58 loss predictions and (b) $S_a(1s)$ amplitudes. Data points within each observed damage group with identical/similar x coordinates are offset vertically to aid visualization.

If we take account of uncertainty in predicted loss ratios, and perform the statistical tests for each set of Monte Carlo samples of predicted loss ratios, we find that a considerable proportion (32% on average across the six statistical tests) of these sets of predicted loss ratios result in smaller p-values than those of the ground shaking benchmark (Figure 10), and the proportion is always notably greater than that expected by chance. The sets of predicted loss ratios with smaller p-values than the ground shaking benchmark provide stronger evidence than the ground shaking intensity against an inability to order based on observed damage. Therefore, we conclude that P-58 predicted losses provide significant ability to predict variations in losses to buildings, and some benefit over simply using ground shaking intensity as a predictor of damage severity, even when there is substantial variation in ground shaking.

CONCLUSIONS

This study proposed a methodology for evaluating component-level loss predictions of the FEMA P-58 Seismic Performance Assessment methodology across a group of buildings subjected to a given seismic event, using damage data collected in post-earthquake damage surveys. There is usually not enough information recorded on the surveys to directly assess quantitative loss predictions, so the methodology uses recorded categorical component damage informa-

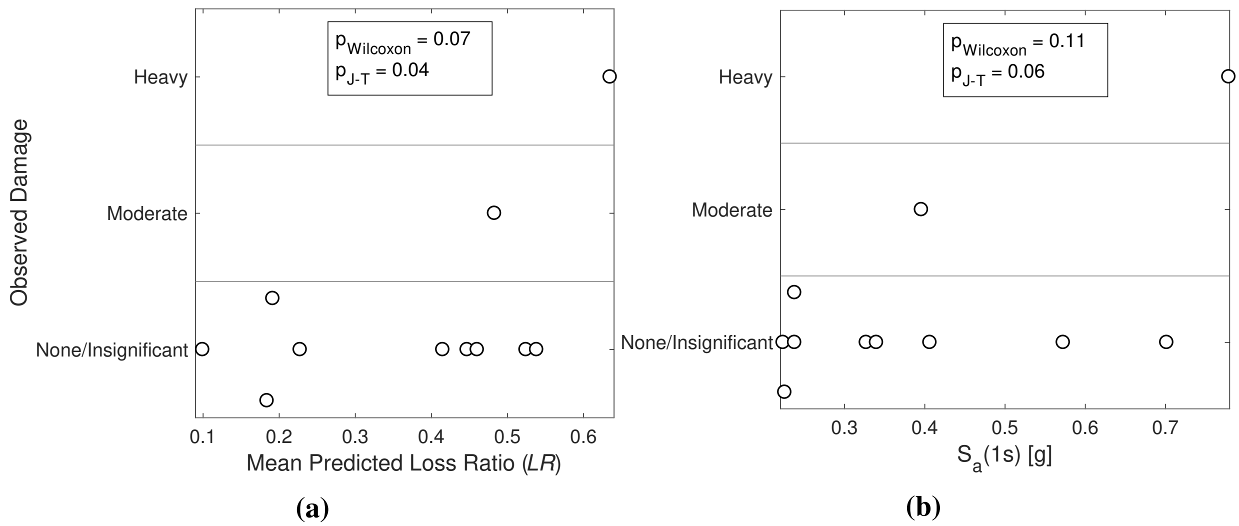


Figure 8. Observed damage to the Elevator grouping of components in Northridge buildings, plotted versus (a) FEMA P-58 loss predictions and (b) $S_a(1s)$ amplitudes. Data points within each observed damage group with identical/similar x coordinates are offset vertically to aid visualization.

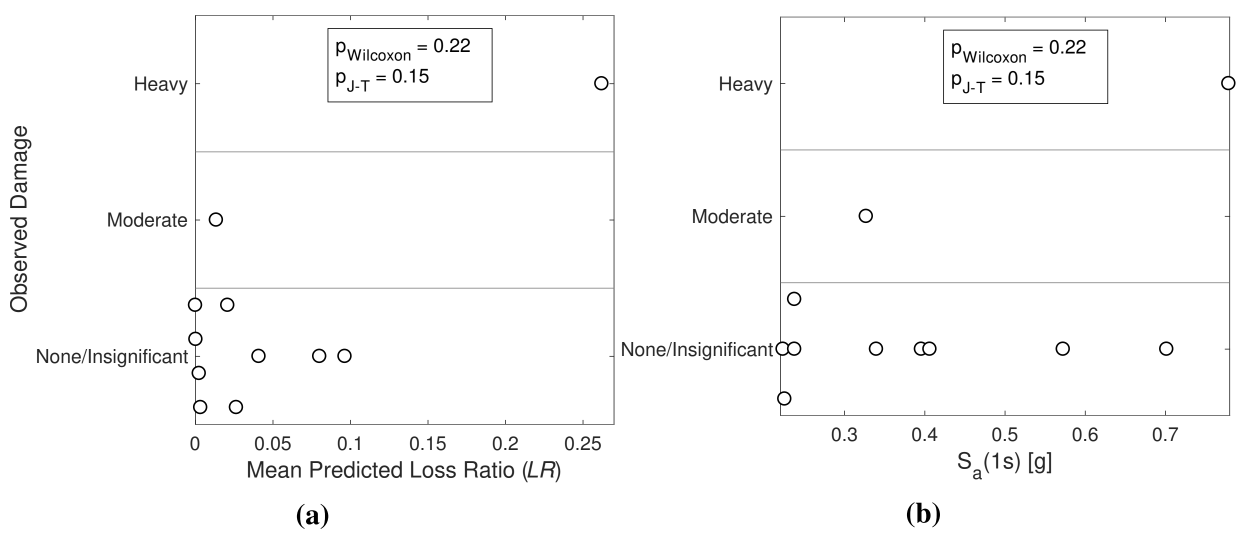


Figure 9. Observed damage to the Sprinkler grouping of components in Northridge buildings, plotted versus (a) FEMA P-58 loss predictions and (b) $S_a(1s)$ amplitudes. Data points within each observed damage group with identical/similar x coordinates are offset vertically to aid visualization.

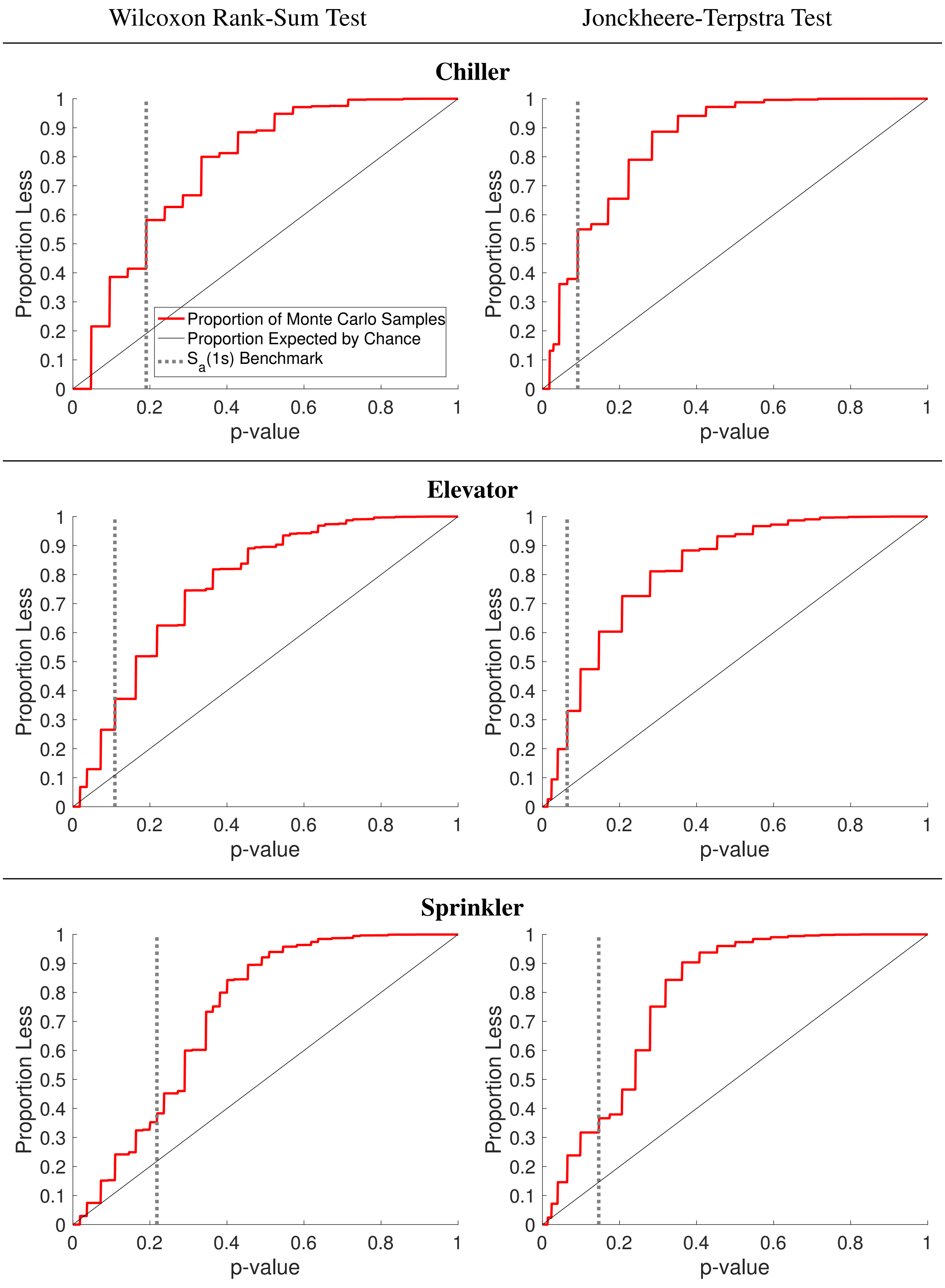


Figure 10. Proportion of predicted loss ratio Monte Carlo samples with p-values less than that on the x-axis, plotted with the proportion expected by chance and the benchmark $S_a(1s)$ p-values, for both statistical tests and all component groupings in Northridge buildings.

tion to determine if the P-58 component-level loss predictions can be ordered according to the observed component damage severity.

The proposed methodology includes a novel loss ratio metric that links P-58 predicted losses to the categorical component damage reconnaissance data. Statistical tools are used to evaluate the comparison of numerical and categorical data, accounting for uncertainties in predictions. Ground shaking intensity at the different buildings is used as a benchmark to investigate whether P-58 component-level loss predictions, using knowledge of building properties, are a better predictor of component damage severity than the variation in shaking from building to building.

Non-structural component-level data from the 2011 Christchurch and 1994 Northridge earthquakes are used to illustrate the methodology. There is negligible variation in ground shaking between buildings for the Christchurch event, and notable variation in ground shaking intensity between buildings for the Northridge event. We find that, overall, the FEMA P-58 loss predictions perform better than the ground shaking benchmark in both cases. It is concluded that FEMA P-58 provides benefit over simply using ground shaking intensity measures as a predictor of component-level damage, when there is both small and large variation in the ground shaking between buildings. It is particularly beneficial when there is small variation in ground shaking.

While the methodology is limited to evaluating relative rankings of FEMA P-58 component-level loss predictions across a set of buildings and does not eliminate the need for other building-specific studies or direct validation of loss predictions, it is broadly applicable and offers an understanding of the degree to which performance-based earthquake engineering calculations reflect real-life consequences of seismic events. This is important as the FEMA P-58 methodology becomes more widely adopted in design and risk analysis practice.

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APPENDIX A

STRUCTURAL RESPONSE DATA USED FOR THE EXAMPLE CASES

CHRISTCHURCH EVENT

We use responses predicted by the intensity-based FEMA P-58 simplified analysis procedure in this case (FEMA, 2012a). This method requires peak ground acceleration (PGA) and spectral acceleration at the fundamental period ($S_a(T_1)$) input values, and computes structural responses using linear models, static analyses, and an estimate of the lateral yield strength of the structure (V_y). Table 1 provides parameter values used for the 95 buildings in the method's equations, included in Section 5.3.1 of FEMA (2012a). a and α are used to compute Δ_i in equation 5-10 of FEMA (2012a), using the equations for lateral displacement provided in Miranda (1999).

The PGA and $S_a(T_1)$ input values are obtained using 24 strong motion recordings that were recorded within a site-to-source distance of 30km, in conjunction with both an empirical ground motion model and a spatial correlation model (Bradley, 2012, 2013). We use log-log interpolation to find the spectral acceleration associated with the fundamental period of the building. All other parameter values used are the default values set by the SP3 program for each model.

Table 1. Summary of FEMA P-58 Simplified Method input data for the Christchurch buildings.

#	PGA [g]	$S_a(T_1)$ [g]	V_y	First Mode Mass Ratio	Soil Site Class	α	a
1	0.42	0.96	0.033	0.90	D	12.50	0.01
2	0.43	0.96	0.033	0.90	D	12.50	0.01
3	0.42	0.96	0.033	0.90	D	12.50	0.01
4	0.42	0.96	0.033	0.90	D	12.50	0.01
5	0.42	0.96	0.033	0.90	D	12.50	0.01
6	0.43	0.90	0.033	0.90	D	12.50	0.01
7	0.43	0.89	0.033	0.90	D	12.50	0.01
8	0.43	0.89	0.033	0.90	D	12.50	0.01
9	0.44	0.95	0.033	0.90	D	12.50	0.01
10	0.41	0.86	0.033	0.90	D	12.50	0.01
11	0.42	0.89	0.040	0.80	D	3.75	0.01
12	0.43	0.88	0.033	0.90	D	12.50	0.01
13	0.44	0.88	0.040	0.80	D	3.75	0.01
14	0.43	0.91	0.040	0.80	D	3.75	0.01
15	0.43	0.96	0.050	0.80	D	1.00	0.01
16	0.42	0.96	0.050	0.80	D	1.00	0.01
17	0.41	1.00	0.033	0.90	D	12.50	0.01
18	0.42	0.81	0.033	0.90	D	12.50	0.01
19	0.43	0.93	0.050	0.80	D	1.00	0.01
20	0.43	0.82	0.033	0.90	D	12.50	0.01
21	0.45	0.95	0.067	0.90	D	12.50	0.01
22	0.44	0.96	0.050	0.80	D	3.75	0.01
23	0.43	0.92	0.050	0.80	D	1.00	0.01
24	0.43	0.96	0.050	0.80	D	1.00	0.01
25	0.44	0.91	0.067	0.90	D	12.50	0.01
26	0.43	0.95	0.067	0.90	D	12.50	0.01
27	0.44	0.79	0.038	0.80	D	3.75	0.01
28	0.45	0.87	0.050	0.80	D	1.00	0.01
29	0.45	0.81	0.067	0.90	D	12.50	0.01
30	0.44	0.84	0.067	0.90	D	12.50	0.01
31	0.43	0.83	0.050	0.80	D	1.00	0.01
32	0.44	0.82	0.075	0.80	D	1.00	0.01
33	0.44	0.86	0.100	0.80	D	1.00	0.01
34	0.44	0.86	0.067	0.90	D	12.50	0.01
35	0.43	0.67	0.038	1.00	D	1.00	0.01
36	0.41	0.83	0.050	0.80	D	1.00	0.01
37	0.44	0.82	0.133	0.90	D	12.50	0.01
38	0.45	0.80	0.133	1.00	D	12.50	0.01
39	0.42	0.74	0.067	1.00	D	12.50	0.01
40	0.45	0.85	0.133	0.90	D	12.50	0.01

41	0.42	0.35	0.017	1.00	D	12.50	0.01
42	0.44	0.83	0.133	0.90	D	12.50	0.01
43	0.44	0.90	0.133	0.90	D	12.50	0.01
44	0.45	0.95	0.133	0.90	D	12.50	0.01
45	0.44	0.86	0.133	0.90	D	12.50	0.01
46	0.43	0.95	0.133	0.90	D	12.50	0.01
47	0.44	0.85	0.133	0.90	D	12.50	0.01
48	0.43	0.95	0.133	0.90	D	12.50	0.01
49	0.44	0.93	0.166	0.90	D	12.50	0.01
50	0.42	0.94	0.080	0.90	D	12.50	0.01
51	0.43	0.86	0.133	0.90	D	12.50	0.01
52	0.45	0.93	0.100	0.80	D	1.00	0.01
53	0.39	0.96	0.100	0.80	D	1.00	0.01
54	0.44	0.82	0.075	0.80	D	1.00	0.01
55	0.43	0.65	0.038	1.00	D	1.00	0.01
56	0.39	0.73	0.075	0.80	D	1.00	0.01
57	0.44	0.90	0.100	0.80	D	1.00	0.01
58	0.44	0.83	0.050	0.80	D	1.00	0.01
59	0.39	0.72	0.057	0.80	D	1.00	0.01
60	0.40	0.85	0.166	0.90	D	12.50	0.01
61	0.44	0.90	0.100	0.80	D	1.00	0.01
62	0.44	0.66	0.075	1.00	D	1.00	0.01
63	0.43	0.80	0.075	0.80	D	1.00	0.01
64	0.44	0.82	0.200	0.80	D	1.00	0.01
65	0.44	0.84	0.200	0.80	D	1.00	0.01
66	0.44	0.84	0.120	0.80	D	3.75	0.01
67	0.44	0.80	0.150	0.80	D	1.00	0.01
68	0.44	0.85	0.200	0.80	D	1.00	0.01
69	0.43	0.81	0.080	0.80	D	3.75	0.01
70	0.43	0.95	0.200	0.80	D	1.00	0.01
71	0.44	0.93	0.200	0.80	D	1.00	0.01
72	0.44	0.79	0.150	0.80	D	1.00	0.01
73	0.43	0.66	0.075	1.00	D	1.00	0.01
74	0.42	0.78	0.075	0.80	D	1.00	0.01
75	0.44	0.85	0.200	0.80	D	1.00	0.01
76	0.44	0.93	0.200	0.80	D	1.00	0.01
77	0.44	0.81	0.200	0.80	D	1.00	0.01
78	0.44	0.93	0.200	0.80	D	1.00	0.01
79	0.44	0.90	0.200	0.80	D	1.00	0.01
80	0.44	0.93	0.200	0.80	D	1.00	0.01
81	0.43	0.85	0.200	0.80	D	1.00	0.01
82	0.44	0.33	0.067	1.00	D	12.50	0.01
83	0.41	0.79	0.200	0.80	D	1.00	0.01
84	0.43	0.93	0.100	0.80	D	1.00	0.01
85	0.42	0.80	0.150	0.80	D	1.00	0.01
86	0.42	0.92	0.200	0.80	D	1.00	0.01
87	0.41	0.70	0.150	1.00	D	1.00	0.01
88	0.42	0.91	0.200	0.80	D	1.00	0.01

89	0.41	0.81	0.200	0.80	D	1.00	0.01
90	0.39	0.96	0.150	0.80	D	1.00	0.01
91	0.44	0.83	0.150	0.80	D	1.00	0.01
92	0.44	0.96	0.200	0.80	D	1.00	0.01
93	0.43	0.69	0.150	1.00	D	1.00	0.01
94	0.43	0.90	0.160	0.80	D	3.75	0.01
95	0.40	0.66	0.075	1.00	D	1.00	0.01

NORTHRIDGE EVENT

The structural response data used in this case are building instrument response data recorded during the Northridge earthquake. These data are provided as part of the SMIP Information System. We use cubic spline interpolation of the response time series at instrumented floors to determine responses at non-instrumented floors. This technique has been used in previous studies to recover demands at non-instrumented floors (Limongelli, 2003; Naeim et al., 2004).

We assume that the peak floor acceleration (PFA) for a given floor i in a given direction is given by:

$$PFA_i = max|a_i(t)| \tag{7}$$

where $a_i(t)$ is the acceleration at floor i for time t, obtained from instrumented records or interpolation. We assume that the story drift ratio (SDR) for a given story i in a given direction is given by:

$$SDR_i = \frac{max|d_i(t) - d_{i+1}(t)|}{h_i} \tag{8}$$

where $d_i(t)$ is the displacement at floor i for time t, obtained from instrumented records or interpolation, and h_i is the height of story i. We include a log-standard deviation (β) of 0.4 on all responses, to account for any errors introduced in using interpolation at non-instrumented floors, as well as discrepancies observed in the responses recorded by different instruments positioned in identical directions on the same floor.

APPENDIX B

CHRISTCHURCH BUILDING DATA USED IN EXAMPLE CASES

The New Zealand research database (Kim, 2015) is used to obtain building data for the Christchurch event. Details of the 95 buildings we studied from this database are provided in Table 2, in descending order of the average mean predicted loss ratio across the three component groupings examined. These buildings were chosen since there is reasonably complete damage data available for them in the database. Shear Wall (SW) buildings are modeled as a Reinforced Concrete Shear Wall, Moment Frame (MF) and Moment Frame with Infill (MFIF) buildings are modeled as a Perimeter Reinforced Concrete Moment Frame, and Moment Frame/Shear Wall (MF/SW) buildings are modeled as a Reinforced Concrete Shear Wall and Frame (Dual System).

Table 2. Summary of the Christchurch building data used. Note that Observed Damage and Mean Predicted Loss Ratio are reported in the following order: Elevator Group, Cladding Group, Stairs Group. For Observed Damage, N = 'None/Insignificant', M = 'Moderate', and H = 'Heavy'.

#	# Stories	Built Era	Period (s)	Lateral System	Observed Damage	Mean Predicted Loss Ratio	$S_a(1s)$ [g]
1	2	Pre-1965	0.48	MFIF	(N,N,N)	(0.49,0.99,0.93)	0.89
2	3	Pre-1965	0.48	MF	(N,N,N)	(0.49,0.98,0.88)	0.89
3	3	Pre-1965	0.48	MF	(N,M,N)	(0.48,0.97,0.89)	0.89
4	3	Pre-1965	0.48	MF	(N,N,N)	(0.48,0.98,0.88)	1.06
5	3	Pre-1965	0.48	MF	(M,H,N)	(0.48,0.98,0.89)	0.86
6	4	Pre-1965	0.62	MF	(N,N,M)	(0.49,0.89,0.70)	0.89
7	4	Pre-1965	0.62	MF	(N,N,N)	(0.48,0.89,0.68)	0.89
8	4	Pre-1965	0.62	MF	(N,M,N)	(0.48,0.88,0.68)	0.89
9	3	1965-1975	0.48	MF	(N,M,N)	(0.49,0.64,0.86)	0.86
10	4	Pre-1965	0.62	MF	(N,N,N)	(0.47,0.87,0.65)	0.86
11	3	Pre-1965	0.30	MF/SW	(H,N,M)	(0.48,0.83,0.58)	0.86
12	4	1965-1975	0.62	MFIF	(H,H,H)	(0.50,0.59,0.78)	0.86
13	3	Pre-1965	0.40	MF/SW	(M,M,N)	(0.49,0.74,0.44)	0.89
14	4	Pre-1965	0.40	MF/SW	(N,N,N)	(0.49,0.70,0.41)	0.89
15	3	Pre-1965	0.48	SW	(N,M,M)	(0.49,0.64,0.33)	0.89
16	3	Pre-1965	0.48	SW	(N,N,N)	(0.49,0.64,0.34)	0.86
17	5	Pre-1965	0.76	MF	(N,N,M)	(0.48,0.62,0.33)	0.86
18	4	Pre-1965	0.76	MFIF	(N,N,M)	(0.48,0.59,0.30)	0.86
19	4	Pre-1965	0.57	SW	(N,M,N)	(0.48,0.53,0.23)	0.89
20	5	Pre-1965	0.76	MFIF	(N,N,N)	(0.48,0.51,0.23)	0.86
21	3	1965-1975	0.48	MFIF	(N,N,N)	(0.50,0.30,0.38)	0.88
22	3	1965-1975	0.40	MF/SW	(N,N,N)	(0.49,0.29,0.37)	1.06
23	5	Pre-1965	0.57	SW	(M,N,N)	(0.49,0.44,0.18)	0.89
24	4	Pre-1965	0.48	SW	(N,M,N)	(0.48,0.42,0.16)	0.89
25	4	1965-1975	0.62	MFIF	(N,N,N)	(0.51,0.23,0.29)	0.89
26	3	1976-1991	0.48	MF	(N,N,N)	(0.49,0.29,0.12)	0.86
27	8	Pre-1965	0.90	MF/SW	(N,N,M)	(0.50,0.27,0.08)	0.89
28	6	Pre-1965	0.73	SW	(N,M,H)	(0.50,0.27,0.08)	0.89
29	6	1965-1975	0.89	MF	(N,H,N)	(0.50,0.14,0.18)	0.89
30	5	1965-1975	0.76	MF	(N,M,N)	(0.49,0.13,0.17)	0.89
31	7	Pre-1965	0.73	SW	(N,N,N)	(0.50,0.21,0.05)	0.86
32	8	Pre-1965	0.81	SW	(N,M,H)	(0.50,0.20,0.05)	0.89
33	7	Pre-1965	0.73	SW	(N,M,M)	(0.50,0.18,0.04)	0.89
34	4	1976-1991	0.76	MF	(N,M,N)	(0.48,0.19,0.04)	0.89
35	13	Pre-1965	1.23	SW	(N,N,N)	(0.50,0.17,0.04)	0.89
36	5	1965-1975	0.65	SW	(N,N,M)	(0.46,0.10,0.13)	0.86
37	6	1976-1991	0.89	MFIF	(N,M,M)	(0.50,0.16,0.03)	0.89
38	7	1976-1991	1.03	MF	(N,N,N)	(0.50,0.13,0.02)	0.88
39	6	1965-1975	1.03	MF	(N,N,N)	(0.49,0.14,0.03)	0.86
40	6	1976-1991	0.89	MF	(N,N,N)	(0.49,0.13,0.03)	0.88
41	13	1965-1975	1.91	MF	(N,N,N)	(0.48,0.07,0.09)	0.86

42	6	1976-1991	0.89	MF	(N,N,N)	(0.50,0.12,0.02)	0.86
43	4	1976-1991	0.62	MF	(N,N,N)	(0.50,0.12,0.02)	0.86
44	3	1976-1991	0.48	MF	(N,M,N)	(0.50,0.11,0.02)	0.88
45	5	1976-1991	0.76	MF	(N,H,N)	(0.49,0.11,0.02)	0.86
46	3	1976-1991	0.48	MF	(N,N,N)	(0.49,0.12,0.02)	0.86
47	5	1976-1991	0.76	MF	(N,N,M)	(0.50,0.11,0.02)	0.86
48	3	1992-2003	0.48	MF	(N,M,N)	(0.49,0.12,0.02)	0.86
49	4	1992-2003	0.62	MF	(N,N,N)	(0.51,0.10,0.02)	1.06
50	4	1965-1975	0.40	MFIF	(N,N,N)	(0.49,0.05,0.07)	1.06
51	4	1992-2003	0.62	MF	(N,H,M)	(0.48,0.10,0.02)	0.86
52	5	1965-1975	0.57	SW	(N,N,N)	(0.50,0.04,0.06)	0.89
53	4	1965-1975	0.48	SW	(N,N,N)	(0.48,0.04,0.06)	0.86
54	10	1965-1975	0.95	SW	(N,N,M)	(0.50,0.03,0.05)	0.89
55	13	1965-1975	1.29	SW	(N,N,N)	(0.50,0.03,0.05)	0.89
56	7	1965-1975	0.81	SW	(N,N,N)	(0.47,0.04,0.06)	0.86
57	5	1976-1991	0.65	SW	(N,N,N)	(0.51,0.05,0.00)	0.89
58	7	1965-1975	0.81	SW	(N,N,N)	(0.48,0.07,0.01)	0.89
59	9	Pre-1965	0.95	SW	(N,N,M)	(0.47,0.04,0.06)	0.86
60	4	1992-2003	0.62	MF	(N,N,N)	(0.47,0.07,0.01)	0.86
61	5	1976-1991	0.65	SW	(N,N,N)	(0.49,0.05,0.00)	0.89
62	14	1976-1991	1.29	SW	(N,H,N)	(0.51,0.03,0.00)	0.86
63	9	1992-2003	0.95	SW	(N,N,N)	(0.48,0.05,0.01)	0.86
64	3	1992-2003	0.39	SW	(N,N,N)	(0.54,0.00,0.00)	0.88
65	7	1976-1991	0.81	SW	(N,N,N)	(0.52,0.02,0.00)	0.86
66	8	1976-1991	0.80	MF.SW	(N,N,M)	(0.51,0.03,0.00)	0.89
67	9	1992-2003	0.88	SW	(M,M,N)	(0.51,0.03,0.00)	0.89
68	7	1992-2003	0.73	SW	(M,M,N)	(0.52,0.01,0.00)	0.89
69	3	1992-2003	0.40	MF/SW	(N,M,N)	(0.49,0.04,0.00)	0.86
70	4	1976-1991	0.48	MF	(N,N,M)	(0.52,0.01,0.00)	0.86
71	5	1976-1991	0.57	SW	(N,N,N)	(0.52,0.01,0.00)	0.86
72	10	1992-2003	0.95	SW	(N,N,M)	(0.51,0.02,0.00)	0.89
73	13	1976-1991	1.23	SW	(N,M,N)	(0.50,0.03,0.00)	0.89
74	8	1976-1991	0.88	SW	(N,M,M)	(0.49,0.03,0.00)	0.86
75	3	1976-1991	0.39	SW	(N,M,N)	(0.52,0.00,0.00)	0.86
76	4	1976-1991	0.48	SW	(M,H,M)	(0.51,0.01,0.00)	0.86
77	3	1976-1991	0.39	SW	(N,N,N)	(0.52,0.00,0.00)	0.86
78	5	1976-1991	0.57	SW	(N,N,N)	(0.51,0.01,0.00)	0.89
79	3	1992-2003	0.39	SW	(M,M,N)	(0.51,0.00,0.00)	0.89
80	5	1976-1991	0.57	SW	(N,M,N)	(0.51,0.01,0.00)	0.89
81	7	1976-1991	0.73	SW	(M,N,N)	(0.50,0.01,0.00)	0.89
82	20	1976-1991	2.00	MF	(N,N,N)	(0.49,0.02,0.00)	0.89
83	7	1976-1991	0.73	SW	(N,N,M)	(0.50,0.01,0.00)	0.86
84	5	1976-1991	0.57	SW	(N,N,N)	(0.50,0.01,0.0)	0.89
85	8	1976-1991	0.81	SW	(N,N,N)	(0.49,0.02,0.00)	0.86
86	3	1992-2003	0.39	SW	(N,N,N)	(0.50,0.01,0.00)	0.86
87	12	1976-1991	1.09	SW	(N,M,M)	(0.49,0.01,0.00)	0.86
88	3	1976-1991	0.39	SW	(N,N,N)	(0.50,0.00,0.00)	0.89
89	6	1976-1991	0.65	SW	(N,N,N)	(0.48,0.02,0.00)	0.86

90	3	1976-1991	0.48	SW	(N,N,N)	(0.46,0.04,0.00)	0.86
91	8	2004-Present	0.81	SW	(N,M,N)	(0.44,0.02,0.00)	0.89
92	4	2004-Present	0.48	SW	(N,N,N)	(0.44,0.01,0.00)	0.89
93	13	2004-Present	1.16	SW	(N,N,N)	(0.44,0.01,0.00)	0.89
94	6	2004-Present	0.60	MF/SW	(N,N,N)	(0.41,0.02,0.00)	0.89
95	12	2004-Present	1.16	SW	(N,N,N)	(0.40,0.02,0.00)	0.86

ASSUMPTIONS USED IN CHRISTCHURCH P-58 BUILDING MODELS

1. To facilitate discrepancies between New Zealand seismic design standards and the US building codes used in the SP3 software tool (Davenport, 2004; Hamburger et al., 2012; Olshansky, 1998; MacRae et al., 2011; Taylor et al., 1997), we assign design years before 1941 in the software for pre-1965 buildings, and buildings from the built era 1992-2003 are assigned design years after 1994. Seismic building standards were largely similar in the two regions for the built eras 1965-1975, 1976-1991, and 2004-present, so any design year within the applicable built era is selected for associated buildings.
2. We do not model building basements, assuming they do not contain any components of interest.
3. Since stairs with sliding detail were typically used in post-1976 buildings (e.g. Kam and Pampanin, 2011), the stairs included in each post-1976 building model have Fragility ID 'C2011.011a' in FEMA P-58, which represents concrete stairs with seismic joints, while the stairs included in each pre-1976 building model have P-58 Fragility ID 'C2011.011b', which represents concrete stairs with no seismic joints. Exceptions are made for two buildings designed between 1965 and 1976 (Building Nos. 39 and 58), for which the 'C2011.011a' component is used since there is specific reference to sliding ability and 'gaps' in descriptions of the buildings' staircases in the database documentation. We only consider in-plane (i.e. drift-controlled) damage for the 'Precast Concrete Panels' components (P-58 Fragility ID 'B2011.201a'), as the out-of-plane (i.e. acceleration-controlled) fragility function requires a site-specific code-calculated acceleration capacity that is not available for the set of buildings in this event. Precast concrete panels have only been used in New Zealand since the 1960's (Seifi et al., 2016), so we use the brick-clad facade represented by P-58 Fragility ID 'B2011.203a' instead for pre-1965 buildings, to model the masonry/plaster cladding that was typically used at that time according to the database documentation. The elevators used in each building are the default type autopopulated in the models by the SP3 software tool; post-2004 building models contain elevators with

P-58 Fragility ID 'D1014.011', while all other building models contain elevators with P-58 Fragility ID 'D1014.012'.

4. The observed component damage is altered in our analyses from the level recorded on the Christchurch Earthquake Level 2 rapid assessment forms for a small number of cases, based on our interpretation of more detailed descriptions of component damage provided in the database documentation:

   - The observed level of damage for 'Stairs/Exits' is changed from 'None/Insignificant' to 'Moderate' in Building No. 11, since each flight of stairs showed cracking and spalling of the underside concrete, which resulted in the stairs being tied to the floor landings as a safety precaution.

   - The observed level of damage for 'Stairs/Exits' is changed from 'None/Insignificant' to 'Moderate' in Building No. 15, since there was cracking of concrete landings as large as 3mm at various levels in the main stair case, as well as cracking and spalling of stair flights.

   - The observed level of damage for 'Stairs/Exits' is changed from 'None/Insignificant' to 'Moderate' in Building No. 47, since there was cracking of the stairs and landings, and the cracking in the main stair required it to be replaced for all stories above the first.

   - The observed level of damage for 'Cladding,glazing' is changed from 'None/Insignificant' to 'Moderate' in Building No. 15, since there was movement and joint damage to the stonework façade, as well as cracking of the external masonry wall.

   - The observed level of damage for 'Cladding,glazing' is changed from 'None/Insignificant' to 'Moderate' in Building No. 19, since there were cracks in the plaster render that required it to be replaced.

   - The observed level of damage for 'Cladding,glazing' is changed from 'None/Insignificant' to 'Moderate' in Building No. 3, since there was cracking in the rear blockwall and disturbance to the block/brick junction in the northeast corner of the building.

While some limited information on building periods and the layout of building dimensions can be obtained from various engineering reports conducted after the Canterbury earthquakes, the level of data available is not the same across each study building. We neglect this information to ensure consistency in the amount of known information included in the analysis for each building. We make the following assumptions for each building:

5. We assume the fundamental period in both directions is equivalent to the value suggested by HAZUS, as reported in the SP3 software tool. If a suggestion is not provided, we assume the fundamental period (in seconds) in both directions is 0.1 times the number of stories in the building.
6. We assume that the buildings are square in plan.

APPENDIX C

NORTHRIDGE BUILDING DATA USED IN EXAMPLE CASES

The SMIP Information System (Naeim, 1997; Naeim and Lobo, 1998) is used to obtain building data for the Northridge event. It provides detailed information on building characteristics, including non-structural systems and contents, as well as damage observations, for 19 instrumented buildings throughout the Los Angeles area. Details of the 11 buildings that we studied from this database are provided in Table 3 below, in descending order of the average mean predicted loss ratio across the two or three component groupings examined. These 11 buildings were chosen since their associated lateral systems can be modeled in a FEMA P-58 analysis and there are sufficient damage descriptions available for them in the database. The Reinforced Concrete and Steel Shear Wall (RC/S SW) building is modeled as a Reinforced Concrete Shear Wall, the Reinforced Concrete Moment Frame and Shear Wall (RC MF/SW) buildings are modeled as Reinforced Concrete Shear Wall and Frame (Dual System), the Steel X-Braced Frame and Moment Frame (S XF/MF) building is modeled as a Steel Concentrically Braced Frame in the transverse direction and as a Steel Perimeter Moment Frame in the longitudinal direction, the Steel Moment Frame (S MF) buildings are modeled as a Steel Perimeter Moment Frame, the Reinforced Concrete Column-Slab Frame and Column-Spandrel Beam Frame (RC F) building is modeled as a Reinforced Concrete Perimeter Moment Frame, the Precast Shear Wall (PC SW) buildings are modeled as a Reinforced Concrete Shear Wall, the Steel Concentrically Braced Frame with Moment Resisting Connections and Outrigger Moment Frames (S CBF/MF) building is modeled as a Steel Concentrically Braced Frame, and the Steel Chevron Braced and Moment Frame (S CB/MF) building is modeled as a Steel Perimeter Moment Frame.

Table 3. Summary of the Northridge building data used. Note that Observed Damage and Mean Predicted Loss Ratio are reported in the following order: Chiller Group, Elevator Group, Sprinkler Group. For Observed Damage, N = 'None/Insignificant', M = 'Moderate', and H = 'Heavy'. Chiller Group components did not feature in hotels or residential structures.

#	# Stories	Design Year	Period (s)	Lateral System	Observed Damage	Mean Predicted Loss Ratio	$S_a(1s)$ [g]
1	6	1976	0.46	RC/S SW	(H,H,H)	(0.90,0.63,0.26)	0.78
2	15	1964	2.75	RC MF/SW	(N,N,N)	(0.62,0.54,0.08)	0.57
3	23	1967	3.00	S XF/MF	(N,N,N)	(0.65,0.41,0.10)	0.41
4	6	1976	1.28	S MF	(M,N,M)	(0.49,0.46,0.01)	0.33
5	7	1965	1.50	RC F	($\sim$,N,N)	($\sim$,0.52,0.03)	0.70
6	21	1967	2.20	RC MF/SW	($\sim$,M,N)	($\sim$,0.48,0.02)	0.40
7	10	1974	0.60	PC SW	($\sim$,N,N)	($\sim$,0.45,0.04)	0.34
8	17	1980	0.90	PC SW	($\sim$,N,N)	($\sim$,0.23,0.00)	0.22
9	57	1988	6.00	S CBF/MF	(N,N,N)	(0.07,0.19,0.00)	0.24
10	6	1988	0.85	S CB/MF	(N,N,N)	(0.06,0.18,0.00)	0.23
11	56	1988	5.40	S MF	(N,N,N)	(0.00,0.10,0.00)	0.24

ASSUMPTIONS USED IN NORTHRIDGE P-58 BUILDING MODELS

1. Building periods used are those reported in the SMIP information system, except for that of Building No. 5 which is obtained from Krawinkler (2005).
2. We do not model building basements, assuming they do not contain any components of interest.
3. We use the P-58 combined anchorage/internal equipment fragility functions for both the chiller and cooling tower components in buildings designed after 1976, with capacities calculated according to Chapter 13 of ASCE (2016), and the vibration isolated equipment fragilities for both components in all other buildings. Note that, in accordance with section 2.5.13 of FEMA (2012b), the combined anchorage/internal equipment fragility functions should be used for both components in all buildings. However, use of these fragility functions for buildings designed before 1977 produces unrealistically high component capacities and results in a significantly poorer comparison between predicted losses and observed damage data. The P-58 Fragility IDs of the chiller and cooling tower components included in each relevant building model are provided in Table 4. The elevators included in each post-1976 building model have P-58 Fragility ID 'D1014.011', while all other building models contain elevators with P-58 Fragility ID 'D1014.012'. Post-1976 building models contain fire sprinkler water piping with P-58 Fragility ID 'D4011.023a',

and all other building models contain fire sprinkler water piping with P-58 Fragility ID 'D4011.022a'.

Table 4. P-58 Fragility IDs of chiller and cooling tower components included in relevant building models.

#	Chiller	Cooling Tower
1	D3031.011d	D3031.021d
2	D3031.011c	D3031.021c
3	D3031.011d	D3031.021d
4	D3031.011b	D3031.021b
9	D3031.013l	D3031.023l
10	D3031.013f	D3031.023f
11	D3031.013l	D3031.023l