Enhanced Loss Estimation for Buildings with Explicit Incorporation of Residual Deformation Demands

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SUMMARY:
A new loss estimation methodology is developed for performance based earthquake design. Unlike current loss estimation methodologies which typically estimate economic losses based only on peak response quantities such as peak interstory drift ratios or peak floor accelerations the proposed approach explicitly incorporates residual interstory drifts. The new approach explicitly accounts for the probability of having to demolish a building as a function of residual interstory drifts. The proposed approach is illustrated with four reinforced concrete moment resisting frame buildings in Los Angeles, California. Results from this study indicate that residual drift may have a large impact on economic losses. This is particularly true in the case of ductile buildings that have a larger deformation capacity and therefore smaller probability of collapse at during intense ground motions, but have a considerable probability of experiencing residual displacement. It is concluded that neglecting losses from residual drifts can lead to significant underestimation of economic losses.

Keywords: Performance based earthquake engineering; residual drifts; loss estimation; building demolition.

1. INTRODUCTION

Careful observation of the performance of buildings that have sustained inelastic deformations in previous earthquakes indicates that significant residual displacements may occur. Recent analytical and experimental studies (Mahin and Bertero 1981, MacRae and Kawashima 1997, Pampanin et al. 2002, Ruiz-Garcia and Miranda 2005) have shown that the likelihood of experiencing residual deformations increases as the level of inelastic deformation increases. This suggests that lateral force resisting systems that are capable of sustaining large lateral displacements are more likely to experience residual deformations when subjected to seismic ground motions. Although it has been demonstrated that structural systems with large deformation capacity have relatively small probabilities of collapse (Haselton and Deierlein, 2007), these structures are more likely sustain residual drifts that may result in poor performance when considering other metrics such as economic losses and the loss of facility use after and earthquake has occurred.

Residual displacements and its consequences play an important role in structural performance (Ruiz-Garcia & Miranda 2005). In the 1985 Michoacán earthquake, for instance, numerous reinforced concrete buildings were demolished in Mexico City because of the complications associated with repairing and straightening the permanent deformations the structures experienced (Rosenblueth and Meli, 1986). In another example, many reinforced concrete bridge piers in Kobe sustained excessive residual displacements during the Hyogo-Ken-Nambu earthquake, forcing government officials to demolish and replace them (Kawashima, 2000). In both examples, the structures performed well in terms of preventing collapse, as they were able to remain standing; however, from a loss standpoint, these structures exhibited poor structural performance as they had to be demolished leading not only to large economic losses associated with demolition and reconstruction but also with the loss of use of the facility for very long periods of time until reconstruction was completed.

Recent advances in earthquake engineering – particularly the advent of the Pacific Earthquake Engineering Research Center's (PEER) performance-based design framework – have attempted to use economic loss as a performance metric for design. PEER's evaluation framework can be traced back to building-specific loss estimation methodologies first proposed by Scholl et al. (1982). Initially, PEER's framework was based methods that only considered economic losses due to repairable damage given the building has not collapsed. Monetary losses induced by seismic ground motions are calculated by using structural response parameters (peak interstory drift, peak floor accelerations…etc.) to estimate damage in different building components. The cost to repair each damaged component is then summed for the entire building to obtain the total loss of the building. This methodology was eventually improved to include economic losses due to building replacement given that the building has collapsed (Miranda et al. 2004, Aslani and Miranda 2005). However, there is nothing in the current framework that accounts for the economic loss due to demolishing a building that has not collapsed but cannot be repaired because of excessive residual deformation.

The objective of this paper is to present a summary of an improved loss estimation methodology that explicitly incorporates economic losses resulting from the possibility of having to demolish buildings that have experienced large residual interstory drifts. The improved methodology is illustrated by computing economic losses in four reinforced concrete moment-frame buildings. In each case economic losses are estimated with the existing loss estimation methodologies and the proposed approach. In each case economic losses are estimated with the existing and improved methodologies.

2. IMPROVED LOSS ESTIMATION METHODOLOGY

In recently proposed loss estimation methodologies (Krawinkler and Miranda 2004; Miranda et al. 2004; Aslani and Miranda 2005, Mitriani-Rieser 2007) expected losses at a given level of ground motion intensity $E[L_T \mid IM]$, are computed as the weighted sum of expected losses in two mutually exclusive, collectively exhaustive events. Namely: (1) collapse does not occur (non-collapse, NC) and damage in the building is repaired, R, (i.e., $NC \cap R$); and (2) collapse occurs and the building is rebuilt, C (Miranda et al. 2004, Aslani and Miranda 2005, Mitriani-Rieser 2007). The expected value of the loss in the building for a given ground motion intensity $IM=im$ is computed as

$$E\left[L_T \mid IM\right] = E\left[L_T \mid NC \cap R, IM\right] P\left(NC \cap R \mid IM\right) + E\left[L_T \mid C\right] P\left(C \mid IM\right) \tag{2.1}$$

Figure 1. Examples of buildings with large residual displacements that typically lead to demolition (left photo by M. Bruneau, MCEER; right photo by A. Whittaker, NISEE, EERC, UC Berkeley).

where $E[L_T \mid NC \cap R, IM]$ is the expected value of the total loss in the building provided that collapse does not occur (non-collapse) and the building is repaired given that it has been subjected to earthquakes with a ground motion intensity $IM=im$, and $E[L_T \mid C]$ is the expected loss in the building when collapse occurs in the building. The weights on these two expected losses are $P(NC \cap R \mid IM)$ which is the probability that the building will not collapse and that it will be repaired given that it has been subjected to earthquakes with a ground motion intensity $IM=im$, and $P(C \mid IM)$ is the probability that the structure will collapse under a ground motion with a level of intensity, im.

Previous investigations, however, have failed to recognize that another potential consequence of earthquake damage is when collapse does not occur but it has to be *demolished*, D (i.e., $NC \cap D$). Hence, this study proposes that a third intermediate term be added to equation (2.1), to account for economic losses that may result from this outcome. With this new term, equation (2.1) becomes

$$\begin{aligned} E\left[L_T \mid IM\right] &= E\left[L_T \mid NC \cap R, IM\right] P\left(NC \cap R \mid IM\right) \\ &\quad + E\left[L_T \mid NC \cap D\right] P\left(NC \cap D \mid IM\right) + E\left[L_T \mid C\right] P\left(C \mid IM\right) \end{aligned} \tag{2.2}$$

where $E[L_T \mid NC \cap D]$ is the expected loss in the building when there is no collapse but the building needs to be demolished. The weight on this expected loss is $P(NC \cap D \mid IM)$, which is the probability that the building will not collapse but that it has to be demolished, given that it has been subjected to earthquakes with a ground motion intensity $IM=im$. As will be shown later in many cases, ignoring this term can lead to significant underestimations of earthquake losses.

The probability that the building will not collapse and that it will be repaired given that it has been subjected to earthquakes with a ground motion with a level of intensity, im is given by

$$P\left(NC \cap R \mid IM\right) = P\left(R \mid NC, IM\right) P\left(NC \mid IM\right) \tag{2.3}$$

Similarly, the probability that the building will not collapse but that it will be demolished given that it has been subjected to earthquakes with a ground motion intensity $IM=im$ can be computed as

$$P\left(NC \cap D \mid IM\right) = P\left(D \mid NC, IM\right) P\left(NC \mid IM\right) \tag{2.4}$$

Since the repair and demolition events are mutually exclusive events given that no collapse has occurred and collapse and non-collapse are also mutually exclusive events, then

$$P\left(R \mid NC, IM\right) = 1 - P\left(D \mid NC, IM\right) \tag{2.5}$$

$$P\left(NC \mid IM\right) = 1 - P\left(C \mid IM\right) \tag{2.6}$$

Substituting (2.3) to (2.6) into (2.1) we obtain

$$\begin{aligned} E\left[L_T \mid IM\right] &= E\left[L_T \mid NC \cap R, IM\right] \left\{1 - P\left(D \mid NC, IM\right)\right\} \left\{1 - P\left(C \mid IM\right)\right\} \\ &\quad + E\left[L_T \mid NC \cap D\right] P\left(D \mid NC, IM\right) \left\{1 - P\left(C \mid IM\right)\right\} + E\left[L_T \mid C\right] P\left(C \mid IM\right) \end{aligned} \tag{2.7}$$

Previous studies (Aslani and Miranda 2005, Mitriani-Rieser 2007) have used a *component-based* methodology to calculate $E[L_T \mid NC \cap R, IM]$. The damage and corresponding loss for every component in the building is first calculated, and then summed up using the following equation:

$$E\left[L_T \middle| NC, IM\right] = \sum_{j=1}^{N} a_j E\left[L_j \middle| NC, IM\right] \tag{2.8}$$

where $E[L_j \mid NC,IM]$ is the expected normalized loss in the jth component given that global collapse has not occurred at the intensity level IM, a_j is the cost of a new jth component, and L_j is the normalized loss in the jth component defined as the cost of repair or cost to replace the component normalized by a_j.

Calculation of this term can be simplified by using a *story-based* approach presented by Ramirez and Miranda (2009). This approach relies on assumptions about the building's cost distribution to calculate loss at each story instead of at every component. If the repair costs are normalized in the same manner show in equation (2.8), only the value of the story needs to be specified, rather than the values and quantities of each component. Further, relationships that relate structural response directly to loss can be developed, such that the intermediate step of estimating component damage become unnecessary. This approach is used in this study and the reader is referred to Ramirez and Miranda (2009) for further details of the methodology.

In Equation (2.7) $E[L_T \mid C]$ corresponds to cost of removing the collapse structure from the site plus the replacement cost of the building. Similarly, $E[L_T \mid NC \cap D]$ is equal to the cost of demolishing the existing structure, plus cost of removing debris from the site plus the replacement cost minus the cost of building components that may be salvageable because the structure did not collapse.

As shown in Equation (2.7) all three terms are influenced by the probability of collapse, $P(C \mid IM)$. Previous studies (e.g., Krawinkler et al. 2005, Haselton and Deierlein 2007, Liel and Deierlein 2008) have used nonlinear structural simulation models analyzed using incremental dynamic analysis (Vamvatsikos and Cornell, 2002) to estimate the probability of collapse as a function of the ground motion intensity. Using a suite of ground motion records, incremental dynamic analysis captures the effects of variation in frequency content and other ground motion characteristics, on structural response. The outcome of incremental dynamic analysis is a set of statistical results that permit the estimation of the probability of exceedance of critical engineering demand parameters (EDP's) at a number of ground motion intensities and a collapse fragility function, which describes the probability of collapse as a function of the ground motion intensity.

In this study, the probability that a structure will have to be demolished is computed as a function of the maximum residual interstory drift in the building. The probability that a building will need to be demolished after an earthquake given that it has not collapse in an earthquake with intensity $IM = im$ can be computed applying the theorem of total probability as follows

$$P(D \mid NC, IM = im) = \int_0^\infty G_{K|RIDR}(D \mid RIDR = ridr, NC) \left| dP(RIDR > ridr \mid NC, IM = im) \right| \tag{2.9}$$

where, $G_{K|RIDR}(D \mid RIDR = ridr, NC)$ is the cumulative distribution function (CDF) for the probability that the structure will be demolished, given that the building has not collapsed but that it has experienced a maximum residual drift is equal to $ridr$, when subjected to an earthquake with intensity level, im, and $P(RIDR > ridr \mid NC, IM=im)$ is the probability of exceeding $ridr$ given the that the building has not collapsed at the intensity, im. Note that $RIDR$ is the maximum residual drift in any story in the building. $P(RIDR > ridr \mid NC, IM=im)$ can be determined from the structural response simulation similarly to other types of engineering demand parameters. In this study, $G_{K|RIDR}(D \mid RIDR = ridr, NC)$ was assumed to be lognormally distributed. This probability may be interpreted as the percentage of engineer that would recommend demolition of the building with increasing levels of residual interstory drift.

3. APPLICATIONS

To evaluate the influence of the explicit consideration of having to demolish a building due to excessive residual drifts, four reinforced concrete frame buildings whose seismic response was previously studied by other investigators were considered. The four case study buildings are: a 4-story building with ductile detailing, a 12-story building with ductile detailing, a 4-story building with non-ductile detailing, and a 12-story building with non-ductile detailing. All four structures were assumed to be located at a site in Los Angeles, CA, south of the city's downtown area, and is representative of a typical urban California site with high levels of seismicity, but not subject to near-fault directivity effects (Haselton et al., 2007). In this study, the spectral acceleration at the first mode period of the structure, $Sa(T_1)$ [g], was used to quantify characterize the ground motion intensity.

The two structures with ductile detailing were modern buildings designed by Haselton and Deierlein (2007) according to the 2003 International Building Code and related ACI and ASCE provisions (ACI 2002, ASCE 2002, ICC, 2003). Design spectra for the site was based on adjusted mapped values of S_s = 1.5g and S_1 = 0.6g. Both concrete frame structures are perimeter, special moment frame structures that comply with capacity design provisions, strong column-weak beam ratios, joint shear strength and detailing regulations, in addition to requirements for strength and stiffness. As modeled, the 4-story building and the 12-story building have fundamental periods of 0.94 and 2.14 seconds, respectively. Interested readers can refer to Haselton and Deierlein (2007) and Haselton et al. (2007)for more details about the designs and modeling parameters of the modern example structures.

For the structures with non-ductile detailing, the 1967 Uniform Building Code (UBC 1967) was used to design buildings that were more representative of older concrete frame structures erected in California from approximately 1950 to 1975. Both designs complied in accordance with Zone 3 requirements, the highest seismic design provisions of this era. Although these designs met all requirements of the 1967 UBC (ICBO, 1967) (such as maximum and minimum reinforcement ratios, maximum stirrup spacing, bar spacing and anchorage…etc.), these provisions did not require as much transverse reinforcing as modern day building codes, which result in members that are non-ductile. Further, at the time, no special provisions for design or reinforcement of beam-column joints were required and the strong-column, weak beam ratio requirement had yet to be introduced. Thus, these structures are susceptible to joint-shear failure and column hinging, respectively. The buildings are also much more flexible because of lower reinforcing restrictions and lower seismic design forces, as demonstrated by their fundamental periods of 1.96 and 2.75 seconds for the 4-story and 12-story buildings, respectively. Detailed information on the designs and modeling parameters of these structures can be found in Liel and Deierlein (2008).

Obtaining realistic loss estimations requires developing architectural layouts for the buildings being considered, to inventory the amount of damageable non-structural components and to determine the median total replacement cost. This study uses architectural layouts documented in Mitriani-Reiser (2007) and Ramirez and Miranda (2009). A rectangular footprint that is 120-ft wide and by 180-ft long, was used for the 4-story buildings. A smaller layout, with a 120-ft square footprint, was developed for the 12-story buildings. The architectural layouts were used to develop estimates of the total replacement costs of the buildings.

Each of the reinforced concrete frame structures of interest in this study was modeled in OpenSees (PEER, 2006) using a two-dimensional, three-bay model of the lateral resisting system and a leaning (P-Δ) column. The model does not incorporate strength or stiffness from components designed to resist gravity loads. Beams and columns were modeled with concentrated hinge (lumped plasticity) elements and employ a material model developed by Ibarra et al. (2005). Haselton and Deierlein (2007) have shown that the lumped plasticity modeling approach provides reasonable results (compared to fiber-element models) at low levels of deformation and in addition, captures material nonlinearities as the structure collapses.

The nonlinear simulation models of reinforced concrete frames were analyzed using the incremental dynamic analysis technique. The selected ground motions records are from large magnitude events and recorded at moderate fault-rupture distances on stiff soil or rock sites (ATC, 2008). The collapse fragility, typically represented by cumulative probability distribution, was adjusted to account for uncertainties in the structural modeling process and the expected spectral shape of rare ground motions in California. EDP data for the ductile, 4-story building considered in this study are shown in Figure 3.1.

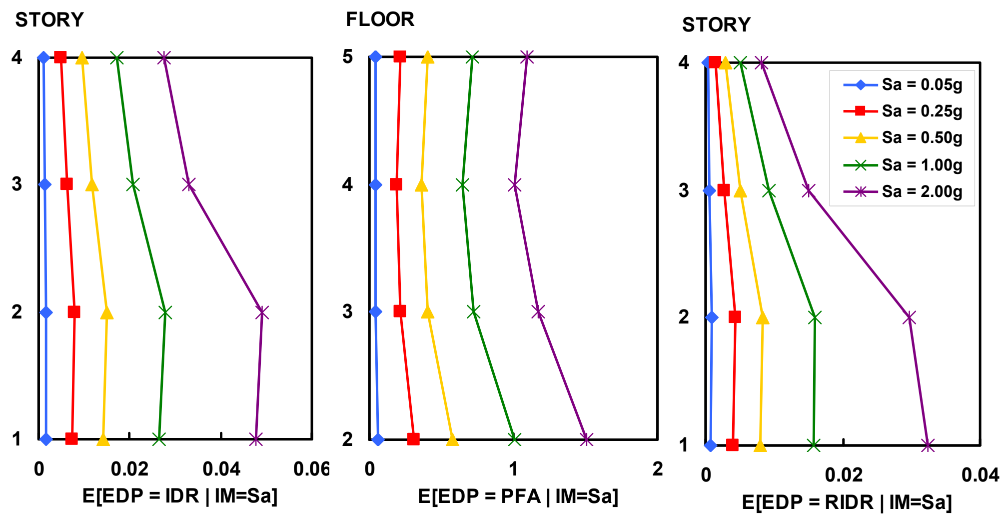

Figure 3.1. EDP data as a function of building height for ductile 4-story reinforced concrete structure

The economic loss results at the DBE normalized by the building replacement value for all four structures considered in this study are summarized in Table 3.1. The normalized values of each type of loss – non-collapse losses due to repair, non-collapse losses due to demolition and collapse losses – are reported in columns (1) through (3). Each of the values in columns (1), (2) and (3) are computed by the first, second and third terms in equation (2.7), respectively. Column (4) reports the overall economic loss when the three types of losses are summed together. The last three columns in the table report percent contributions of each type of loss (i.e. these values represent the losses computed in columns (1) to (3) but are normalized by the total loss in column (4)).

Table 1 Expected loss results at design basis earthquake (DBE) as a percentage of building replacement value.

Building	Expected Loss at design level EQ				Disaggregation		
	Eq. (9.2) Term 1	Eq. (9.2) Term 2	Eq. (9.2) Term 3	Total	Eq. (9.2) Term 1	Eq. (9.2) Term 2	Eq. (9.2) Term 3
	(1)	(2)	(3)	(4)	(1) / (4)	(2) / (4)	(3) / (4)
4-story Ductile	25%	15%	3%	42%	58%	36%	6%
12-story Ductile	15%	13%	6%	34%	44%	38%	18%
4-story Non-ductile	12%	12%	51%	74%	16%	16%	68%
12-story Non-ductile	4%	12%	65%	81%	5%	15%	80%

Results from the 4-story, modern structure with ductile detailing reveal a substantial increase in total expected losses when the effects of excessive residual drift are included in economic performance. Figure 3.2 compares the expected economic losses with and without considering losses due to demolition for three different levels of seismic hazard. The middle pair of bars corresponds to the expected economic losses that are incurred at the DBE. The pair to the left corresponds to the losses for a seismic due to a seismic event with a seismic intensity that has a probability of exceedance of 50% in 50 years (this hazard level occurs more frequently and is often referred to as the service-level earthquake). The pair to the right corresponds to the losses due to seismic event that has a probability of exceedance of 2% in 50 years (this hazard level occurs less frequently and is often referred to as the Maximum Credible Earthquake, MCE). The values of the seismic ground motion intensity that correspond to all three hazard levels are listed at the bottom of the figure. For each hazard level, the left bar corresponds to losses that do not consider losses due to demolition and the right bar corresponds to the losses that consider losses due to demolition.

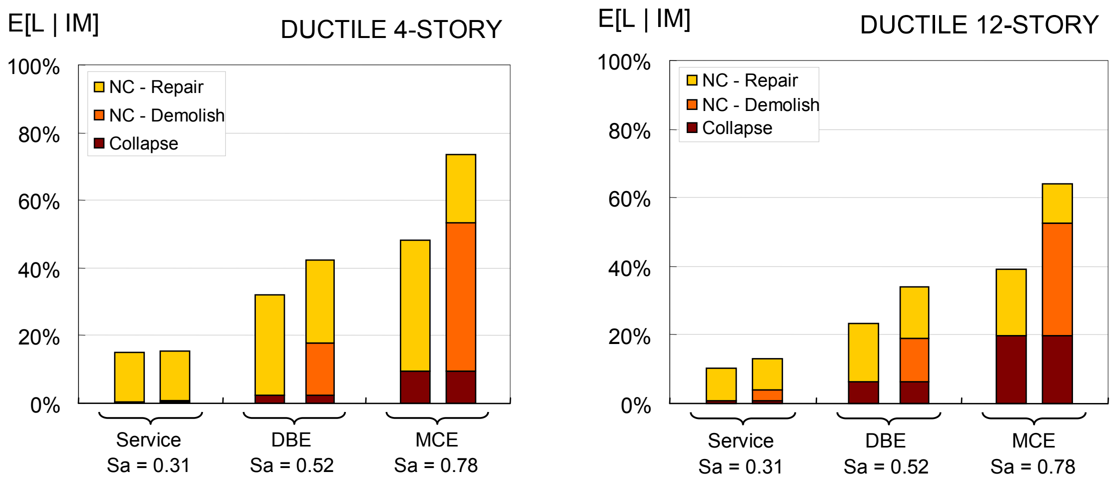

Figure 3.2 Effect of considering loss due to demolition conditioned on non-collapse on normalized expected economic losses for the 4-story and 12-story buildings at three different levels of seismic intensity.

At the service-level earthquake, the effect of losses due to building demolition does not have an influence on the overall normalized loss. On the other hand, the normalized economic losses increase from 31% to 42% at the DBE. This represents a relative increase of 35% (the relative increase is the difference between the two values of expected loss, with and without considering losses due to demolition, divided by the expected loss with considering losses due to demolition, multiplied by 100). At the MCE, the normalized economic losses increase from 48% to 73% representing a relative increase of 52%. This means that considering the losses due to demolition has a large influence on the overall loss estimate, particularly for seismic events that have smaller ground motion intensities but occur more frequently. At this level of ground motion intensity, the effect of economic losses due to demolition is even larger than it was at the DBE.

Loss results at these levels were disaggregated to observe how much each term in equation (2.7) contributes to the overall performance as shown in Figure 3.2 (Loss disaggregation was performed in a similar manner as documented by Aslani and Miranda, 2005). Each bar in the figure is divided up into collapse losses, non-collapse (NC) losses due to building demolition and non-collapse losses due to repair costs. The proportions of each bar are equal to how much each type of loss contributes to the overall loss. Demolition losses have the largest contributions to the overall loss at the MCE. At this intensity level, losses conditioned on non-collapse due to demolition represent 60% of the total loss. This is more than twice as large as the contributions from losses conditioned on collapse, which comprise 27% of the overall loss at the MCE. At the MCE, the probability of demolition is much higher (45%) than the probability of collapse (8%). This means that the structure is more likely to experience large residual deformations that will lead to demolition, as compared to collapse due to an earthquake for the given level of ground motion intensity. Consequently, the non-collapse losses due to demolition are much larger than the losses due to collapse because the total expected loss is the sum of these two types of losses weighted by the probability that these events will occur as demonstrated by equation (2.7).

These results suggest that ignoring the financial consequences of excessive residual drift can severely underestimate economic loss predictions. Previous loss estimation methods that do not include these effects may be misleading, predicting better performance than actually experienced. Performance can be underestimated by as much as 35% at the DBE and 52% at the MCE as demonstrated by this case-study.

The results from Ramirez and Miranda (2009) demonstrated that the effect of building height can have a substantial influence on predicted economic losses. The effect of building height was also investigated in this study by comparing the resulting economic losses of the 4-story structure to the results of a 12-story structure. Figure 3.2 shows the loss results for the 12-story building at the service-level earthquake, the DBE and the MCE. For each hazard level, the total normalized economic losses are smaller than the losses of the 4-story structure. Note that the losses are smaller when normalized by the replacement cost. The replacement costs and losses in the 12-story are larger in magnitude. This trend holds whether or not non-collapse losses due to demolition are considered.

Figure 3.2 also compares the losses with and without considering loss due demolition, for each hazard level. Despite demonstrating lower total losses, the relative increase in losses due to considering the effect of demolition losses is larger in the 12-story structure than it is in the 4-story building. When demolition losses are considered at the DBE, normalized losses are 45% greater than when the demolition losses are ignored. This is larger than the 35% increase in normalized loss observed in the loss analysis of the 4-story structure. This is also true for the MCE, where the relative increase due to demolition losses is 63% for the 12-story building and only 52% for the 4-story building. This suggests that large residual drifts may play a more significant role in estimating loss for high-rise buildings, than it does for low-rise buildings.

The loss results in 3.2 were disaggregated as described previously to determine the value of contributions from each of the terms in equation (2.7). As was the case for the 4-story building, demolition losses comprise the largest portion of the overall loss at the MCE. At this level, losses due to demolition make up 51% of the overall loss. These case-studies suggest that accounting for excessive residual drift in loss estimations is important in both low-rise and high-rise structures. Although the proportion of demolition losses relative to the overall loss is approximately the same in both cases, it was demonstrated in the previous paragraph that demolition losses may have a much larger influence in high-rise buildings than they do in low-rise buildings.

Despite exhibiting larger overall losses, the effects of residual drift on performance did not have as significant an influence on the older structures (non-ductile detailing) as it did with the modern structures (ductile detailing). Figure 3.3 shows that the total economic loss at the DBE for a 4-story non-ductile building (74%) is greater than the corresponding total loss for the ductile 4-story structure reported previously (42%). However, when comparing the effect of demolition losses, the influence of loss due to demolition is not as great in the non-ductile structure as it is in the ductile structure. At the DBE, the increase in loss experienced by the non-ductile building is only 12% while the increase in the ductile structure is 35%. Similar results can be observed in the non-ductile, 12-story structure as illustrated by Figure 3.3 The probability of collapse plays a much larger role in building performance than the probability of demolition for non-ductile structures. This can be better demonstrated by examining the deaggregation of these losses.

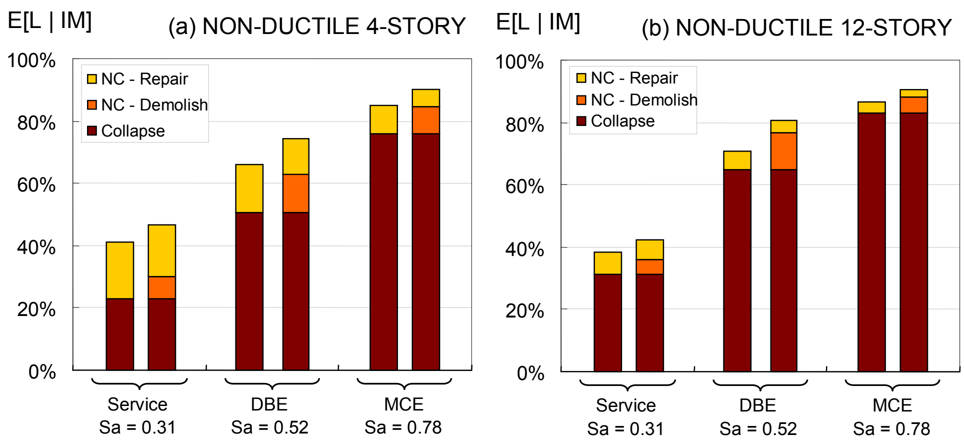

Figure 3.3 Loss results for non-ductile buildings studied (a) 4-story (b) 12-story.

In addition to showing total losses, Figure 3.3 also disaggregates the loss of non-ductile buildings. Comparing these disaggregations with those shown in Figure 3.2 it can be seen that collapse plays a much larger role in loss for the non-ductile buildings than it does for the ductile structures. According to these figures, the contributions of demolition losses to the overall losses are greatest at the DBE (unlike the ductile structures, where the demolition loss contributions were largest at the MCE). However, the demolition loss contributions are much smaller in these building than they are in the ductile structures (16% for the non-ductile 4-story and 15% for the non-ductile 12-story). Note that the collapse loss contributions are much larger than the other two types of losses. The 4-story non-ductile structure attributes 86% of its total loss to collapse, while the 12-story non-ductile structure accredits 89%.

4.1 SUMMARY AND CONCLUSIONS

An approach to expand and improve the PEER performance-based design methodology has been presented. The proposed approach now explicitly takes into account the fact that a building may have to be demolished after an earthquake. The economic loss conditioned on the level of ground motion intensity is computed as the sum of three terms: two terms previously identified corresponding to losses resulting if the building collapses and to losses associated with repairs given that the structure has not collapsed plus a third term which accounts for losses resulting from having to demolish buildings that have experienced excessive residual drifts.

Four case-study buildings were examined to investigate the impact of incorporating losses due to forced demolition on total economic losses. Results indicate that losses due to demolition can had a large influence in both the 4-story and the 12-story ductile reinforced concrete moment-resisting frame buildings that were considered in this study. Including losses due to demolition increased loss estimates for the DBE event by as much as 45% larger than the loss estimates that ignored its contribution. This may suggest at that current methods of loss estimation may be severely underestimating the building performance of these types of structures by not accounting for the effects of permanent displacement in structural damage. The influence of demolition losses was not as substantial in the 4-story and 12-story non-ductile reinforced concrete moment-resisting frame structures. These building exhibited a much larger probability of collapse at the ground motion intensities of interest, and consequently yielded economic loss results that were primarily due to collapse.

The proposed loss estimation methodology provides a powerful tool and framework for practicing engineers to quantifying differences in building performance between structures with systems that are susceptible to experience large permanent deformations and those that incorporate new self-centering lateral resisting systems. In particular the proposed approach to assess the tradeoffs and benefits of various design alternatives.

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