HOW HAVE CHANGES IN BUILDING CODE PROVISIONS FOR REINFORCED CONCRETE FRAME STRUCTURES IMPROVED SEISMIC SAFETY?

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ABSTRACT

This study provides an analytical comparison of seismic performance of a typical California office building designed according to the 1967 Uniform Building Code and the 2003 International Building Code. The seismic performance predictions are based on a performance assessment method developed by the Pacific Earthquake Engineering Research (PEER) Center, which employs incremental nonlinear dynamic time-history analyses.

Comparisons are made for a four-story reinforced concrete (RC) moment frame building designed to be representative of a) pre-1970 non-ductile reinforced concrete construction and b) modern (2003) ductile reinforced concrete construction. The plan and elevation of the building are identical for both structures; differences are evident in the magnitude of design loading, the relative strength of structural elements, and detailing of beams, columns, and beam-column joints. For each building, a nonlinear dynamic analysis model captures the behavior of the important failure modes up to the onset of collapse, accounting for uncertainties in structural behavior, modeling, and ground motions. The performance quantity of interest in this study is the collapse risk, particularly mean annual frequency of collapse. By comparing the computed collapse risk for the two structures, performance improvements in RC frame buildings over the three decades since the San Fernando earthquake can be quantified.

Introduction

Older reinforced concrete buildings, especially frame structures, are a prevalent and potentially vulnerable set of existing structures which may present a hazard to life safety. The risk of structural collapse, as it correlates with threat to life safety, can be quantified – for old or newer buildings - by applying the principles of performance-based engineering and nonlinear dynamic analysis. Collapse risk is of primary concern to engineering professionals responsible for building code development and to society in general.

The structure under consideration is a four-story reinforced concrete frame office building designed for a site in the Los Angeles area. Two designs are compared: the first with member sizing and detailing according to the 1967 Uniform Building Code (UBC), and the second

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according to the requirements of the 2003 International Building Code (ICBO 1967, ICC 2003). The 1967 design represents typical non-ductile reinforced concrete construction prior to when the UBC began to require ductile detailing in high seismic areas. The 2003 is a modern code-conforming design.

Analytical models in PEER's earthquake engineering simulation software, OpenSees (http://opensees.berkeley.edu), are used to quantify the structural collapse capacities and, through integration with the hazard curve, their mean annual frequencies of collapse. An explicit comparison of these two structures allows a direct examination of the increased collapse risk associated with older reinforced concrete frame structures, quantifying the often-mentioned "threat posed by existing buildings". This relative risk is a necessary input to policy decisions regarding retrofit of older structures.

Figure 1 shows the plan and elevation of the 1967 space frame building. The 2003 conforming building has the same layout, but is designed as a perimeter frame, as is judged to be more representative of current engineering practice. For each building, a two-dimensional model is created of a typical four-bay frame in the N-S direction. The designs represent typical rather than code minimum design, including expected overstrengths, etc. At the chosen site (Site Class D) the 2% in 50 year ground motion corresponds to $S_a(T = 1.0sec) = 0.82g$ and the site is not in a near-fault region (Haselton et al. 2005).

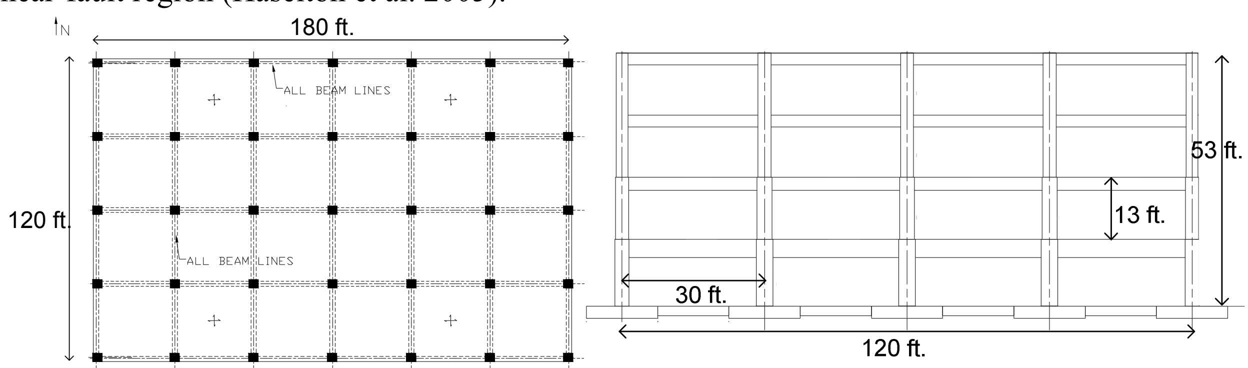

Figure 1. Plan and elevation of four-story office building.

The 2003 structure has a seismic base shear coefficient of 0.094g, corresponding to a calculated structural period $T_{code} = 0.80s$.[4] The perimeter frame columns range in size from 24 in. x 28 in. to 30 in. x 40 in.; beam dimensions are 24 in. x 32 in. to 24 in. x 42 in. The design was controlled by strength, the strong column-weak beam requirement, joint shear capacity provisions, and drift limitations (Haselton et al. 2005).

The 1967 design is based on $T_{code} = 0.40s$. The seismic base shear coefficient ($C = 0.05/T^{1/3}$) is 0.068g. The columns are square, varying in dimension from 20 in. to 24 in. Beam depths are between 20 and 26 in. The design was controlled by strength demands and the 1967 code has no strong column-weak beam requirement (ICBO 1967).

Many of the differences in the two designs lie in the detailing requirements. Column

[4] This value includes the maximum allowable 40% increase from the initial period calculated from the simplified IBC code formula (ICC 2003).

hoop-ties in the 1967 building are #3 at 10 in. on center, anchored with a 90° hook, with one set of cross-ties. Column lap splices are located at the base of the column in the region of maximum moment and are not detailed to prevent pullout. The 1967 UBC also had no specific provisions for joint design (ICBO 1967). In contrast, the 2003 structure has column hoop-ties that are typically #4 at 4 in. on center, with 135° hooks and several cross-ties at the location of each hoop. Lap splices are located at column mid-height or mechanical splices are used to develop full strength of reinforcing bars. The joints meet capacity design and confinement requirements (ICC 2003).

Methodology

The evaluation of these structures is based on the PEER performance-based earthquake engineering (PBEE) methodology which provides a framework for relating ground motion intensity (intensity measure ,or IM) to the structural response (termed engineering demand parameter, or EDP) through analytical models and structural simulation; the framework extends to the prediction of structural damage and monetary loss (Deierlein 2004). The application of the PEER methodology used in this paper focuses on collapse prediction and consists of inelastic dynamic response to simulate sidesway collapse using a two-dimensional model. Local collapse modes, which are difficult to simulate, are evaluated by combining predicted structural responses with fragility (or damage) models (Deierlein and Haselton, 2005).

To conduct an incremental dynamic analysis (IDA), a group of earthquake ground motions are selected and scaled to a specified earthquake intensity, nonlinear time history analysis is performed and the intensity of the earthquake is related to the structural response through an engineering demand parameter (in this case, peak interstory drift ratio). This procedure is repeated until reaching the IM level that causes dynamic instability (when drifts in one or more stories increase without bounds), resulting in sidesway collapse (Vamvatsikos 2002). Uncertainty associated with record-to-record variation is explicitly included in the IDA process. In this study, the spectral acceleration at 1 second, [S_a(T=1.0sec)], is used to characterize the earthquake intensity. Thirty-six ground motion records (each with two horizontal components) were selected based on magnitude, distance, and epsilon. (Goulet et al. 2006, Mitrani-Reiser et al. 2006)

The development of analytic simulation models requires explicit consideration of the possible collapse scenarios that may occur. Table 1 details possible collapse scenarios for reinforced concrete frame structures. They may include flexural hinging in beams and columns, shear failure in joints or beam-column elements, or reinforcing bar pull-out and splice failure. As Table 2 shows, few of these collapse scenarios need to be considered in the 2003 IBC structure; the strong column weak beam provisions, element and joint shear capacity design and other detailing requirements significantly reduce the likelihood of many of the collapse scenarios. In contrast, the range of collapse scenarios to be considered in the 1967 model is much greater. Joint shear failure or column shear failure, with a subsequent loss of vertical carrying capacity, are of particular concern. Simulation is conducted to model sidesway collapse caused by flexural (or flexure-shear) hinging in beams and columns and joint shear failure. Simulation of other deterioration modes is more difficult, and are considered through post-processing by combining structural response information from simulation (EDPs) with fragility functions that relate the EDP to damage (Elwood 2002, Aslani 2005).

Table 1. Possible collapse scenarios for reinforced concrete frame structures.

Sidesway Collapse Scenarios

Scenario	Description
FS1	Beam and column flexural hinging, forming sidesway mechanism
FS2	Column hinging, forming soft-story mechanism; a combination soft-story and sidesway mechanism (FS1/FS2) is also possible
FS3	Beam or column flexural-shear failure, forming sidesway mechanism
FS4	Joint shear failure, possibly with beam and/or column hinging
FS5	Reinforcing bar pull-out or splice failure in columns or beams, leading to sidesway mechanism

Vertical Collapse Scenarios

Scenario	Description
FV1	Column shear failure, leading to column axial collapse
FV2	Column flexure-shear failure, leading to column axial collapse
FV3	Punching shear failure, leading to slab collapse
FV4	Failure of floor diaphragm, leading to column instability
FV5	Crushing of column, leading to column axial collapse; possibly from overturning effects
FV6	Foundation failure due to uplift or overturning

Table 2. Likelihood of collapse scenarios for two different RC frame structures.

	Sidesway Collapse					Vertical Collapse				
	FS1	FS2	FS3	FS4	FS5	FV1	FV2	FV3	FV4	FV5
2003 IBC	H	L-M	L	L	L-M	L	L	L	L	L-M
1967 UBC	H	H	H	M	H	M	H	L	M	M-H

(blank)	Collapse mode not included in model, because occurrence is very unlikely.
(grey)	Collapse scenario simulated in OpenSees.
(hatched)	Collapse scenario incorporated through a combination of simulation and post-processing using fragilities.
(dotted)	Collapse mode not accounted for to date.

H: High
M: Medium
L: Low

Analytic Model

The structural models directly simulate the sidesway collapse of the RC frame structures. For this reason, a lumped plasticity model, which is able to capture the cyclic strength deterioration and the post-capping negative stiffness caused by rebar buckling and fracture, is used for beam-column elements (Ibarra 2003). Fig. 2 shows a schematic diagram of a portion of the model. Columns and beams are modeled with rotational springs at each end, which combine flexural or flexure-shear behavior and bond-slip in the joints. The joint elements account for the finite joint sizes and incorporate a spring to model joint panel shear (Altoontash 2003). The calibration of the joint panel shear spring is based on experimental data assembled by Mitra and Lowes (2005).

These models use a backbone/hysteretic model developed by Ibarra (2003), which incorporates cyclic deterioration and post-capping negative stiffness. Fig. 3 displays the element backbone model, including the key parameters: M_y, θ_y, K_s, θ_{cap}, K_c. Cyclic strength and stiffness deterioration are incorporated through two additional parameters. The properties of these concentrated plastic beam-column hinges have been developed through a combination of past research (Fardis et al. 2003) and calibration to tests of approximately 260 rectangular columns in the PEER Structural Performance Database (http://maximus.ce.washington.edu/ ~peera1). The authors' calibration work provides clear differentiation between models for well-detailed columns (like those in the 2003 IBC structure), and non-ductile columns (found in the 1967 structure.) Element models for non-ductile columns include a steeper post-capping slope, smaller plastic rotation capacity and faster cyclic deterioration, as shown in Fig. 3. The model parameters are assigned mean values rather than lower bounds to predict expected behavior and to reduce conservatism in the estimate of collapse. (Deierlein and Haselton 2005).

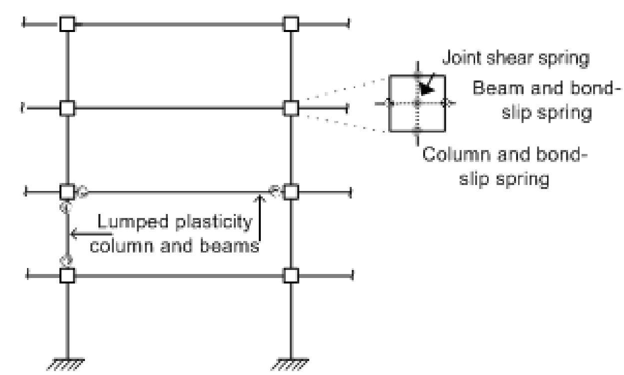

Figure 2. Schematic of OpenSees model.

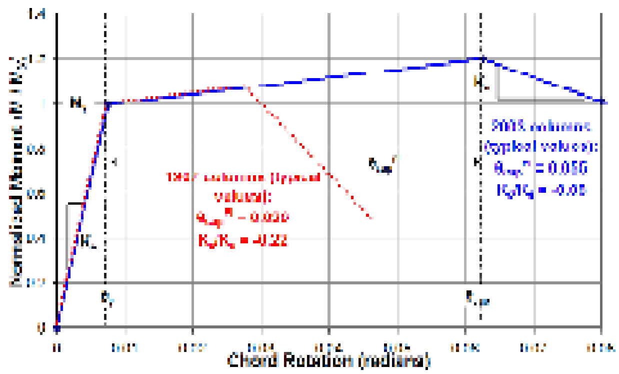

Figure 3. Element backbone curves.

Collapse Performance Assessment

2003 Modern RC Frame

As shown in Fig. 4, static pushover results for the code-conforming 2003 reinforced concrete frame reveal an overstrength of 2.5, as compared to the design base shear. From an eigenvalue analysis the fundamental period is 1.15s. This effective period is based on element cracked stiffnesses and includes flexibilities from bond and shear deformations. It neglects the effects of interior gravity frames; when these frames are included the period decreases to 1.0s. The geometry of the building (30 ft. column spacing and 13 ft. story heights) contributes to the structure's flexibility, and additional stiffness from nonstructural components is not considered.

Figure 5 presents the results from the incremental dynamic analyses. To approximate three-dimensional effects, we use only the controlling component for each earthquake record set, assuming the structure has collapsed when the first horizontal component causes collapse. Fig. 5 shows the results for the controlling components, resulting in median S_a at collapse of 2.20g and standard deviation $\sigma_{\ln,RTR}$ = 0.30g. The interstory drift ratio at collapse ranges from 0.07 to 0.12.

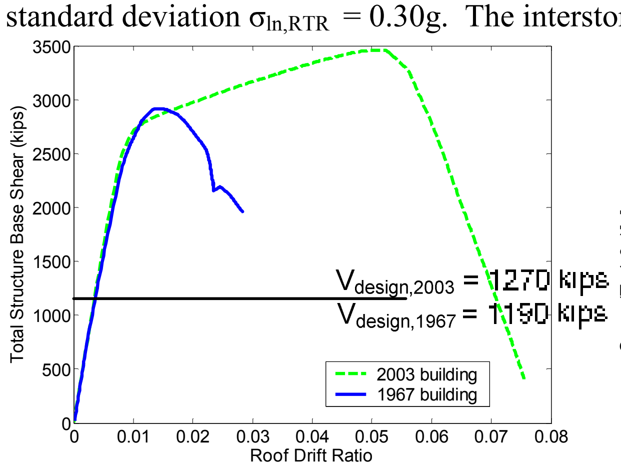

Figure 4. Static pushover results.

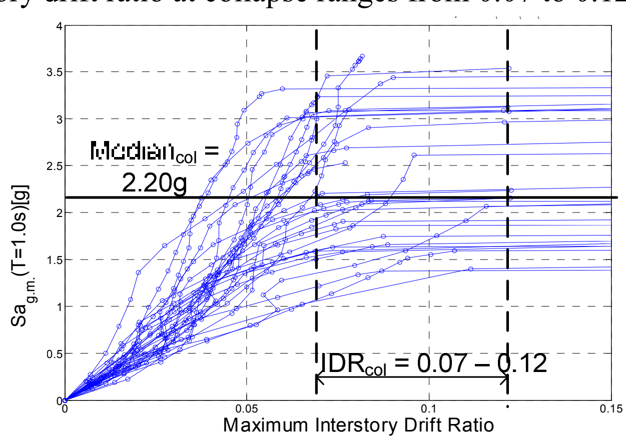

Figure 5. IDA results: 2003 Model.

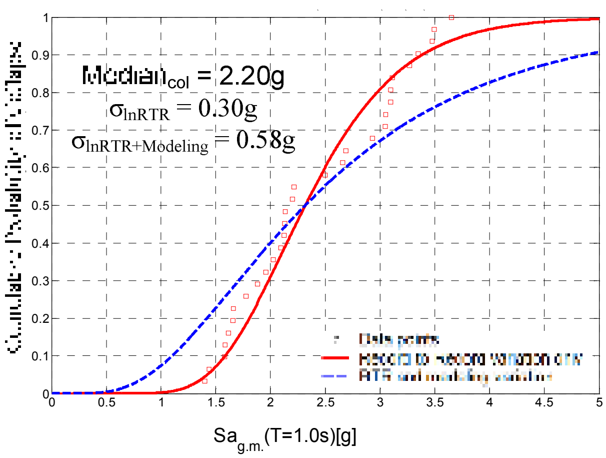

Figure 6. Probability of Collapse: 2003 model.

Figure 6 shows the cumulative probability of collapse given S_a level. For the 2% in 50 year ground motion for this site, $S_a = 0.82g$, the probability of collapse is less than 1% when considering only the uncertainty from variations between ground motions. Including modeling uncertainty, this probability increases to 4%, showing that estimation of modeling uncertainty is of critical importance to the collapse performance assessment. When integrated together with the hazard curve for the Los Angeles site, the mean annual frequency (MAF) of collapse is 0.8 x 10^{-4}. Only the sidesway collapse modes are considered.

1967 Non-Ductile RC Frame

The overall methodology for modeling collapse of the 1967-code non-ductile reinforced concrete frame is the same as that used for the modern frame, but the element model parameter values are quite different. Recalling Table 2, the design details of the 1967 structure also necessitate consideration of more failure modes. The potential for column shear failure and subsequent loss of vertical carrying capacity is accounted for in post-processing using a fragility model.

Again, the results of the static pushover are shown in Fig. 4, indicating an overstrength of approximately 2.5. The fundamental period, as calculated from an eigenvalue analysis, is 1.35s. The lower ductility of the 1967 frame is clearly evident these results.

Figure 7 displays the results of the incremental dynamic analyses with the same 36 records. The median S_a, considering just the controlling components, is 0.96 g, with $\sigma_{ln,RTR}$ = 0.30g. In comparison to IDA results for the 2003 frame we find that the 1967 has lower strength (median $S_{a\ col}$ = 0.96g compared to median $S_{a\ col}$ = 2.20g) and much lower ductility (failing at peak interstory drift ratios between 0.03 and 0.06, rather than in the range of 0.07 and 0.12). The cumulative probability of collapse, given S_a, is shown in Fig. 8. Where only record to record variation is considered Fig. 8 shows a 0.26 probability of collapse under the 2% in 50 year ground motion. With modeling uncertainty this probability increases to 0.37 and, when integrated with the hazard curve, the mean annual frequency of collapse is 15 x 10^{-4}.

The possibility of column shear failure and loss of vertical carrying capacity is incorporated through the use of fragility curves defined by Aslani (2005). These curves define for each column the median and standard deviation of column drift ratio at shear failure and, subsequently, at the point the column loses its ability to carry gravity loads and collapses vertically. From these relationships it is possible to compute the probability of column shear failure (or loss of vertical carrying capacity) in the most vulnerable column, given that sidesway collapse has not occurred: $P[C_{other}|NC_{sim}, IM = im]$. The total probability of collapse can be computed from the total probability theorem (Aslani 2005, Deierlein and Haselton 2005):

$$P[C \mid IM] = P[C_{sim} \mid IM] + P[C_{other} \mid NC_{sim,} IM] P[NC_{sim} \mid IM] \tag{1}$$

where C_{sim} is simulated sidesway collapse, C_{other} is another collapse scenario, and NC_{sim} is no sidesway collapse.

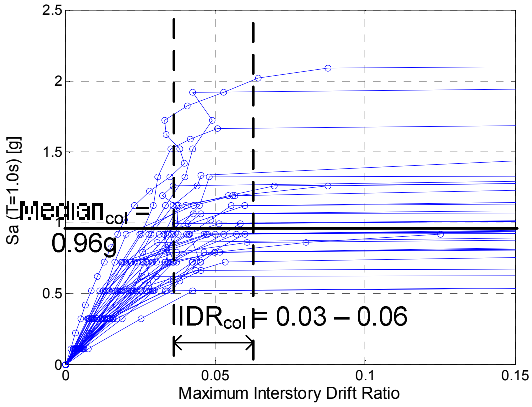

Figure 7. IDA results: 1967 model.

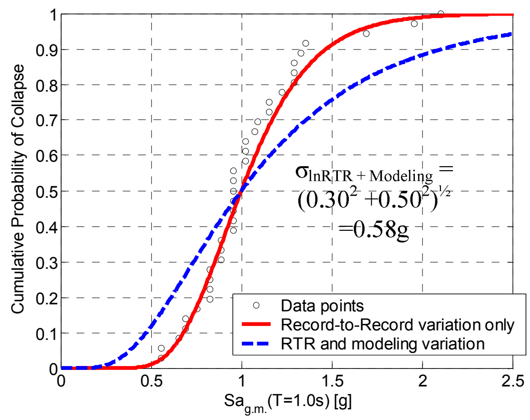

Figure 8. Probability of collapse for 1967 model, including only sidesway collapse.

Equation (1) allows us to re-compute the probability of collapse for the 1967 structure including these additional failure modes. If collapse is considered to be either sidesway collapse or the shear failure of at least one column, the median S_a at collapse is reduced from 0.96g to 0.65g, as shown in Fig. 9. The conditional probability of collapse at the 2% in 50 year level is 0.70. However, research by Elwood and others has shown that it is highly conservative to assume that the onset of column shear failure corresponds to structural collapse (Elwood 2005). Therefore, Fig. 9 alternatively defines structural collapse as either sidesway collapse or loss of axial load carrying capacity of a column (precipitated by column shear failure, but occurring at higher drift levels). Recalculating, P[Collapse| 2% in 50 yr. ground motion] = 0.29, which is only slightly higher than the sidesway only case. Combining the total probability of collapse from sidesway and loss of vertical carrying capacity in the columns and the uncertainty associated with modeling, P[Collapse| 2% in 50 yr. ground motion] is 0.39. The computed mean annual frequency of collapse at the Los Angeles site is 18 x 10^{-4}.

As Fig. 9 shows, defining column shear failure as collapse greatly increases the probability of collapse, whereas including column loss of vertical carrying capacity has a fairly small impact on the overall collapse assessment. For a typical column in this structure, Aslani's fragility information predicts a median column drift ratio at shear failure of 0.021 (Aslani 2005, Elwood 2002). Referring to Fig. 7, this corresponds[5] to an interstory drift ratio of approximately 0.028, where sidesway failure has not occurred in many cases. In contrast, the median column drift ratio at loss of vertical carrying capacity for a typical column is 0.047 (interstory drift ratio ≈ 0.054), where the majority of the records have already collapsed in the sidesway mode. Therefore, shear failure has a larger impact on collapse assessment than loss of vertical carrying capacity because it may occur before sidesway collapse. It is important to note that since shear failure is not simulated this analysis misses important feedback loops between shear failure and incremental dynamic analyses, such as structural softening after column shear failure. The mean

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[5] Column drift ratio includes only that drift that is undergone by the columns, and is smaller than the total interstory drift ratio. The approximation relating column drift ratio and total interstory drift ratio is based on Aslani (2005).

annual frequency of collapse, including sidesway collapse and column shear failure and modeling uncertainty, is 46 x 10^{-4}, can be considered an upper bound on the collapse limit state since we are not simulating shear failure.

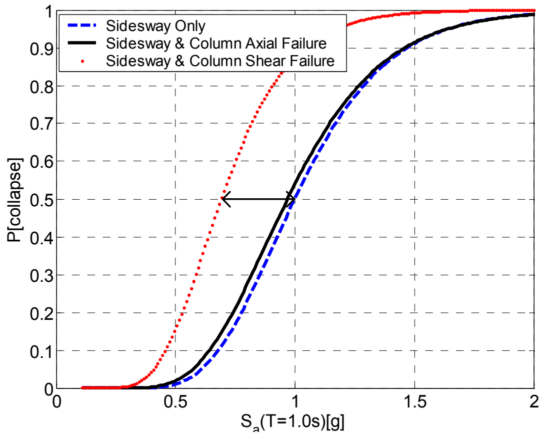

Figure 9. Probability of collapse for 1967 model, considering sidesway collapse and additional failure modes, including a) column shear failure and b) column loss of vertical carrying capacity.

Comparisons to FEMA 356 Assessment

The results of this methodology can be compared to results from simpler analytical methods such as the nonlinear static analysis described in FEMA 356. Under these FEMA guidelines a pushover is performed with reference to a specified target displacement and results are compared to the element performance criterion (ATC 2000). For this structure, the computed target displacement for the 2% in 50 year ground motion corresponds to a roof drift ratio of 0.019 for the 1967 building and 0.016 for the 2003 building (Fig. 4).

Figure 10 compares the distribution of interstory drift ratios (IDRs), an indicator of structural damage, from the pushover analysis and dynamic analysis procedures. The distribution of IDRs from the pushover analyses are taken at the target displacement.[6] The distribution of interstory drift ratios from dynamic analysis are obtained from two selected ground motions (one above and one below the median collapse levels from IDA) and S_a levels are close to collapse. As Fig. 10 shows, the distribution of IDRs during each dynamic analysis differs, depending on both the earthquake record and its intensity level. Given the variability in the dynamic analysis results, the distribution from the pushover matches the dynamic results more closely in some cases than in others. The static pushover analysis does not capture this potential variability. Whether the distribution of drift ratios from a pushover analysis represents a central tendency of the IDAs is difficult to judge, and would need to be researched further.

As a final step in the FEMA analysis the plastic hinge rotations in the critical columns at the target displacement from the pushover analyses are compared to the acceptance criteria. (In concept, this process would be repeated for other structural elements as well.) The critical column in the 2003 model has a plastic hinge rotation of 0.007 radians, meeting FEMA's life safety limit state of 0.015 radians for conforming columns with low axial load ratio. The 1967 model has a plastic hinge rotation of 0.016 radians in the most critical column, greatly exceeding the collapse prevention limit state of 0.006 radians for non-conforming columns and indicating a much higher likelihood of collapse (ATC 2000).

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[6] These pushover analyses follow the FEMA procedure except that the element backbones uses the Ibarra element model in OpenSees, and not the more conservative FEMA 356 backbone parameters.

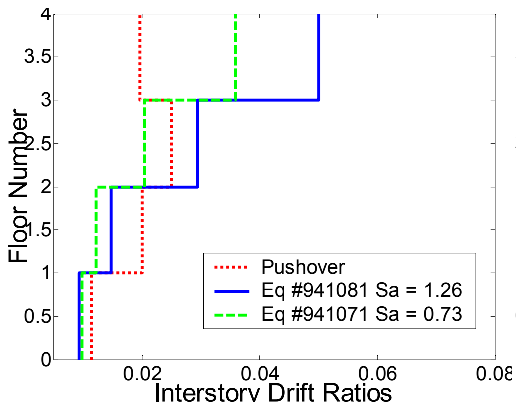

Figure 10a. 1967 model.

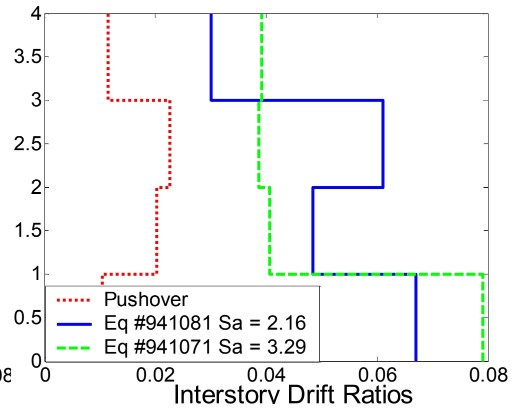

Figure 10b. 2003 model.

Figure 10. Distribution of interstory drift ratios over the height of the building: 1) at the target displacement during static pushover and 2) near collapse during two selected dynamic analyses.

Conclusions

The application of the PEER PBEE methodology to two structures with the same plan and elevation, but based on different design and detailing provisions, explicitly demonstrates the improvement in building code requirements over the last 30 years. The results are consistent with our engineering judgment that the older reinforced concrete frame structure is more hazardous to life safety than the modern well-detailed concrete frame. Moreover, the analytical models allow us to directly quantify the relative difference in safety. We find the non-ductile frame has less than half the collapse capacity and approximately half the ductility, which corresponds to an increase in probability of collapse at the ground motion corresponding to the 2% in 50 year event from 4% to 39% for the 1967 structure, and a significant increase in the mean annual frequency of collapse from 0.8 x 10^{-4} to 18 x 10^{-4} [collapses/year].

The use of a performance-based methodology, such as the one described here, to predict mean annual rates of collapse requires consideration of an acceptable or target level of risk. Does the earthquake engineering community – and the society we aim to protect – accept risks with a mean annual frequency of collapse for an individual building of 10^{-4}, or 10^{-3}? For a given structure, what collapse rate is too high, and, if it is too high, how much of a reduction is necessary? A related question asks over what range of collapse risk is cost-benefit analysis relevant. We can begin to answer these questions by examining the other hazards in our society and understanding how this seismic hazard compares to the other risks that we face. The field of engineering risk analysis and the experiences of other groups such as the Nuclear Regulatory Commission and the off-shore oil industry provide valuable reference points (Pate-Cornell 2002). If combined with a consensus regarding appropriate collapse risk, the framework for collapse analysis can have significant applications to the refinement of building code provisions (for new buildings) or the development of retrofit policies targeting particularly hazardous older buildings. It may be possible, for example, to identify those buildings or sub-classes of buildings within non-ductile concrete frames for which expenditure for seismic retrofit is particularly beneficial.

Planned extension of this work includes: a) comparing design variations within the broader classification of older non-ductile frame structures, b) investigation of appropriate collapse risk (both for retrofit and for new construction), and c) relating collapse risk of individual buildings to the risk of groups of buildings in an urban area (related to spatial correlations of ground motion).

Acknowledgments

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