USING COLLAPSE RISK TO INFORM SEISMIC SAFETY DECISIONS: CALIFORNIA'S EXISTING REINFORCED CONCRETE STRUCTURES

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ABSTRACT

Recent developments in earthquake engineering research have enabled simulation and calculation of structures' risk of collapse under seismic loading, providing explicit measures of collapse risk which can be used to compare the risks posed by different types of structures. Potential policy applications of these measures are considered in the context of the many non-ductile concrete structures in California which may need retrofitting. Historically, seismic safety legislation has been largely driven by damage in past earthquakes, which have triggered legislative action to mitigate a certain set of structural seismic hazards. Metrics that enable assessment and comparison of collapse risk provide a mechanism for standardization and prioritization in the seismic safety policy process. As an example, the collapse risk of an archetypical older concrete moment frame structure is assessed using the framework of performance-based earthquake engineering.

INTRODUCTION

A recent article in the *Los Angeles Times* posed the question, "How Risky Are Older Concrete Buildings?" The answers presented represent two sides of the debate over the importance of retrofitting these structures. One expert is quoted stating, "It's well recognized within the engineering professional community that many California non-ductile buildings are *at unacceptable risk of collapse* in moderately strong shaking." But, the article continues "building owners and business organizations have long fought efforts to require retrofits, *arguing the risk is overstated."*(Bernstein 2005) Recent advances in earthquake engineering, including improved knowledge of the ground motion hazard, availability of component test data and simulation technologies to evaluate response, enable analysis of the structural collapse limit state under seismic loading. Using collapse analysis within the framework of performance-based earthquake engineering the risk posed by older concrete buildings can be explicitly quantified, permitting comparison between classes of vulnerable buildings, and with other societal risks. The risk of collapse of California's approximately 40,000 non-ductile concrete buildings and their impact on public safety is a subject of public and media interest, especially with this year's anniversary of the 1906 San Francisco earthquake and the destruction of concrete structures in the recent Pakistan earthquake.

In this paper, the collapse risk of California's existing reinforced concrete buildings is examined from two, seemingly disparate, perspectives. The first section describes state-wide efforts to legislate the safety of existing buildings. For many in the engineering community, older reinforced concrete buildings and soft-story structures are the needed next step in seismic safety legislation. Yet, its history demonstrates that this legislation has been largely triggered by high-profile collapses of structures in past damaging earthquakes. Modern assessment methods, which are the focus of the second section of this paper, can help reduce reliance on these triggering events, by enabling and motivating a detailed examination of seismic safety and vulnerable buildings before the next devastating earthquake. The application of the collapse assessment methodology is presented for an archetypical existing concrete moment frame. Assessments of seismic performance metrics such as mean annual frequency of collapse and probability of collapse of these types of structures are

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important inputs to a risk-informed seismic safety policy process.

MITIGATING HAZARDS FROM EXISTING BUILDINGS

Legislative Precedents

Reduction of collapse risk presents the most significant motivation for government intervention in the seismic retrofit process. The seismic safety of new buildings in California is vetted through the well-established process of creation and modification of the seismic provisions of building codes, which are regularly updated to incorporate improvements in design and construction. As such, these provisions are largely the province of the engineering and building code communities. By contrast, the safety of existing buildings falls within the jurisdiction of the legislative process, where the issue must compete with the myriad other demands on public and legislators' attention. Mandating particular levels of seismic safety in existing buildings has been further complicated by potentially large costs and uncertainties associated with upgrading structures.

A 1970 SEAOC resolution urged "the State of California to adopt the laws necessary to provide a uniform program to require the correction of the many unsafe buildings throughout the state."(Murphy 1973) The history of these efforts shows that seismic safety legislation has been largely piecemeal, and has occurred primarily in response to damaging earthquake events. California's Field Act, Hospital Safety Act and Un-reinforced Masonry Act are discussed below. Each of these aims to ensure life safety by instituting provisions to reduce the likelihood of collapse. Still, these laws leave many potentially vulnerable existing structures unregulated, and the process lends itself toward inconsistent safety standards among different types of existing construction. Their challenges indicate the difficulties faced by seismic safety advocates in addressing the collapse risk posed by non-ductile concrete structures in the State.

The Field Act

The 1933 Long Beach earthquake provided the impetus for the first policies to mandate safety of existing buildings in California. Fortunately, classes were not in session when the earthquake occurred (5:55 PM), since the magnitude 6.3 quake damaged 75% of public school buildings in the city. Not surprisingly, a public outcry about the lack of seismic safety of structures erupted, and particularly focused on school buildings.(Jephcott 1986; Olson 2003) The California Joint Technical Committee on Earthquake Protection responded:

> *Insofar as the police power of the state will permit, it should be required that all privately owned buildings be made earthquake resistant. Strengthening of public buildings, however, is subject to the will of the people and there should be no delay in making these buildings – particularly school buildings – safe. (Alesch and Petak, 1986)*

Responding to the pressure exerted by lobbying groups such as teacher's organizations, women's clubs, and the engineering community, the state legislature passed the Safety of Design and Construction of Public School Buildings Act of 1933. The Field Act, as it became known, created the first state-wide building requirements in California and applied to all public school buildings in the state.[2] Administered by the Office of the State Architect, the Act required plans, specifications and calculations for new buildings to be submitted to the state's Schoolhouse Section. The law was flexible with regard to existing buildings, but it did permit the State Architect to evaluate an existing school building if a certain percentage of the students or parents requested it.(Olson 2003)

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[2] The Field Act's jurisdiction included community college buildings, but excluded universities and private schools.

Since 1933, the Field Act has evolved with changing political pressures and advances in earthquake engineering. The 1939 Garrison Act made the requirements for existing school buildings more stringent by legislating that if a structural engineer found a pre-1933 school building to be unsafe the structure must be updated to the California Building Code.[3] However, the Garrison Act did not mandate the inspection of school structures until amended in 1968. This amendment required that all school buildings not meeting Field Act standards be abandoned by 1975; school-district trustees were to be held liable for potential injuries or casualties if they failed to launch a good-faith effort to conduct repairs. State legislators provided some funding for poorer school-districts and modified laws so that bonds could be used to raise money for the rehabilitation work.(CA Dept. of General Services 2002; Geschwind 2001)

The Field and Garrison Acts are generally seen to have reached their goals of improving the overall safety of school buildings. After the San Fernando Earthquake in 1971, 160 of the 600 schools in the area experienced some damage, with most of the damage occurring in those structures that were already scheduled for strengthening under the Garrison Act. (Murphy 1973) The amended Garrison Act largely met its rehabilitation goals; whereas 1,593 pre-Field Act buildings were in use in 1972, only 19 were still used in 1977. (Geschwind 2001)

Nevertheless, the controversy over public school safety begun by the Field Act has persisted. In the late 1970s, the state began examining nonstructural seismic hazards in its schools. (CA Office of Emergency Services 2003) Further legislation in 1999 instituted plans to evaluate and rehabilitate non-wood frame school buildings that do not meet the requirements of the 1976 UBC. (CA Dept. of General Services 2002) Under this legislation, the Office of the State Architect has examined schools to see which were non-ductile concrete (estimated at approximately 8,000), but is releasing it to individual school districts only if requested. There has also been pressure to eliminate Field Act exemptions for existing private and all charter schools. (CA Office of Emergency Services 2004) The Field Act's implementation, and persistent revisions, indicates the movable target of seismic safety resulting from advancements in engineering and changing political pressures.

Hospital Safety

The 1971 San Fernando Earthquake pushed the seismic safety of hospital structures into the public eye. Several hospitals, including the Sylmar Veteran Administration Hospital and the Olive View Hospital, collapsed in the earthquake. The 1973 Alquist Hospital Safety Act mandated that hospitals have higher seismic safety standards:

> *It is the intent of the Legislature that hospitals, which house patients who have less than the capacity of normal healthy persons to protect themselves, and which must be reasonably capable of providing services to the public after a disaster, shall be designed and constructed to resist, insofar as practical, forces generated by earthquakes, gravity and wind. (Alesch and Petak, 2004)*

The law required the Schoolhouse Section (as established by the Field Act) to review plans for design and construction. The Legislature's intention regarding the technical specifications was left for debate, particularly the requirement that hospitals remain functional following an earthquake, with the qualification, "insofar as practical". (Alesch and Petak 2004; Geschwind 2001)

Like the Field Act, the Hospital Safety Act did not originally apply to existing structures, but pressure mounted to deal with the 90% of California's hospitals that predated the Hospital Safety Act, particularly after the 1994 Northridge Earthquake. Senate Bill 1953, passed in 1994, required that acute care facilities built before 1973 (including some 2700 buildings) be upgraded to certain

[3] UBC with some additional amendments.

nonstructural and structural standard; by 2008, these structures should not pose a significant threat to life; by 2030, hospitals are to be retrofitted to a level capable of providing services to the public after disasters.[4] (Alesch and Petak 2004; CA Office of Emergency Services 2004) Data assembled by the Office of Statewide Health and Planning in 2001 found that 61% of San Francisco's hospitals needed to be retrofitted to meet the first deadline. (Russell 2001) According to a study conducted by Alesch and Petak, most of the hospitals had met the initial requirement of reporting their pre-1973 structures to the State by 2001. Some structures had also been modified from acute-care facilities to other uses so they would not need to meet the requirements. Further compliance is straining the financially vulnerable health-care industry, placing especial burden on public and poorer hospitals. By 2001, 140 hospitals had requested waivers for delayed compliance due to financial difficulties. (Alesch and Petak 2004)

Un-reinforced Masonry Buildings Act

While the Field/Garrison Acts and the Hospital Safety Act identified critical structures based on occupancy type, other retrofit ordinances have been motivated by a particular category of structural hazard. An early example of this type is the 1947 City of Los Angeles Parapet Correction Ordinance which required parapets of buildings facing streets, sidewalks, or exit paths to be removed or braced. (Green 1993; Murphy 1973)

The state of California's legislation of existing un-reinforced masonry (URM) structures grew out of regulations that developed at the municipal level. Long Beach took an initial step toward legislation of un-reinforced masonry in the 1960s when the city's building department condemned unsafe masonry structures as a "public nuisance", requiring their rehabilitation or demolition. This proactive role of the Long Beach building department met significant complaints from owners, but these were effectively quieted after the 1971 San Fernando earthquake damaged 70 – 75% of San Fernando's un-reinforced masonry structures. Long Beach's ordinance for "Rehabilitation of Existing Structures" passed in June 1971. It applied to all un-reinforced masonry structures built before 1934. Repaired buildings are required to meet the standards of the 1970 UBC.(Alesch and Petak 1986; Green 1993) Other California municipalities, including Los Angeles and Santa Ana, gradually followed Long Beach's lead. (Alesch and Petak 1986; Green 1993) Early evaluation of the programs showed a reduction in the number of hazardous buildings in these cities. When Alesch and Petak studied these policies in 1986, Long Beach had achieved 31% compliance in their 15-year program, though 242 of these 288 structures had been demolished to meet the requirements. In Los Angeles, where the structures were subject to less stringent requirements, 458 of the 2097 structures ordered to rehabilitate were in full compliance, only 67 of them through demolition. (Alesch and Petak 1986) However, engineers questioned the usefulness of the less onerous LA ordinance; Henry Degenkolb later commented, "I have a lot of doubts in my mind as to its real effectiveness, if they get a big earthquake. In order to make the ordinance palatable, they watered it way down." (Degenkolb 1994) The Northridge earthquake demonstrated that the life-safety objectives of the Los Angeles URM ordinance had been poorly communicated to owners, as many owners of rehabilitated structures were surprised at the damage their structure sustained.(CA Seismic Safety Commission 1995)

State-wide development of a mitigation proposal for the more than 20,000 un-reinforced masonry structures in California reached fruition in the mid-1980s. The 1986 law required all local governments in Seismic Zone 4 to inventory hazardous un-reinforced masonry buildings and establish a risk reduction program by 1990. The law recommends, but does not require, that local governments adopt mandatory strengthening programs, establish seismic retrofit standards and enact measures to

[4] The Legislature recently delayed compliance with the 2008 requirement until 2013; a further delay until 2020 has also been under consideration.

reduce the number of occupants in URM buildings. As such, the law's implementation depends on the actions of each municipality. Local governments have principally adopted three types of programs: mandatory strengthening (including 52% of municipalities and 64% of the population), voluntary strengthening programs (16% of municipalities) which require owners only to evaluate their structures, but may include financial incentives for upgrading, and notification programs (18% of municipalities), which alert owners of the potential seismic risk of their structure. Of the 26,000 inventoried structures in the state approximately 53% have been retrofitted, another 13% have been demolished. Compliance has been much higher in local jurisdictions with mandatory programs than voluntary programs (53% vs. 19% compliance). As of 2003, about 8000 of the identified URM structures remained to be upgraded.(CA Seismic Safety Commission 2003; Green 1993; Macleod and Scott 1987)

These state-wide regulations demonstrate the key challenges of regulating the safety and upgrading of existing structures. Seismic safety proposals typically meet significant opposition from building owners. In this sense, hospitals, schools and essential public service buildings, many of which are publicly owned and which invoke special public sympathy, represent perhaps the easiest cases. Legislation for non-ductile concrete is likely to be even more controversial. Even when opposition can be overcome, funding to support the programs is extremely difficult. The financial burdens on hospitals and their delayed compliance to the Hospital Safety Act is a case-in-point. Moreover, following implementation, the targeted structures are likely safer, but inconsistently so, especially in the case of municipal implementation of the URM law. The various proposals for vulnerable soft story and reinforced concrete structures under consideration at the local and state level face the same challenges of overcoming opposition, finding funding and insuring effective implementation and have so far lacked the urgency from a recent California earthquake.

ASSESSING COLLAPSE RISK

Archetypical Building

Accurate quantitative assessment of the collapse risk of existing buildings plays an essential role to formulate public policy. To illustrate recent advancements in this area, an assessment is presented based on recent research by the author and colleagues associated with the PEER Center. The structure analyzed here is chosen as an archetypical example of a potentially vulnerable subset of existing reinforced concrete buildings in California. The building's design and layout are representative of buildings constructed between 1950 and the mid 1970s. During this period improved analytic methods led to the construction of more flexible and open concrete structures, reducing redundancy and overstrength in comparison to earlier concrete structures. Despite increasing awareness of ductility in the 1960s, major changes in building codes requiring ductility, joint shear design and strong-column weak beam ratios did not occur until 1976. (Moehle 1998) Potential vulnerabilities in existing building of this type have been identified many times, eg. (CA Seismic Safety Commission 1985) and in San Francisco's recent CAPSS Study (ATC 2003). The archetypical structure's design and detailing typify the key characteristics of this class of structures for the purpose of assessing their seismic collapse risk. A group of similar assessments, each representative of a sub-class of structures, could be very informative in regional loss modeling and, from a policy perspective, systematically identifying the most vulnerable structures.

The chosen archetypical structure is a reinforced concrete moment frame office building located on a non near-fault site in Los Angeles with a 2% in 50 year ground motion corresponding a 1-second spectral acceleration of 0.82g. The 1967 Uniform Building Code calculated required base shear strength according to V = ZKCW where, K = 1 for a moment frame, Z = 1 for Zone 3, and C, the seismic weight coefficient, is calculated to be 0.068. (ICBO 1967) The resulting base shear is approximately 7% less than an identically sized building at the same site designed according to the 2003 International Building Code. The 1967 Map of Seismic Probability shows Zone 3 covering most of California excepting the Central Valley; therefore the same design forces would be calculated at any of these locations. [5] This is contrast to today's Maximum Considered Earthquake design value maps. As a result for another site in California with a different MCE we would find a stronger or weaker 1967 building relative to MCE (and in comparison to modern designs).

Under 1967 code requirements this structure is defined as a "moment-resisting frame", and as such has no specific requirements for ductility.[6] (ICBO 1967) As a result, beam and column design are strength controlled. Joints are not explicitly designed, and there are no capacity design requirements regarding relative strength of beams and columns. These provisions differ significantly from those found in today's building codes for high seismic regions. As designed, the space-frame structure has 13 ft. story heights, column spacing at 30 ft., column sizes ranging from 20 in. x 20 in. to 24 in. x 24 in, and beam depths between 20 and 26 inches. Column ties are spaced at 10 inches on center. The plan and elevation are shown in Figure 1.

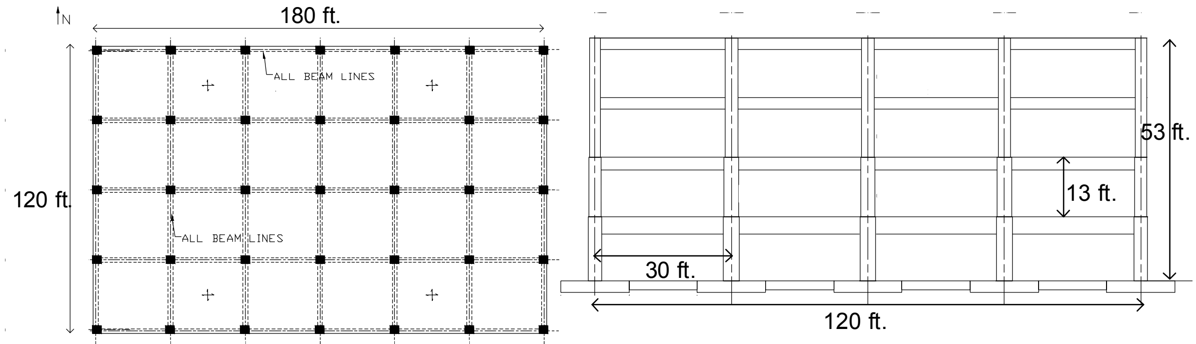

FIGURE 1. Plan and elevation of archetypical four-story office building.

Framework and Modeling

The methodology used to estimate collapse risk follows the PEER performance-based engineering framework that has been presented in detail elsewhere.(Deierlein and Haselton 2005) Following the procedures of incremental dynamic analysis the modeled structure is subjected to scaled ground motions until reaching the intensity measure (in this case $S_{a,T=1sec}$) that causes dynamic instability and sidesway collapse.(Vamvatsikos and Cornell 2002) The simulation model captures the flexural hinging in beams and columns and joint shear failure which may create a sidesway collapse mechanism. Local collapse modes, which are difficult to simulate, are evaluated by combining predicted structural responses (engineering demand parameters) with fragility (or damage) models. (Elwood and Moehle 2005) Incremental dynamic analysis directly incorporates the uncertainty associated with record-to-record variation in earthquake records. Thirty-six ground motion records (each with two horizontal components) were selected based on magnitude, distance, fault type and other factors. (Goulet et al. 2006) The analysis model is created in OpenSees (opensees.berkeley.edu/).

This structural model was created to simulate collapse. For this reason, a lumped plasticity model, which is able to capture the cyclic strength deterioration and the negative post-capping stiffness caused by rebar buckling and fracture, is used for beam-column elements.(Ibarra 2003)

[5] Zone 4, a recognition of the high seismic risk in areas such as California, was added in 1988. (http://quake.wr.usgs.gov/prepare/factsheets/RiskMaps/)

[6] The 1967 UBC does include optional provisions for ductile detailing of concrete which were not followed in this design. If ductility provisions were met a reduction in design strength (K = 0.67) was allowed.

These models have been calibrated with approximately 260 rectangular columns in the PEER Structural Performance Database (http://nisee.berkeley.edu/spd/). Columns and beams are modeled with rotational springs at each end, which combine flexure or flexure-shear behavior and bond-slip in the joints. The parameters used to model the 1967 frame are shown in Figure 2. The values shown are based on the calibration study. The effective stiffness represents the stiffness up to 60% of yield. For collapse modeling, the plastic rotation capacity and the negative post-capping slope are particularly important. The plastic rotation capacity for these elements is approximately one-third what we would use for a highly-detailed 2003 seismic design, the post-capping slope is more negative, and backbone deteriorates more rapidly. The cyclic deterioration parameters capture the expected flexure-shear failure mode in these columns. The joint elements account for the finite joint sizes and incorporate a spring to model joint panel shear.(Altoontash 2004) The calibration of the joint panel shear spring is based on experimental data assembled by Mitra and Lowes (2005). When these component models are assembled, the structure's fundamental period is calculated at T_1=1.3sec.

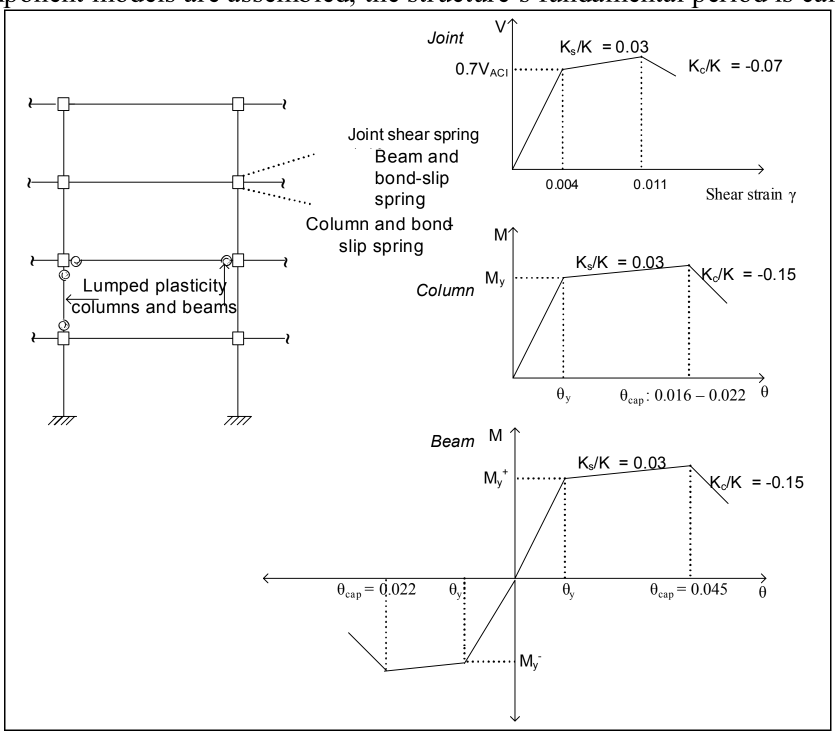

FIGURE 2. Element backbone models used in modeling the 1967 frame

Collapse Risk

The incremental dynamic analysis (IDA) results for this structure and models described are shown in Figure 3. In the IDA process, pairs of earthquake components from an individual earthquake record are considered; provided that the structure is equally strong in the two orthogonal directions the structure is assumed to collapse when the first component in the pair causes collapse, as a proxy for 3-dimensional analysis. The median spectral acceleration at collapse is 0.91g; the structure fails at peak interstory drifts ratios between 0.03 and 0.06 (3% to 6%). From the IDA results the probability of collapse given a spectral acceleration level can be calculated; these results are shown in Figure 3b. The uncertainty associated with the record-to-record variation is $\sigma_{\ln,RTR} = 0.31$. To account for additional uncertainties associated with component models $\sigma_{\ln,modeling} = 0.50$ is assumed based on previous studies, and the curve representing modeling and record-to-record variation is obtained. (Haselton et al. 2006) The mean annual frequency of collapse of 0.0021 is calculated from integration with the site specific hazard curve (which describe the mean annual frequency of exceeding a specified spectral acceleration). The probability of collapse at the 2% in 50 year ground motion (0.82g) is 0.44. Representative collapse modes are included in Figure 4. The dynamic collapse mode depends very much on the earthquake record chosen. For this poorly detailed structure, joint shear failure and column hinging are common collapse modes.

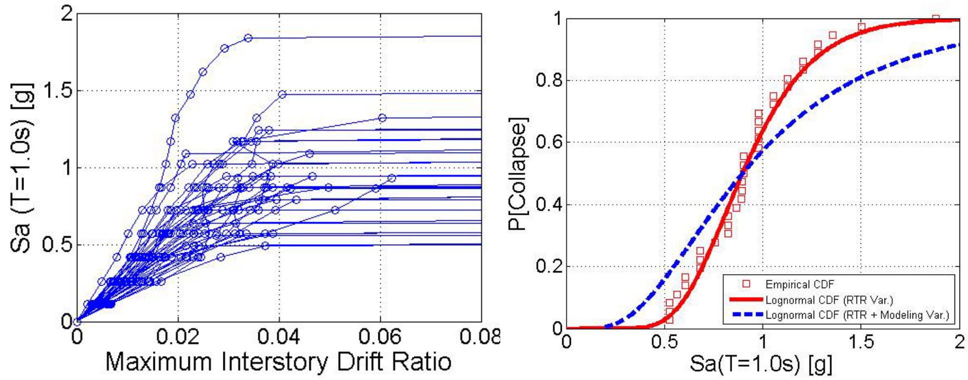

FIGURE 3. (a) Incremental dynamic analysis; (b) Probability of collapse given $S_{a(T=1.0s)}$.

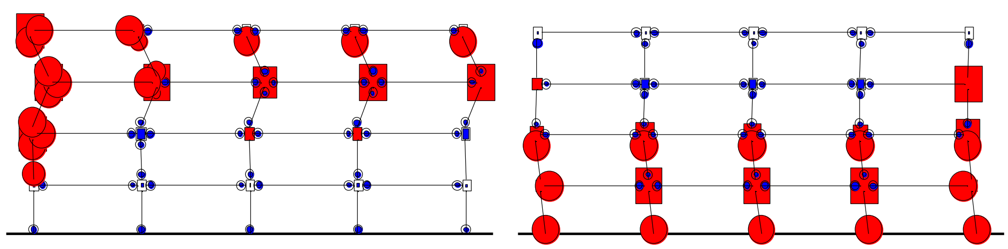

FIGURE 4. Representative collapse modes from dynamic analysis for two different earthquake records.

Column shear failure and subsequent loss of vertical carrying capacity are not simulated in the dynamic analysis model. These failure modes are incorporated through post-processing of analysis results using fragility data for column shear failure (Aslani 2005; Elwood and Moehle 2005). From fragility information these columns may experience shear failure between 1.5 to 3% column drift, while loss of vertical carrying capacity in the column occurs at greater than 4% column drift. These column drift ratios correspond to equivalent building drift ratios of 3 to 5%, considering the additional rigid body column rotations associated with the flexibility of beams and foundations. As shown in Figure 3, sidesway collapse occurs at 3 – 6% interstory drift. Thus, while column shear failure is possible, the structure has likely collapsed in sidesway before the shear-failed column has lost its capacity to carry axial load. The collapse probabilities (including modeling uncertainty) are shown in Figure 5 for three limit states. The first considers sidesway collapse only and is taken directly from incremental dynamic analysis (as shown in Figure 3b). The second represents a limit state where there are shear cracks in at least one column, but the column is still able to withstand gravity loads. The third is shown for illustration purposes only as columns are unlikely to lose axial load carrying capacity before sidesway failure occurs. The mean annual frequency associated with the column shear failure limit state is 0.0048 (a two-fold increase from the sidesway-only case), and the probability of reaching the limit state at the 2% in 50 year ground motion is 0.60. The shear failure limit state does not signify collapse; columns may be marred by diagonal shear cracking, but they are still able to carry vertical loads, likely preventing loss of life. Research is ongoing to directly incorporate shear failure mode into dynamic simulations, eg. (Elwood 2004).

The significance of modeling uncertainty is demonstrated by sensitivity analyses. The IDA results described above are based on the mean model for this structure, using mean properties for each of the element parameters. The sensitivity analysis here investigates the effect of varying the ductility of different types of structural components in the building. For each component, the primary parameters contributing to ductility are rotation capacity, post capping stiffness, hysteretic energy dissipation, and hardening stiffness, and these are highly correlated: increased plastic rotation capacity, a shallower post-capping slope and increased hysteretic energy dissipation, are all associated with more ductile behavior. The mean and coefficient of variation of the basic random variables are estimated from calibration studies, and shown in Table 1. For this structure, changes in column ductility and joint ductility have a larger impact on the median collapse results than beam ductility. The "high" beam ductility model increases the collapse capacity by 20%, whereas the "high" column ductility model leads to a 45% increase. This observation is consistent with the representative collapse modes shown in Figure 4, which show significant joint shear failure and column capping, but less deformation in the beams.

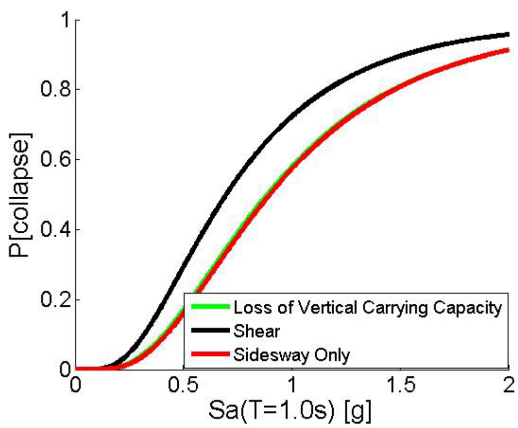

FIGURE 5. Probability of collapse given S_a when non-simulated failure modes are included.

TABLE 1. Distributions of random variables used in sensitivity analysis. (Haselton et al. 2006; Ibarra 2003)

Random Variable	*Mean (μ)*	*C.O.V*
Beam/Column Plastic Rotation Capacity[2]	1.00	0.5
Normalized Hysteretic Energy Capacity	44.3	0.5
Post-Capping Stiffness[2]	1.00	0.6
Element Hardening Stiffness[2]	1.00	0.5

[2]Represented as fraction of mean values

The same method has been used to estimate the collapse risk of modern code-conforming structures. Haselton et al. 2006 conducted a detailed study of a 4-story special moment frame designed according to the 2003 International Building Code. This structure is designed for the same site in Los Angeles, but illustrates the significant changes in the building code requirements over the last 40 years. The 2003 special frame has significantly larger member sizes (beams: 32 to 40 inches deep, columns 24 in. x 28 in. to 30 in. x 40 in.). Strength and stiffness controlled beam design, column size was governed by joint strength requirements, and column strength by the strong column-weak beam ratio. Collapse assessments for the 2003 design and the 1967 moment frame are compared in Table 2. This table provides three explicit and related metrics for quantification of structural safety: mean annual frequency of collapse, probability of collapse at the 2% in 50 year ground motion, and the margin of safety between the median collapse capacity and the design level. By all measures, the 1967 structure shows significantly worse performance and a lower level of safety than the 2003 structure. These differences quantitatively demonstrate nearly four decades of building code improvements, and the variability possible in collapse assessments of different types of old and new structures. With further research, these values may also be linked to potential fatalities.

TABLE 2. Comparison of two structures' collapse risk.

	1967 space frame[7]	*2003 perimeter frame*
Mean Annual Frequency Collapse	21×10^{-4}	1×10^{-4}
P[Collapse|2% 50 yr.]	44%	3%
Margin: $S_{a,median}/S_{a,2\% \text{ in } 50 \text{ yr.}}$	1.1	2.7

[7] Considering sidesway collapse limit state.

USING COLLAPSE RISK TO INFORM DECISION-MAKING

In recent years earthquake engineering researchers such as those at the PEER center have developed simulation tools and analytical models that can be used to quantify the collapse risk of existing reinforced concrete structures. This assessment of collapse risk provides a group of measures (like those shown in Table 2) which may be used to compare the safety of a given structure or group of structures to other structures, and to other risks we face as a society. These types of comparative measures can help to sharpen the efficiency and efficacy of seismic safety legislation by providing a mechanism for standardization and prioritization.

A preliminary comparison shows the risks associated with collapse of existing reinforced concrete structures under seismic loading to be significantly higher than other structure-related risks. Ellingwood and Tekie (1999) reported the mean annual probability of failure under dead and live loads to be approximately 7×10^{-4}, where "failure" is defined as the strength limit state associated with exceeding the nominal plastic yield strength of a structural member.[8] Similarly, Ellingwood and Corotis (1991) find a 10^{-6} mean annual frequency of a structurally significant (flashover) fire in modern office buildings.[9] When compared to Table 2, these statistics demonstrate the relative dominance of earthquake collapse risks over other considerations for office structures in high seismic areas such as California.

This computational assessment of collapse risk cannot, however, tell us how safe is safe enough. For an existing office building such as the one described here, is a mean annual frequency of collapse of 0.0021 acceptable? Are we satisfied that existing buildings may be more than 10 times more likely than new buildings to collapse? In other industries acceptability has been judged through a variety of different approaches, including market mechanisms, the institution of professional standards, cost-benefit analysis, decision analysis, or participatory processes engaging the public. One possible framework for evaluating earthquake risks in a societal context, used by governments in Hong Kong and the Netherlands among others, involves the creation of F-N diagrams which relate probability of at least N fatalities with the number of fatalities. Every region on the diagram is defined as "acceptable", "reduction desired" or "unacceptable" risks, eg. a very high probability of fatalities or a very high number of fatalities (even with a small probability) fall in the unacceptable region. (Christian 2004)

In the words of political scientist Peter May, performance based earthquake engineering, and its application to assessment of seismic collapse risk, lends itself to "discussion of desired safety goals, the costs involved of achieving these, and the trade-offs imposed."(May 2001) Earthquake engineering researchers can contribute to this discussion by further validating the accuracy of assessed failure probabilities, such as those presented here. This validation may involve reaching a consensus on some of the underlying issues such as ground motion scaling and selection, conducting further component testing at deformations large enough to capture key characteristics for collapse modeling, and further improving simulation capabilities to directly incorporate vertical collapse modes. Structural engineering professionals' judgment is an important and needed confirmation of these methods and their overall results. The discussion itself falls within public/policy arena, and government officials and the policy community are the best-suited facilitators. Public engagement is needed to gage desired safety, as well as perception of seismic risks in relation to other societal risks face. This discussion is an essential component of a systematic examination of why (or not) government intervention in retrofit is needed – before the next major earthquake.

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[8] This limit state is not synonymous with collapse, so we would expect an even smaller likelihood of collapse under gravity loading.

[9] Data based on office buildings; frequency and losses are higher in single-family residences.

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