Comparing Seismic Collapse Safety of Modern and Existing Reinforced Concrete Frame Structures in California

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INTRODUCTION

In this study, the framework of performance-based earthquake engineering is used to predict the seismic collapse risk of California's older (non-ductile) and modern (ductile) reinforced concrete frame structures. Collapse performance assessments are conducted for two sets of structures, those designed according to an out-dated building code, the 1967 Uniform Building Code (UBC), and those designed according to modern building code provisions, the 2003 International Building Code (IBC). Each set includes 2, 4, 8 and 12 story reinforced concrete frame buildings, designed as space and perimeter frame systems. These preliminary results provide measures of collapse safety of reinforced concrete frame structures and are used to evaluate the level of safety provided by modern building codes, and to quantify differences in safety between older and moden structures, answering questions such as "how safe are code-conforming reinforced concrete frame structures?" and "how much more likely are existing reinforced concrete frames to collapse in an earthquake?"

REPRESENTATIVE REINFORCED CONCRETE FRAME STRUCTURES

This study builds on previous studies [Liel et al. 2006; Goulet et al. 2007] that described collapse assessments of *individual buildings*, and analyzes suites of structures for the purpose of evaluating the performance of two *classes* of structures: (1) existing, non-ductile reinforced concrete moment frames, and (2) modern, code-conforming reinforced concrete moment frames. These suites of structures include a number of individual buildings, which are selected to be representative of the range of design and behavior for the structural systems of interest.

The non-ductile reinforced concrete frame structures are of four different heights, 2, 4, 8 and 12 stories, and different framing systems, including both space and perimeter frames. The buildings are designed for office occupancies with an 8-inch flat slab floor system, and 25 foot bay spacing. Each of these structures is fully designed according to the requirements of the 1967 UBC [ICBO 1967], such that the structures are representative of California seismic design between approximately 1950 and 1975. The structures are designed for the highest seismic zone at the time, zone 3, which included most of California. The designs meet all requirements present in the 1967 UBC, including maximum and minimum reinforcement ratios, maximum stirrup spacing, and requirements on hooks, bar spacing and anchorage, etc. All structures are designed with the standard level of detailing. Reflecting conventional practice, the interstory drifts are limited to 2% under design lateral forces.

The modern reinforced concrete frame structures are designed according to the provisions of the 2003 IBC [ICC 2003], ASCE 7-02 [ASCE 2002], and ACI 318-02 [ACI 2002], and meet all governing code requirements for strength, stiffness, capacity design, and detailing. They include both space and perimeter frames with 2, 4, 8 and 12 stories, and 20 foot column spacing. These structures benefit from the additional provisions governing design and detailing of these structures, which have been incorporated into seismic design codes since the 1970s, including an assortment of capacity design provisions (eg. strong column-weak beam ratios, element and joint shear capacity design), and detailing improvements (eg. increased lap splice requirements, 135° hooks on transverse stirrups). The 2003 reinforced concrete frames are designed for a far-field site in the Los Angeles area, with maximum considered earthquake S_{M1} = 0.90g [Goulet et al. 2007]. To ensure that the designs are representative of current design practice, each of the designs was reviewed by a practicing engineer [Hooper 2006].

COLLAPSE ASSESSMENT PROCEDURE AND NONLINEAR MODELING

The procedure for collapse assessment utilizes the methodology for performance-based earthquake engineering methodology developed by the Pacific Earthquake Engineering Research Center, which provides a probabilistic framework for relating ground motion intensity to structural response and building performance through nonlinear time-history simulation [Deierlein 2004]. Assessment of global sidesway collapse capacity uses the Incremental Dynamic Analysis (IDA) technique [Vamvatsikos and Cornell 2002]. In IDA, the nonlinear structural model is subjected to a recorded ground motion, and dynamically analyzed to predict the structure's response. This time-history analysis is repeated, each time increasing the scale factor on the input ground motion, until that record causes structural collapse, as identified by interstory drifts that increase without bounds (i.e. dynamic instability). This process is repeated for an entire set of ground motion records. For this study, 44 ground motions were selected to represent large earthquakes with moderate fault-rupture distances (i.e., not near-fault conditions) [ATC 2007]. The outcome of the IDA procedure is a collapse fragility function, a cumulative probability distribution that defines the probability of structural collapse as a function of the ground motion intensity. Ground motion intensity is defined by the spectral acceleration at the first mode period of the building [$Sa(T_1)$]. The collapse assessment procedure also accounts for the contribution of non-simulated failure modes [Liel et al. 2006], structural modeling uncertainties [Liel et al. 2007], approximate consideration of three-dimensional effects [Haselton and Deierlein 2007] and adjustment for proper spectral shape [Baker and Cornell 2005; Haselton and Deierlein 2007]. In addition, pushover analysis is conducted to determine the level of static overstrength in the design.

Example results from static pushover analysis and incremental dynamic analysis are shown in Figures 1 and 2, respectively. The static pushover analyses illustrate the much smaller ductility of reinforced concrete structures designed without ductile detailing. In addition, it is observed that the static overstrength of the 2003 special moment frames is typically larger, due to overstrength (Ω) obtained from strong column-weak beam ratios and the required ratios of positive to negative beam flexural strength. (Note: It is typically observed that the 2003 and 1967 structures differ in terms of both yield and ultimate strength. That the yield strength of the structures in Figure 1 is the same is an anomaly.) The incremental dynamic results in Figure 2a are for a 4-story modern (code-conforming) reinforced concrete moment frame. Each line represents the analysis results from one ground motion scaled until the structure collapses; the results of one ground motion are bolded. Collapse occurs when the IDA curve is horizontal, such that further increase in the ground motion intensity causes interstory drifts to increase without bounds. On average, considering all the ground motions, this structure collapses at

Sa(1s) = 2.2g. These results are plotted as a collapse fragility function in Figure 2b. This probability distribution incorporates both the variability in ground motions (termed "record-to-record" variability) and in modeling (see [Liel et al. 2007]).

From the collapse fragility, several metrics of collapse performance can be obtained: the median collapse capacity, the collapse margin (the ratio of the median collapse capacity to the 2% in 50 year ground motion hazard level of $Sa_{2/50}$), the collapse probability (probability of collapse given an extreme ground motion, usually taken as the 2% in 50 year ground motion), and the mean annual frequency of collapse or collapse rate, obtained by integrating the structure's collapse fragility with the site specific hazard curve. The buildings in this study are assumed to be located at a specified far-field site in Los Angeles, for which the hazard curve has been computed through probabilistic seismic hazard analysis [Goulet et al. 2007]. These metrics are illustrated graphically in Figure 2b.

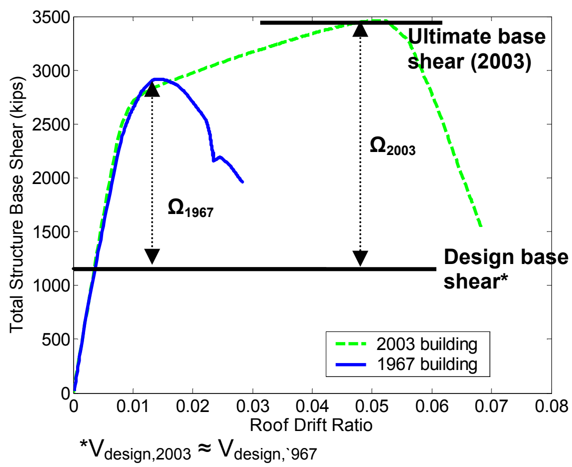

FIGURE 1 – STATIC PUSHOVER ANALYSIS FOR A 4-STORY REINFORCED CONCRETE MOMENT FRAME DESIGNED ACCORIDNG TO OLDER (1967 UBC) AND MODERN (2003 IBC) CODE PROVISIONS

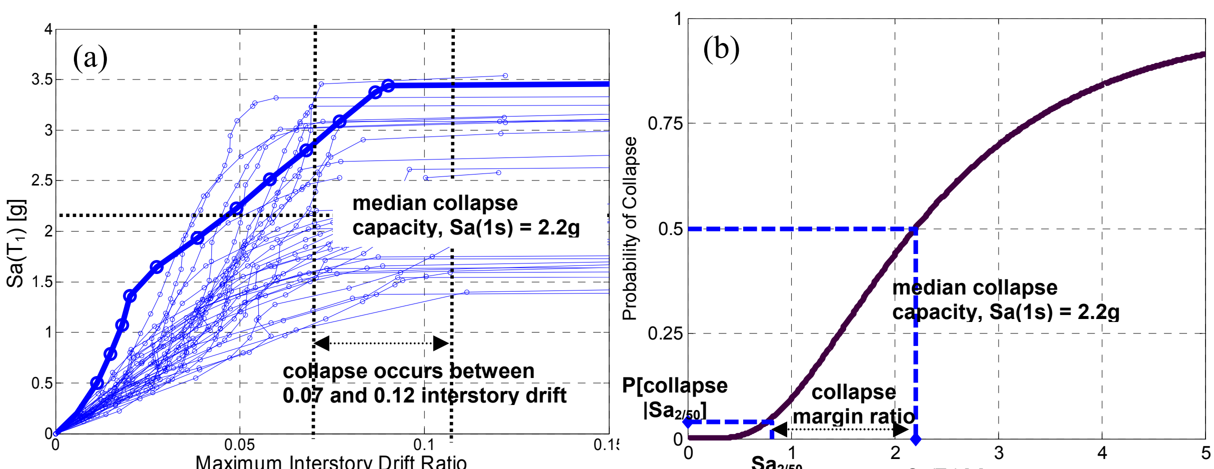

FIGURE 2 – EXAMPLE INCREMENTAL DYNAMIC RESULTS FOR A MODERN RC FRAME STRUCTURE, INCLUDING (A) IDA OUTPUT AND (B) COLLAPSE FRAGILITY FUNCTION

Nonlinear analysis models for the reinforced concrete moment frame structures consist of the two-dimensional three-bay frame, as shown in Figure 3a. Modeled using OpenSees [PEER 2006], the simulation model captures material nonlinearities in beams, columns, and beam-to-column joints, as well as large deformation (P-Δ) effects. The model includes a leaning column to account for additional seismic mass on the gravity system, but does not account for the contribution of the gravity system to the lateral resistance of the frame. Inelasticity in the beams, columns, and joints are modeled with concentrated springs idealized by the tri-linear backbone response curve shown in Figure 3b and the associated hysteretic rules developed by Ibarra et al. [2005]. An important attribute of this model is the negative stiffness associated with the post-peak response, which enables modeling of strain softening behavior associated with phenomena such as concrete crushing, and rebar buckling and fracture. Properties of the inelastic springs representing beam and column elements are obtained from calibration to experimental tests of over 250 beam-columns, as described by Haselton [2007]. Modeling of joint elements is based on data assembled by Mitra and Lowes (see Liel [2008]). All element model properties are calibrated to mean or expected values of the structural components. When used in combination with nonlinear geometric transformations and robust convergence algorithms, these structural models are capable of simulating structural response into the collapse limit state. Reflecting the expected flexure and flexure-shear failure of these models, shear failure in columns (and the axial collapse of the column that may follow) is not explicitly included in the analysis models, and is incorporated for the 1967 frames through post-processing using component fragility functions [Elwood 2004; Aslani 2005].

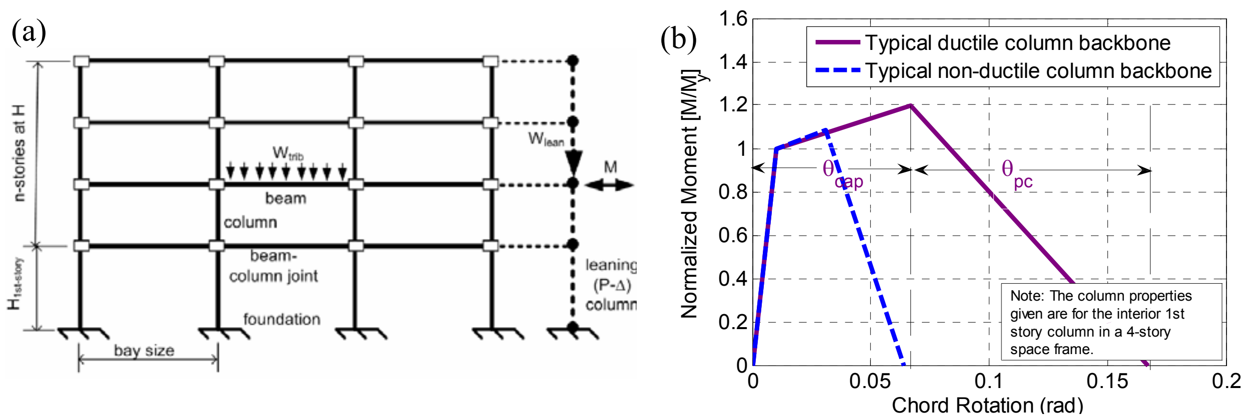

FIGURE 3 – (A) NONLINEAR ANALYSIS MODEL FOR REINFORCED CONCRETE FRAME STRUCTURES, (B) NONLINEAR MATERIAL FEATURES OF BEAM-COLUMN HINGES

COLLAPSE SAFETY OF CALIFORNIA'S REINFORCED CONCRETE FRAME STRUCTURES

Each of the archetypical reinforced concrete frame structures shown in Table 1 was modeled and analyzed according to the collapse assessment procedure described above. A complete set of results for the 2003 reinforced concrete frame structures is available in [Haselton and Deierlein 2007; Haselton et al. 2007]. Collapse assessment of the 1967 reinforced concrete frame structures is documented in Liel [2008].

The results of static pushover analysis provide the first evidence of the superior seismic performance associated with the improvement in seismic design and detailing rules between the 1967 Uniform Building Code and the 2003 International Building Code. On average, the static overstrength (Ω) is 30% higher in the 2003 designs than the 1967 designs. This higher overstrength is due to the strong column-weak beam requirements, required ratios of positive and negative bending strength of beams, as well as specifications for joint capacity design, which are present in the 2003 IBC and tend to increase the lateral strength of the structure. The much higher ductility of the 2003 designs is also apparent from the pushover analyses. When ductility is measured in terms of ultimate roof drift ratio (RDR_{ult}), which aggregates member and system level ductility effects, the modern reinforced concrete structures have approximately three times the deformation capacity of the existing 1967-era designs. Some of this increase comes from improved member level deformation capacity in terms of plastic rotation capacity and post-capping rotation capacity associated with improved detailing requirements in the modern structures; system level ductility is improved by strong column-weak beam requirements. The 1967 structures also tend to be more flexible, and are more sensitive to P-Δ effects.

The collapse metrics evaluated for the representative reinforced concrete frame structures are shown in Figure 4. These collapse performance metrics are appropriately adjusted for spectral shape of rare ground motions, and use first-order-second moment methods to incorporate modeling uncertainty (see [Haselton and Deierlein 2007]). The collapse assessments for the 1967 reinforced concrete frames account for non-simulated failure modes, such that the collapse fragility includes both simulated (sidesway) collapse and collapse due to loss of vertical carrying capacity in columns associated with column shear failure. Due to modern capacity design provisions, the 2003 structures are not expected to experience column shear failure, so the post-processing for non-simulated failure modes is unneeded.

On average, the collapse margin of the new structures is approximately three times higher than the collapse margin of the existing reinforced concrete structures, and this trend is consistent for both perimeter and space frame structures. The modern reinforced concrete frames have collapse margins ranging from about 2 to 3, signifying that the structures are expected to withstand a ground motion with at least twice the intensity of the 2% in 50 year ground motion ($Sa_{2/50}$), on average. In contrast, the 1967 frames, with collapse margins between 0.5 and 0.9, are expected to collapse before the intensity of $Sa_{2/50}$ is reached, on average. If a rare ground motion occurs, such as the one that has a 2% likelihood of occuring every 50 years, the 1967 era frames have high likelihood of collapse, while the 2003 have only a small probability of collapse. These computed probabilities of collapse are approximately eight times higher for the existing reinforced concrete frame structures. In California, the 2% in 50 year ground motion is typically a *rare ground motion* (2475 year return period) that results from a more *frequent event* (perhaps a 150 to 500 year return period), so these data do not imply that most of 1967-era structures would collapse in a significant earthquake, but that we would expect significant damage in areas of low-moderate shaking and collapses in those areas with the highest level of ground shaking. An even greater difference in performance is observed when the collapse fragility is integrated with the site hazard curve to obtain the mean annual frequency of collapse ($\lambda_{collapse}$). The mean annual frequency of collapse can be interpreted as a collapse rate, or the number of collapses per year, which is also related to the collapse return period. For the Los Angeles site of interest, in terms of collapse rate, the 1967 perimeter frames are 20 times more likely to collapse than the 2003 perimeter frames, and the 1967 space frames are 60 times more likely to collapse.

The metrics in Figure 2 also illustrate the effects that building height and framing system have on structural collapse. It is apparent that space frames typically have better collapse performance than perimeter frames; space frames have higher lateral overstrength because of the relative dominance of gravity loading in the design. Perimeter frames, which are more flexible and have higher tributary seismic mass, undergo more rapid strength and stiffness degredation due to P-Δ effects, causing deformations to concentrate in a smaller number of stories. Also, the absence of the gravity system in the analysis model may be a significant source of conservatism for the perimeter frame models. The 1967 space frame structures exhibit relatively constant collapse margins as a function of building height, while the 2003 space frame structures have a negative trend due to the increased importance of design lateral loading (relative to gravity loading) as height increases. However, the low-rise 1967 space frames are particularly vulnerable to column shear failure, and loss of gravity load carrying capacity in columns, accounting for the worse performance of the 2 and 4-story non-ductile reinforced concrete space frames.

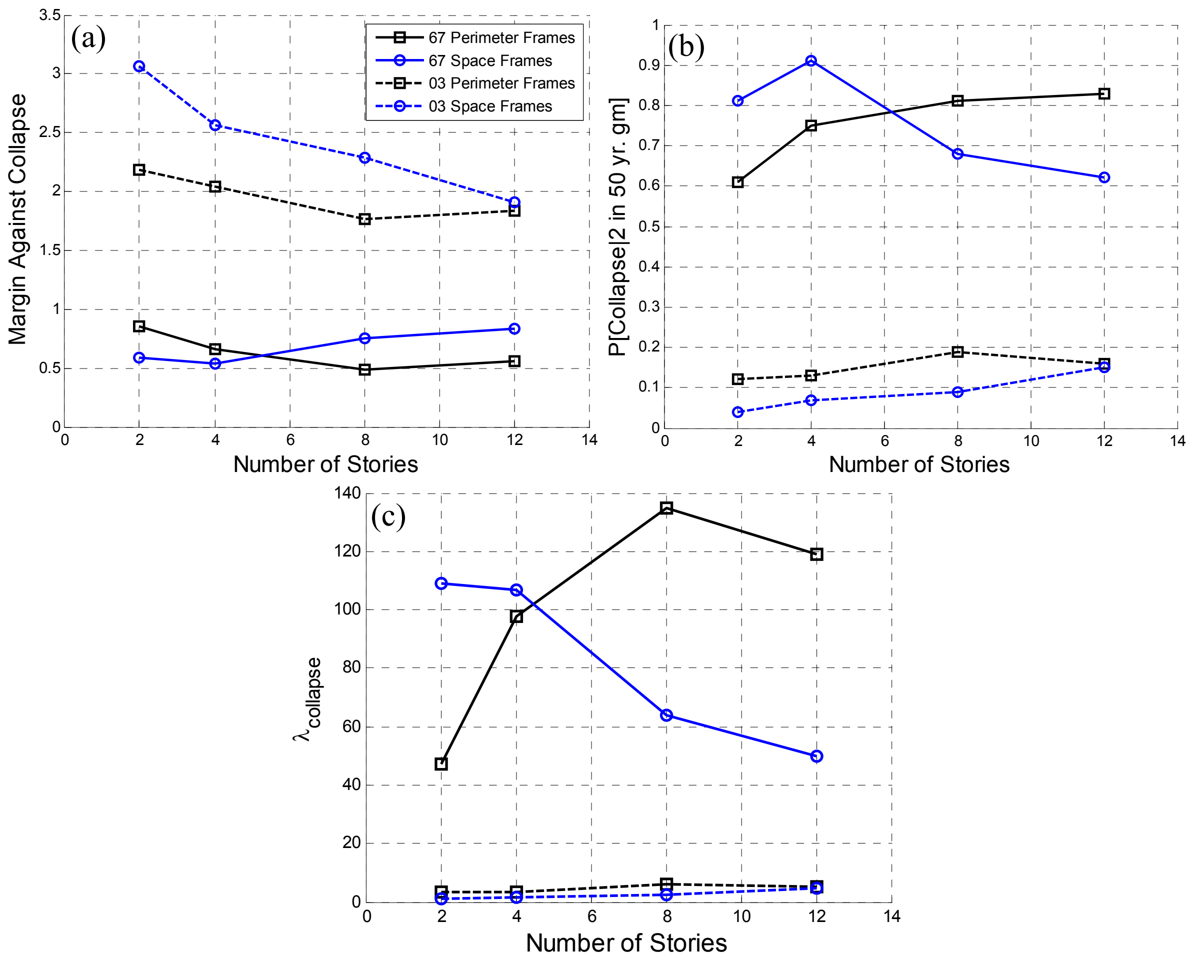

FIGURE 4 – COMPARISON OF COLLAPSE METRICS FOR EXISTING AND CODE-CONFORMING RC FRAME STRUCTURES IN TERMS OF (A) MARGIN AGAINST COLLAPSE, (B) PROBABILITY OF COLLAPSE, CONDITIONED ON THE 2% IN 50 YEAR GROUND MOTION, AND (C) THE MEAN ANNUAL FREQUENCY OF COLLAPSE

As expected, the ductility of the code-conforming reinforced concrete frame structures is much higher than the older structures, with roof drift ratios just before collapse that are 2.4 times larger, and associated interstory drift ratios that are 1.8 times larger. These differences seem to result largely from increases in member-level deformation capacity associated with improved detailing requirements. The plastic rotation capacities of code-conforming beams and columns are typically three times larger than the beams and columns with non-ductile detailing, and there are similar improvements in post-capping rotation capacity and the cyclic deterioration parameters. These results also suggest that the strong column-weak beam ratio in the 2003 design helps to spread damage among a larger number of stories, improving system level ductility. While modern seismic provisions delay the formation of story mechanisms by slowing the concentration of deformations in less ductile columns, they are insufficient to distribute damage over the entire structure, even for shorter buildings.

CONCLUSIONS

The 2003 reinforced concrete frame structures demonstrate markedly superior seismic collapse performance for all heights and framing systems when compared to the 1967 reinforced concrete frame structures. The modern structures collapse at higher levels of ground motion intensity, and are capable of undergoing more significant deformations before collapse. Both better detailing in individual members – closer tie spacing, use of closed hooks, transverse ties in joints – and system-level design requirements – strong column-weak beam and other capacity design provisions – have contributed to these improvements. The assessed collapse safety of modern reinforced concrete frames is an indicator of the level seismic safety provided by today's building codes, and provides a yardstick to which other structural systems, new and existing, can be compared. The collapse performance assessment of existing structures confirms the expectation that non-ductile reinforced concrete structures are potentially vulnerable and, for the first time, systematically quantifies differences in safety for new and existing reinforced concrete structures.

These metrics for seismic collapse risk can be used to inform the debate about acceptable collapse risk and public safety in California. It is undisputed that structures should be safe enough in future earthquakes to protect public welfare, but this goal is poorly defined. According to Random House Dictionary, safety is "the freedom from the occurrence or risk of injury, danger, or loss." In a more limited sense, safety is taken here to refer to prevention of loss of life. Since absolute seismic safety is unrealistic, we must ask ourselves, "how safe is safe enough?" for our building structures. Seismic provisions in building codes represent the accumulated judgment of the structural engineering community and, when the collapse safety of these provisions is explicitly examined, as in this study, provide one measure of acceptable collapse safety. Due to the shorter remaining life span of existing buildings and the high cost of retrofit or replacement, some reduced collapse safety is probably acceptable for non-ductile reinforced concrete frames. Work is ongoing to finalize the results of this study. In addition, the consequences of the lower collapse safety of existing reinforced concrete moment frames, in terms of fatalities, economic losses and downtime, are topics for future research.

The collapse assessment procedure described here provides a mechanism through which the relative safety of existing and new reinforced concrete frame structures can be systematically examined. A similar codified methodology is being developed by the Applied Technology Council in the ATC-63 project with the goal reliably quantifying building system performance and response parameters for use in seismic design provisions [ATC 2007].

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