Benefit-cost evaluation of seismic risk mitigation in existing non-ductile concrete buildings

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Abstract

The risks of damage and collapse to older (non-ductile) reinforced concrete buildings and the cost-effectiveness of seismic retrofit are investigated through analyses of archetypical designs representative of construction in California prior to the introduction of more rigorous seismic design requirements in the mid-1970's. These risks for older buildings are compared to those in buildings that are designed to modern building code provisions that employ capacity design and ductile detailing requirements. The comparisons indicated that older non-ductile buildings have expected annual economic losses that are about twice those of the ductile buildings and risks of collapse and fatalities that are about 35 times higher. The cost effectiveness of seismic retrofit is examined to reduce damage and life safety risks. Considering the monetary benefits of both reduced damage and lives saved, these cost-benefit comparisons justify retrofit costs of up to about 20% to 40% of the building replacement value, implying that in most cases the retrofit of non-ductile concrete buildings would be cost-effective.

Introduction

Emerging performance-based earthquake engineering methods can offer significant new insights to the systematic evaluation of design criteria and policy-related questions for new and existing buildings. In this study, recently developed performance-based technologies are applied to assess the comparative performance of older "non-ductile" reinforced concrete (RC) buildings versus modern "ductile" buildings that employ capacity design approaches and ductile reinforcing bar detailing. Concerns with older non-ductile designs stem from buildings constructed in the high seismic regions of the western United States, prior to the mid-1970's when major changes were instituted for seismic design of RC structures as a result of damage observed during the 1971 San Fernando earthquake. Prior to these changes, seismic design requirements for concrete frames did not require capacity design provisions to inhibit the formation of story mechanisms or column shear failures. While it is generally recognized that such buildings do not provide the same level of safety and damage control as modern buildings, there are many debates as to the safety of existing buildings and whether policies should be adopted to require detailed risk assessment and mitigation.

This study aims to improve understanding of earthquake risks in non-ductile RC buildings and the cost-effectiveness of mitigating these risks through building replacement or retrofit. Performance-based methods are applied to assess the risk of damage and collapse to a set of archetypical RC-framed office buildings that are representative of those in the high-seismic regions of California. A related objective is to illustrate the application of the performance-based cost-benefit analyses, which can be generally applied to other building types in other seismic regions.

Performance Assessment Methodology

The performance assessment follows an approach developed by the Pacific Earthquake Engineering Research (PEER) Center that provides a systematic formulation to characterize earthquake ground motions, structural response, building damage, and ultimately decision metrics related to economic losses and life safety risks (Deierlein 2004, Krawinkler and Miranda 2004). Key aspects of the implementation are summarized below, and the reader is referred to the underlying studies by Liel and Deierlein (2008) and Haselton and Deierlein (2007) for further details.

Structural Demand Parameter Assessment: The structural response is determined through nonlinear dynamic analyses of two dimensional RC frame models that characterize the significant design features of the archetype buildings. As shown in Figure 1, the frame models utilize nonlinear springs to characterize the inelastic behavior of beams, columns, beam-column joints, and foundations. The three-bay configuration and finite joint elements reflect the important design-related aspects of interior versus exterior joints and column behavior. The analyses are conducted using the *OpenSees* simulation platform (http://opensees.berkeley.edu). The dynamic time history analyses are organized using the so-called *incremental dynamic analysis* procedure, whereby the response is calculated for specific ground motions that are scaled by increasing intensity (often spectral acceleration intensity, scaled at the first-mode period) up to the point of structural collapse. At each intensity level, statistics of structural response parameters, such as peak story drift or plastic hinge rotations, are compiled to characterize the response.

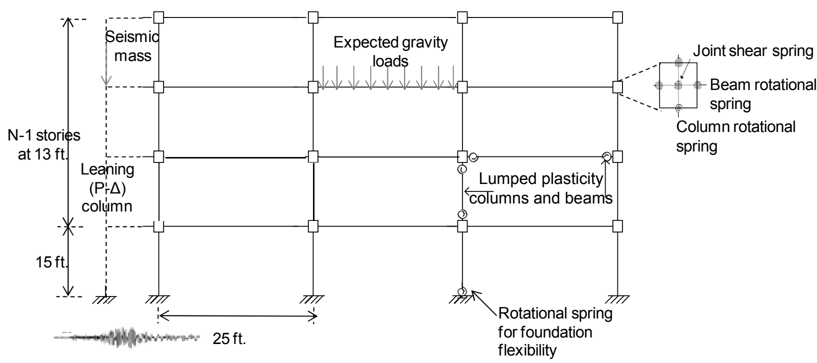


Figure 1. Schematic of the RC frame structural analysis model

Collapse Risk Assessment: Collapse risk statistics are determined from the dynamic analyses, but considering three important adjustments. First, where certain failure modes are not directly simulated in the nonlinear analysis, these are checked through a post-processing of the data to adjust the dynamic analysis results. For example, shear failure and loss of axial load capacity of non-ductile RC columns is a mode of failure that is not simulated directly in the analysis and is incorporated through the post-processing operation. A second adjustment made is to collapse capacity to account for the spectral shape effects in extreme (rare) ground motions that are not reflected in the input ground motions used for the dynamic analyses.

Following an approach described by Haselton and Deierlein (2007), the median collapse capacity (described in terms of spectral acceleration) is adjusted to reflect the characteristic spectral shape of the dominant hazard spectra for the rare ground motions. Finally, the dispersion in collapse capacities from the dynamic analyses is adjusted to consider so-called modeling uncertainties. For the RC frames, the modeling uncertainty increases the total dispersion (standard deviation of the natural logarithm of collapse intensities) from about 0.4, due just to ground motion variability, to about 0.6.

Damage and Loss Assessment: The damage and associated costs for building repair or replacement are calculated considering damage to both structural and nonstructural components. Losses were determined using a toolbox of damage fragility and loss functions developed by Mitrani-Reiser (2007) and applied in a related study by Goulet et al. (2007). The loss components were based on a typical architectural office building layouts, such as shown in Figure 2.

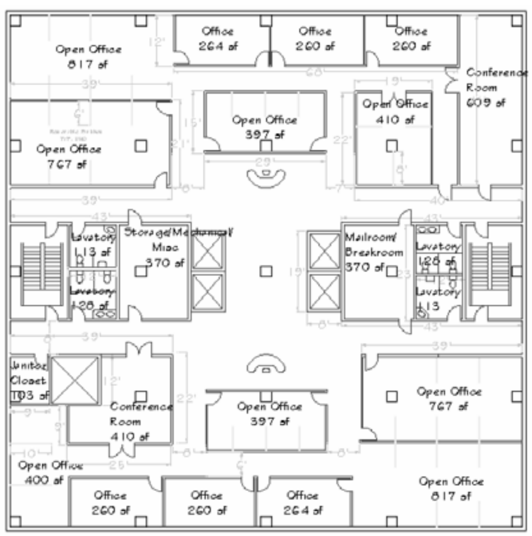


Figure 2. Architectural floor plan of archetype building

Fatality Risk and Loss Assessment: The risks of human fatalities due to building collapse are calculated considering the average building occupancy, the building collapse volume ratio, and statistics on fatalities for persons trapped in collapsed buildings. Parameters assumed for this study include: occupancy density 1 person per 23 m^2 of office space, average occupancy rate of 0.33, trapped occupant rates of 0.3 to 0.6 for collapsed buildings, and an average fatality rate of 0.17 for trapped persons.

Comparative Assessment of RC Building Archetypes

Eight representative building designs were developed and analyzed to generalized performance of buildings with non-ductile and ductile characteristics. As summarized in Table 1, the buildings ranged in height from 2 to 12 stories and utilized either two-way space (S) frame or perimeter (P) frame configurations, e.g., 2S = 2-story space frame. The non-ductile and ductile RC frames were designed according to the 1967 and 2003 building code provisions, respectively, for a typical site in southern California (ICBO 1967 and ICC 2003).

Summarized in Table 1 are the key performance assessment results for each frame, including parameters to characterize the risks associated with collapse, economic loss, and fatalities. One measure of the collapse safety is the *Collapse Margin Ratio* (CMR), which is the ratio between the median collapse spectral acceleration capacity (S_{CT}), as obtained from the dynamic analyses, to the ground motion intensity with a 2% chance of exceedence in 50 years ($Sa_{2/50}$). The resulting collapse fragilities reveal dramatic differences in collapse capacity for the non-ductile versus ductile frames, where the former have collapse probabilities on the order of 0.65-0.85 under the $Sa_{2/50}$ ground motions, as compared to probabilities of 0.05-0.15 for the ductile frames. When the collapse fragilities are integrated with the seismic hazard curve, the resulting *Mean Annual Frequencies of collapse* (*MAFc*) for the non-ductile frames are about 35 times larger (on average) than the ductile frames. Ratios of *Expected Annual Fatality* (*EAF*) rates mirror the collapse rates since the relationships between the two are based on expected values.

In contrast to the collapse and fatality risks, the *Expected Annual Losses (EAL*) associated with building damage and repair are only about twice as large for the older non-ductile frames as compared to the modern ductile frames. As shown in Table 1, the *EAL* are about 0.8 to 1.3% of the building replacement value for the ductile frames and 1.6 to 5.2% for the non-ductile frames. The differences in loss behavior between the two are further illustrated in Figures 3a and 3b. Whereas the economic losses in the ductile frames tend to be dominated by the non-collapse condition at relatively low ground motion intensities, the losses for the non-ductile frames have a much larger contribution from building collapse. The relative performance is further evident by differences in losses under the design earthquake accelerations, indicated by the spectral intensity value S_D, in Figures 3a and 3b.

Whether or not the large apparent difference in safety between older versus modern buildings would warrant proactive mitigation steps,

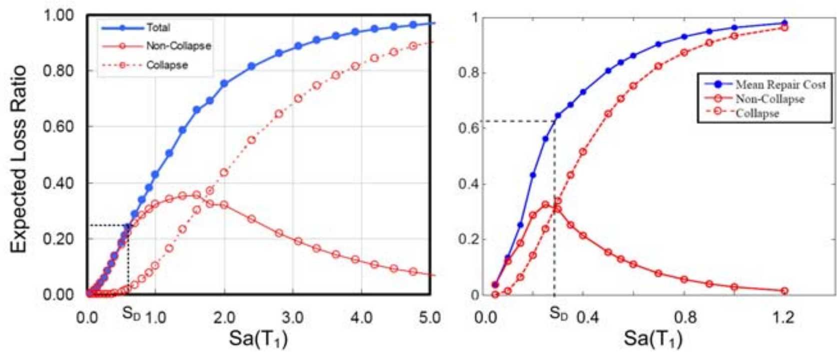
Table 1a. Collapse performance results for non-ductile RC frames

Structure	Ω	S_{CT} [g]	$Sa_{2/50}$ [g]	CMR	MAFc x 10^{-4}	EAL %	EAF x 10^{-3}
2S	1.9	0.47	0.80	0.59	109	5.2%	41
2P	1.6	0.68	0.79	0.85	47	3.2%	24
4S	1.4	0.27	0.49	0.54	107	2.3%	62
4P	1.1	0.31	0.47	0.66	100	2.3%	97
8S	1.6	0.23	0.31	0.75	64	1.8%	77
8P	1.1	0.29	0.42	0.68	135	2.1%	141
12S	1.9	0.29	0.35	0.83	50	1.6%	76
12P	1.1	0.24	0.42	0.56	119	1.6%	192

Table 1b. Collapse performance results for ductile RC frames

Structure	Ω	S_{CT} [g]	$Sa_{2/50}$ [g]	CMR	MAFc x 10^{-4}	EAL %	EAF x 10^{-3}
2S	3.5	3.55	1.16	3.07	1.0	1.0%	0.4
2P	1.8	2.48	1.13	2.19	3.4	1.0%	1.7
4S	2.7	2.22	0.87	2.56	1.7	1.1%	1.3
4P	1.6	1.56	0.77	2.04	3.6	1.2%	2.7
8S	2.3	1.23	0.54	2.29	2.4	1.3%	3.1
8P	1.6	1.00	0.57	1.77	6.3	1.0%	8.3
12S	2.1	0.83	0.44	1.91	4.7	1.1%	9.4
12P	1.7	0.85	0.47	1.84	5.2	0.8%	9.9

Ω: over strength, ratio of ultimate strength to design strength from pushover analysis
S_{CT}: median collapse intensity, based on Sa(T1)
$Sa_{2/50}$: ground motion intensity, Sa(T1), with 2% in 50 year chance of exceedence
CMR: Collapse Margin Ratio = S_{CT} / $Sa_{2/50}$
MAFc: Mean Annual Frequency of Collapse, collapses per year.
EAL: Expected Annual Loss as a percentage of building replacement value
EAF: Expected Annual Fatalities, fatalities per year



(a) (b)

Figure 3. Loss contributions for 4-story space frame buildings (a) ductile design and (b) non-ductile design

such as through mandatory building retrofit or replacement, is an important and vexing question. To examine whether it would be worthwhile, based purely on economic considerations, to replace existing non-ductile buildings with new ductile buildings, the costs and benefits are compared in Table 2. Here the benefits of building replacement are considered to be the savings in economic losses associated with repair and the lives saved, calculated based on the relative risks to the non-ductile and ductile buildings. To compare the benefits on a pure economic basis, the value of life is assumed as 2 million dollars, and the value of the benefits is calculated over an assumed 50-year building life with an annual discount rate of 3%. The cost to achieve these benefits is assumed to be the building replacement value, based on average building costs of about $1,500 to $1,800 per square meter. As indicated in the last column of Table 1, with the exception of the 2-story space frame building, the cost-benefit ratios all violate the breakeven value of 1, indicating that the replacements are not justified on the basis of economic costs.

Cost-benefit assessment of building retrofit

Whereas the previous analysis only considered the cost-benefits of building replacement, a more likely option for risk mitigation is structural retrofit. To examine this, the following three alternative retrofit methods were examined for the 4-story and 8-story space

Table 2. Benefits and costs of replacing non-ductile RC frame structures

Building	Benefits over 50 years			Building Replacement Cost ($, M)	Cost-Benefit Ratio
	Losses Avoided ($, M)[1]	Lives Saved[2]	Total Benefits[3] ($, M)		
2S	6.5	2.0	8.6	6.1	0.7
2P	3.7	1.1	4.8	6.5	1.4
4S	3.9	3.1	7.0	12.5	1.8
4P	3.5	4.7	8.3	12.0	1.4
8S	2.7	3.7	6.5	19.9	3.1
8P	5.3	6.6	12.1	19.4	1.6
12S	4.2	3.3	7.6	29.1	3.8
12P	6.2	9.1	15.6	28.1	1.8

[1]Present value of losses avoided by building replacement, millions of US dollars.
[2] The estimated total number of lives saved by building replacement.
[3] Sum of the net present value (discounted over 50 years) of losses avoided and lives saved. The dollar value of lives saved is based on $2 million/life, discounted over 50 years based on annual discount rate of 3%.

frame buildings: (1) Fiber – wrapping of columns with fiber composites to improved their ductility, (2) Column Jacketing – reinforced concrete jacketing of columns to improve both their strength and ductility, and (3) Wall Piers – addition of wall piers to add strength and ductility to the system. As described by Liel and Deierlein (2008), the retrofit designs are fairly modest and intended to impact minimally the building architecture.

Performance assessment results of the retrofit designs are summarized in Table 3, using the same metrics previously discussed in Tables 1 and 2. In general, fiber wrap retrofits were only marginally effective, since they did not reduce the tendency for story collapses, and the benefits of improved column ductility were marginal. On the other hand, the column strengthening and wall retrofits, which inhibit story mechanisms, provide the largest benefits, as evidenced by increases in the static pushover index, Ω, and commensurate reductions in the risk of collapse (*MAFc*) and fatalities (*EAF*).

The relative amount of benefits gained by reduction in damage losses versus life safety risks can be inferred by comparing the "losses avoided" and "total benefits" columns of Table 3. For the 4-

story building, the total benefits of the retrofit were due in about equal share to reductions in damage losses and lives saved. For the 8-story building, proportionately more benefit was derived from the economic benefits of lives saved.

Table 3. Comparative benefits of retrofitting non-ductile frames

Structure[1]		Ω	Annualized Losses[2]			Benefits over 50 years[2]		
			MAFc x 10^{-4}	EAL (% rep.)	EAF (x 10^{-3})	Losses Avoided (% repl.)	Lives Saved	Total Benefits (% repl.)
4S	1967	1.4	107	2.3%	62	**	**	**
	Fiber	1.4	75	2.1%	50	5%	0.6	10%
	CJ	3.5	7	1.2%	9	28%	2.7	49%
	WP	2.1	20	1.8%	27	13%	1.8	27%
	2003	2.7	2	1.1%	1.3	31%	3.1	56%
8S	1967	1.6	64	1.8%	77	**	**	**
	Fiber	1.6	60	1.6%	68	5%	0.5	8%
	CJ	2.1	6	1.4%	9	10%	3.4	28%
	WP	2.2	7	1.5%	11	8%	3.3	25%
	2003	2.3	2	1.3%	3.1	14%	3.7	32%

[1]Structure types include the original 1967 non-ductile design and retrofits based on: Fiber wrapping, Column Jacketing, and addition of Wall Piers. Results are also shown for 2003 code-conforming ductile design.

[2]Definitions for annualized losses and benefits are the same as in Tables 1 and 2.

Whether or not the retrofits are cost-effective will depend upon the cost of the retrofits. To achieve a favorable cost-benefit ratio less than 1, the retrofit cost would need to be less than the total benefits, as given by the last column of Table 3. For example, the CJ retrofit of the 4-story building could cost up to 49% of the building value and still have a favorable cost-benefit ratio. On the other hand, the least effective retrofit, fiber wrapping of columns in the 8-story building, could only cost up to 8% of the building value. Assuming that building structural retrofits generally cost in the range of $400 to $800 per square meter of building area, and assuming a typical building value of $1,800 per square meter, those retrofits with costs less than 20% to 40% of the building replacement value would have favorable cost-benefit ratios. In practice, one could imagine that thoughtful design of retrofit solutions would further improve their effectiveness and reduce their costs to improve their attractiveness.

Conclusions

As illustrated in this paper, by articulating seismic performance in terms of explicit life-safety and economic metrics, performance-based engineering approaches provide the opportunity to evaluate the relative safety of seismically deficient existing buildings with modern code-conforming buildings. For example, the comparative analyses show the older non-ductile RC buildings to have risks of collapse and fatalities that are about 35 times larger for modern buildings, and damage losses that are about 2 times larger. When applied in conjunction with cost-benefit analyses, the methods can help to inform decisions by owners and other public policy makers on the cost-effectiveness of building retrofit or replacement to mitigate earthquake risks. For the non-ductile buildings investigated in this study, retrofit is shown to be a cost-effective option for risk mitigation.

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References

Deierlein, G.G. (2004), "Overview of a Comprehensive Framework for Earthquake Performance Assessment", *Performance-Based Seismic Design Concepts and Implementation: Proceedings of the International Workshop PEER TR 2004/05,* 15-26.

Goulet, C. A., Haselton, C. B., Mitrani-Reiser, J., Beck, J. L., Deierlein, G. G., Porter, K. A., and Stewart, J. P. (2007). "Evaluation of the Seismic Performance of a Code-Conforming Reinforced-Concrete Frame Building - from Seismic Hazard to Collapse Safety and Economic Losses." *Earthquake Engineering and Structural Dynamics*, 36(13), 1973 - 1997.

Haselton, C.B. and G.G. Deierlein (2007). *Assessing Seismic Collapse Safety of Modern Reinforced Concrete Frame Buildings, PEER TR 2007/08.*

ICBO. (1967). *Uniform Building Code*, Pasadena, CA.

ICC. (2003). *International Building Code*, Falls Church, VA.

Krawinkler, H. and Miranda, E. (2004). "Performance-Based Earthquake Engineering." In Y. Borzognia and V. Bertero (Ed.), *Earthquake Engineering: From Engineering Seismology to Performance-Based Engineering, 1st edition* (pp 9-1 to 9-59). CRC Press.

Liel, A. B and G.G. Deierlein (2008). *Assessing the Collapse Risk of California's Existing Reinforced Concrete Frame Structures: Metrics for Seismic Safety Decisions,* B*lume Earthquake Engineering Center, TR No. 166,* Stanford University.

Mitrani-Reiser, J. (2007). *An Ounce of Prevention: Probabilistic Loss Estimation for Performance Based Earthquake Engineering,* Doctoral Dissertation, Caltech.