Seismic Collapse Safety of Reinforced Concrete Buildings:

I. Assessment of Ductile Moment Frames

Curt B. Haselton[1], Abbie B. Liel[2], Gregory G. Deierlein[3], Brian S. Dean[4], Jason H. Chou[5]

ABSTRACT

This study applies nonlinear dynamic analyses to assess the risk of collapse of reinforced concrete (RC) special moment frame (SMF) buildings, with the goal of quantifying the seismic safety implied by modern building codes. Thirty archetypical RC SMF buildings, ranging in height from 1 to 20 stories, are designed according to ASCE 7-02 and ACI 318-05 for a high seismic region. The results of performance-based seismic assessments find that, on average, these buildings have an 11% probability of collapse under ground motion intensities with a probability of exceedance of 2% in 50 years. The average mean annual rate of collapse of 3.1×10^{-4} collapses/year corresponds to an average 1.5% probability of collapse in 50 years.

The study further examines the influence of specific design provisions on collapse safety. In particular, changes to the minimum seismic base shear requirement between ASCE 7 2002 and 2005 editions and variations in ACI 318 strong-column weak-beam

[1]Assistant Professor, Dept. of Civil Engineering, California State University Chico, CA 95929, USA Email: chaselton@csuchico.edu

[2] Assistant Professor, Dept. of Civil, Environmental, and Architectural Engineering., University of Colorado, Boulder, CO 80309, USA

[3] Professor, Dept. of Civil and Environmental Engineering, Stanford University, Stanford, CA 94305, USA

[4] Engineer, Walter P. Moore and Associates Inc., San Francisco, CA 94105, USA

[5] Engineer, Quincy Engineering Inc., Sacramento, CA 95827, USA

(SCWB) design requirements are investigated. The study finds that the reduction in the minimum base shear, introduced in ASCE 7-05 and subsequently rescinded, dramatically increases the collapse risk of tall (long-period) frame buildings in high seismic regions. Investigation of the SCWB requirements shows that the current ACI 318 provisions delay, but do not prevent, column yielding and the formation of story collapse mechanisms. An increase in the SCWB ratio above 6/5 (1.2) does not significantly improve performance of low-rise frame buildings, but may reduce collapse risk for mid-rise and taller buildings. This study of modern RC buildings is contrasted with the collapse safety of older (non-ductile) RC moment frame buildings in the companion paper.

CE Database Subject Headings: collapse, earthquake engineering, seismic effects, reinforced concrete structures, structural reliability.

INTRODUCTION AND OVERVIEW

The primary goal of building code seismic design requirements is to protect the life safety of building occupants during large earthquakes. Meeting this objective requires that the risk of structural collapse is acceptably low. While straightforward in concept, the collapse safety provided by current building codes and standards is unclear, due to the empirical nature of the design provisions and their development. Advancements in nonlinear dynamic analysis, seismic hazard analysis, and performance-based earthquake engineering are enabling more scientific assessment of structural collapse risk and how it is affected by building code design requirements.

This study assesses the collapse safety of modern reinforced concrete (RC) special moment frame (SMF) buildings designed according to the governing provisions of

current American standards, including ASCE 7 (ASCE 2002, ASCE 2005) and ACI 318 (ACI 2002). The collapse assessments are made for a representative set of 30 archetype buildings that employ RC SMF seismic resisting systems. These assessments explore the collapse behavior of this class of buildings and the effects of design decisions on seismic performance. RC SMF structural systems are the focus of this effort because these buildings are generally perceived to provide acceptable seismic safety and the analytical tools for modeling severe deterioration of RC frames are mature enough to permit a relatively accurate assessment.

In addition, this study illustrates the application of performance-based assessment tools to quantify the impact of specific design requirements on seismic collapse resistance. The influence of both the minimum seismic base shear requirement of ASCE 7-05 and the strong-column weak-beam criteria of ACI 318 are considered.

Assessment of seismic collapse safety for RC SMFs applies tools and methods developed by the authors and others at the Pacific Earthquake Engineering Research (PEER) Center. The collapse assessment method builds on prior research to characterize ground motion hazards, develop and validate structural models with degrading strength and stiffness, perform nonlinear dynamic time history analyses, and incorporate uncertainties in ground motion characteristics and nonlinear response. The results presented herein are part of a larger study by Haselton and Deierlein (2007) and expand upon an earlier evaluation of a single four-story RC SMF building which involved both collapse assessment as well as assessment of damage and monetary losses (Goulet et al. 2007, Haselton et al. 2008a). The collapse assessment methods presented here have been incorporated into the FEMA P-695 (ATC-63) Methodology, a procedure for

systematically assessing seismic collapse safety for the purpose of establishing design parameters and provisions for new structural systems (FEMA 2009).

REPRESENTATIVE SET OF STRUCTURAL DESIGNS

This assessment employs a set of archetypical structural building systems that are representative of engineering design and practice for RC SMFs in high seismic regions. As summarized in Table 1, the 30 building archetypes encompass key structural design parameters including building heights from one to twenty stories, space and perimeter frame systems, and bay widths of 20 and 30 feet (6.1 and 9.1 meters). Archetype structures also vary in terms of the strength and stiffness distribution over the building height and foundation fixity assumed in design.

Each archetype is designed according to the provisions of the International Building Code (ICC 2003), ASCE 7-02 (ASCE 2002), and ACI 318 (ACI 2002), including requirements for strength, stiffness, capacity design and detailing. The buildings have office occupancies with an 8-inch flat slab floor system. The 1, 2 and 4-story buildings have a plan area of 120 ft. by 180 ft. (36.6 m. by 54.98 m.); the 8, 12 and 20-story buildings have a square plan measuring 120 ft. by 120 ft. (36.6 m. by 36.6 m.). Seismic design is based on the mapped hazard for a Los Angeles site with $S_S = 1.5g$ and $S_1 = 0.6g$ and soil site class D. The designs were reviewed by a practicing engineer (Hooper 2006) to ensure they conform to typical design practice. Further design documentation is available in Haselton and Deierlein (2007) and FEMA P-695 (2009).

SITE, SEISMIC HAZARD, AND GROUND MOTION CHARACTERIZATION

Owing to the choice of design parameters and seismic hazard level, the archetype buildings are representative of high seismic regions of California. The designs are for a site located in northern Los Angeles, which falls in the transition region of ASCE 7-02

design maps. Site-specific probabilistic seismic hazard analysis is documented by Goulet et al. (2007).

Ground motions used for the nonlinear dynamic analyses are recordings from large magnitude earthquakes (magnitude 6.5 to 7.6) recorded at moderate fault rupture distances (10 to 45 km). The 39 ground motion record pairs (each with two orthogonal horizontal components) and their selection criteria are documented in Haselton and Deierlein (2007). This ground motion set is an expanded version of the far-field ground motion set utilized in the FEMA P-695 (FEMA 2009).

Ground motion records are selected and scaled without considering the distinctive spectral shape of rare (extreme) ground motions, due to difficulties in selecting and scaling a different set of records for a large set of buildings having a wide range of first-mode periods. To account for the important impact of spectral shape on collapse assessment, shown by Baker and Cornell (2006), the collapse predictions made using the general set of ground motions are modified using a method proposed by Haselton et al. (2009). The expected spectral shape of rare (large) California ground motions is accounted for through a statistical parameter referred to as epsilon (ε), which is a measure of the difference between the spectral acceleration of a recorded ground motion and the median value predicted by ground motion prediction equation. A target value of $\varepsilon=1.5$ is used to approximately represent the expected spectral shape of severe ground motions that can lead to collapse of code-conforming buildings (Appendix B of FEMA P-695 2009; Haselton et al. 2010).

STRUCTURAL ANALYSIS MODEL AND COLLAPSE ASSESSMENT METHODOLOGY

A two-dimensional three-bay nonlinear analysis frame model is created for each archetype RC SMF using the OpenSees structural analysis platform (OpenSees 2009), as illustrated in Figure 1. Three bays are assumed to be the minimum number necessary to reflect the differences between interior and exterior columns and joints, and their impact on frame behavior. Strength and stiffness of the gravity system are not represented in the model, but the destabilizing P-Δ effects are accounted for by applying gravity loads on a leaning column in the analysis model. Previous research by the authors has shown that neglecting the strength and stiffness of the gravity system in RC SMF systems is slightly conservative, underestimating the median collapse capacity by approximately 10% (Haselton et al. 2008a). It is also assumed that the damage to the slab-column connections of the gravity system will not result in a vertical collapse of the slab; test data for slab-column connections with modern detailing are still needed to verify this assumption. The foundation rotation stiffness is calculated from typical grade beam design and soil stiffness properties. Rayleigh damping corresponding to 5% of critical damping in the first and third modes is applied.

Element modeling consists of lumped plasticity beam-column elements and finite joint shear panel springs. Lumped plasticity elements were used in lieu of fiber-type element models, since only the former are able to capture the strain softening associated with rebar buckling and spalling phenomena that are critical for simulating structural collapse in RC frame structures. The beam-columns are modeled using a nonlinear hinge model with degrading strength and stiffness, developed by Ibarra et al. (2005). As illustrated in Figure 2, the Ibarra et al. model captures the important modes of monotonic

and cyclic deterioration that precipitate sidesway collapse. Key parameters of the model include the plastic rotation capacity, $\theta_{cap,pl}$, the post-capping rotation capacity, θ_{pc}, the ratio of maximum to yield moment, M_c / M_y, and an energy-based degradation parameter, λ. Based on calibration to test data for RC columns and beams with ductile detailing and low to moderate axial load, the typical mode parameter values are $\theta_{cap,pl}$ between 0.035 to 0.085 radians, depending on the level of axial load in the beam-column, θ_{pc} equal to 0.10 radians, M_c / M_y between 1.17 and 1.21, and λ between 85 and 130 (Haselton et al. 2007, 2008b). The post-capping deformation capacity, θ_{pc}, of 0.10 is a conservative value used due to lack of data; this value would likely be much larger if additional data were available with specimens tested to larger levels of deformation.

The collapse capacities of the archetype building designs are evaluated using a performance-based methodology, key features of which are briefly summarized as follows:

1. Select ground motions for nonlinear dynamic analysis. In this study, 39 pairs of far-field ground motions are used. Issues related to record selection and scaling have been discussed previously.

2. Utilize incremental dynamic analysis (IDA) to organize nonlinear dynamic collapse analyses of the archetype models subjected to the recorded ground motions (Vamvatsikos and Cornell 2002). Using the IDA approach, each horizontal component of ground motion is individually applied to the two-dimensional frame model.

   In this study, ground motion records are amplitude scaled according to the spectral acceleration at the first mode period, $Sa(T_1)$. The ground motions are increasingly scaled until collapse occurs. In this paper, collapse is defined as the

point of dynamic instability, where the lateral story drifts of the building increase without bounds (often referred to as sidesway collapse). This occurs when the IDA curve becomes flat. Vertical collapse mechanisms, which are not directly simulated in the structural model, are not considered in this assessment. The companion paper (Liel et al. 2010) provides explanation for how these additional collapse modes but could be accounted for.

Figure 3a presents sample results from incremental dynamic analysis for a four-story space frame building (ID1008). For this structure, the median collapse capacity (in terms of Sa(0.94s)) is 1.59g for the set of 39 ground motion pairs.

3. Construct a collapse fragility function based on the IDA results, which represents the probability of collapse as a function of ground motion intensity. To approximately account for three-dimensional ground motion effects (i.e. the maximum ground motion component), the lower collapse capacity (in terms of $Sa(T_1)$) from each pair of motions is recorded as the building collapse capacity. From the resulting collapse data, the median collapse capacity and dispersion, due to record-to-record variability, are then computed.

Figure 3b presents such collapse fragility curves for the four-story building used previously in Figure 3a. The square markers show the empirical cumulative distribution function of the collapse data from Figure 3a (i.e. each point represents the collapse capacity for a single earthquake record), and the solid line shows the lognormal distribution fit to the empirical data. The fitted median collapse capacity (in terms of Sa(0.94s)) is 1.59g and the fitted logarithmic standard

deviation, representing the so-called record-to-record (RTR) variability ($\sigma_{LN,RTR}$), is 0.38.

4. Increase the dispersion in the collapse fragility to account for structural modeling uncertainties.

Figure 3b shows this adjusted collapse capacity distribution by the dashed line. Liel et al. (2009) and Haselton and Deierlein (2007) have shown how introducing this additional dispersion in the collapse fragility can approximately account for the effects of uncertainties in the structural modeling parameters, but this approximation is only suitable for collapse probabilities in the lower tail of the fragility curve (Liel et al. 2009). Based on uncertainties in the nonlinear component models (e.g., the capping rotation and post-peak softening slope shown in Figure 2), the modeling uncertainty is calculated as $\sigma_{LN,modeling} = 0.5$ (Haselton and Deierlein 2007). When combined with the record-to-record uncertainty of $\sigma_{LN,RTR} = 0.38$, the resulting total dispersion is $\sigma_{LN,total} = 0.63$, shown by the dashed curve labeled RTR+Model.

5. Adjust (increase) the median of the collapse fragility curve to account for the ground motion spectral shape effect.

Figure 3b shows this adjusted collapse capacity distribution by the dotted line. For this example, the median collapse intensity is increased from 1.59g to 2.22g (by a factor of 1.4). As described by Haselton et al. (2010) and FEMA P-695 (FEMA 2009, Appendix B), this so-called ε adjustment is based on the large ductility of the RC SMF structures and associated period shift that occurs before collapse, combined with a target value of $\varepsilon = 1.5$ for rare ground motions in the

high seismic regions of California. Buildings with lower deformation capacity, as well as sites and hazard levels with lower expected values of ε, would have a smaller adjustment.

6. Compute the collapse risk metrics of interest.

For the example in Figure 3b, the collapse margin ratio is 2.6, the conditional collapse probability ($P(C|Sa_{2/50})$) is 7%, and the mean annual frequency of collapse (λ_{col}) is 1.7×10^{-4} collapses/year.

COLLAPSE RISK FOR RC SMF BUILDINGS DESIGNED ACCORDING TO ASCE 7-02

Collapse analysis results for the 30 building archetypes are summarized in Table 1. Pertinent data includes the fundamental period of each archetype structural model, static overstrength from pushover analysis, collapse risk predictions, and maximum story and roof drifts at the onset of collapse. The resulting collapse risks are described by the following three measures, as listed in Table 1 and plotted in Figure 4:

Collapse Margin: The collapse margin is the ratio between the median collapse capacity and the 2% in 50 year ground motion level. This metric is similar in concept to a simple factor of safety. Overall, the collapse margins for the 30 RC SMF buildings range from 1.7 to 3.4, with an average value of 2.3.

Conditional Collapse Probability: The probability of collapse for the 2% in 50 year level of ground motion intensity, denoted $P(C|Sa_{2/50})$, can be read directly from the fragility curve. This is a convenient metric to gauge the collapse safety relative to the extreme ground motion intensity that is used as the basis of design in building codes . Overall, the RC SMF buildings have an average $P(C|Sa_{2/50})$ of 11%, with a range from 3% to 20%.

Mean Annual Frequency of Collapse: The mean annual frequency of collapse (λ_{col}) is obtained by integrating the collapse fragility with the site-specific hazard curve. Using the hazard curve from the Los Angeles site, the RC SMF buildings have an average λ_{col} of 3.1×10^{-4} collapses/year, with a range from 0.7×10^{-4} to 7.0×10^{-4} collapses/year. This range translates to a probability of collapse in 50 years of 0.4% to 3.4%.

While there are no clear standards that define the maximum acceptable collapse risk for buildings, there is some consensus that calculated values for the RC SMF archetypes are in a reasonable range. For example, the FEMA P-695 (FEMA 2009) Methodology to determine seismic response factors for new building systems, is based on a maximum collapse risk of 10% to 20%, conditioned on the maximum considered earthquake intensity. Additionally, the ASCE/SEI 7 building code has recently adopted new "risk consistent" seismic design maps, which have an implied collapse risk of 1% in 50 years (Luco et al. 2007), and which were developed based on an assumed collapse probability of 10%, conditioned on the maximum considered earthquake intensity. Finally, it is important to remember that the collapse risks reported herein were calculated from archetype designs that conform to current building code provisions. So, to the extent that the evolution of building codes reflects societal values, the calculated collapse risks have legitimacy implicit in the natural progression of building codes and standards.

In addition to quantifying the collapse risk, the nonlinear analyses provide insights into the collapse behavior and failure mechanisms of the archetype RC SMFs. Figure 5 illustrates the predominant collapse mechanisms for space frame buildings of various heights, and provides the percentage of ground motions that caused the predominant

mechanism. This figure shows the magnitude of the plastic deformation in each plastic hinge region, and when the plastic deformation exceeds $\theta_{cap,pl}$ then the circle is made red. For buildings with eight or more stories, fewer than 25% of the stories are typically involved in the collapse mechanism. For example, in 86% of the time history analyses, the 12-story space frame collapses in a two story mechanism over the first and second floors, and 9% of the collapses occur in a single first-story mechanism. In the remaining 5% of analyses, collapse involves a multi-story mechanism over the first three to four stories.

The IDA results also provide insightful data on the story and roof drifts at the onset of collapse. As shown in Figure 6, both the interstory drift ratios (IDRs) and roof drift ratios (RDRs) at collapse tend to decrease as the building gets taller, saturating at building heights of about 12- and 20-storyies. The maximum IDR decreases with increased height for two reasons. First, column plastic rotation capacities decrease as column axial stresses increase, due to higher compression (gravity) loads in the columns of taller buildings. Second, the taller frames are more susceptible to P-Δ effects, since they are designed with smaller base shear strength ratios (V/W), which leads to the onset of a negative stiffness at lower drift ratios. The roof drift capacity is a less transparent index since the roof drift capacity is based on the drift capacity of each individual story as well as the number of stories that are involved in the collapse mechanism.

INFLUENCE OF DESIGN PARAMETERS ON COLLAPSE SAFETY

Influence of Building Height

Figure 7 illustrates the relationship between building height and collapse safety for the standard perimeter and space frame designs. These results suggest that mid-rise (8-12

story) and low-rise (1-story) buildings have *slightly* higher collapse risk as compared to buildings of other heights. This occurs due to the minimum base shear requirement resulting in increased safety for taller buildings, and the slightly smaller deformation capacities of the 1-story buildings. However, these differences in collapse risk are small relative to the variations that will be observed later for ASCE 7-05 designs with lower base shear strengths, and for older non-ductile RC frames in a companion paper (Liel et al. 2009).

Influence of Space versus Perimeter Framing Systems

Comparisons between space and perimeter frame systems indicate that the perimeter frames have a higher collapse risk as compared to space frames. As observed from static pushover analysis results shown in Figure 8, space frames have higher static overstrength (Ω) as compared to perimeter frames, where overstrength is defined by the ratio of ultimate strength from pushover analysis to the design strength. On average, space frames have overstrength 2.5 times higher than perimeter frames for low-rise buildings and 1.2 times higher for the 12- to 20-story buildings. This difference occurs mainly because the beams in space frames are designed for proportionally more gravity load (relative to lateral load effects), which indirectly increases the lateral strength of the building. Space frame structures also have higher deformation capacity because of proportionally smaller P-Δ effects for the space frame buildings. The roof drift capacity (from nonlinear dynamic analyses) is 10% to 20% higher for space frame buildings (Figure 6). Figure 8 shows that the initial stiffness of the space and perimeter frame building is similar, which results from both building designs being designed up to the drift limits of ASCE 7-05.

Consequently, space frames have a higher level of collapse safety as compared to perimeter frames. On average, margins against collapse are 1.1 to 1.3 times higher for space frame buildings and mean annual frequencies of collapse are 1.3 to 2.2 times lower for space frame buildings. For the example pair of 8-story buildings (ID1012 versus ID1011), the $P(C|S_{a2/50})$ is 9% for the space frame building versus 19% for the perimeter frame building. These variations in collapse risk between the two frame types are related to differences in overstrength and deformation capacity.

Influence of Story-wise Variations in Strength and Stiffness

The distribution of strength and stiffness over the height of the building affects the degree of damage concentration in the structure, which has implications for both static overstrength and dynamic collapse resistance. This study looks at two variations in strength and stiffness distribution relative to a conventional baseline design: (1) a "uniform" design, where element size and reinforcement are kept constant over the full building height, and (2) a strength-irregular design (i.e. the weak-story "WS" designs in Table 1). The strength-irregular designs are based on the ASCE 7-02 definition of either (a) a "Weak Story Irregularity," in which the story lateral strength is less than 80% of the story above, and (b) an "Extreme Weak Story Irregularity," in which the story lateral strength is less than 65% of the story above. The lower (weak) stories of the WS designs are designed to meet code strength requirements, and the upper stories are strengthened such that the ratio between the strength of the weak and strengthened stories is either 80% or 65%. The "weak" stories either occur in the first-story only (WS-1) or in the first and second stories (WS-2), as noted in Table 1.

The resulting static overstrength (Ω) and collapse margin values from Table 1 are graphically illustrated in Figure 9. Referring specifically to Figure 9a, in all cases the design variations resulted in larger static overstrengths than the base case, with overstrength values up to 43% larger for the WS-1(65%) designs. As shown in Figure 9b, the effect of the strength-irregular and uniform member size design variants had a more variable effect on the collapse margins relative to the baseline designs. In all cases, the designs with the uniform member sizes had collapse margins that were equal to or larger (by up to 15%) than the base case. For strength-irregular buildings with "weak" first-stories only (WS-1), the collapse margins are larger than the baseline case (by up to 20%). For strength-irregular buildings with "weak" first and second stories (WS-2), the collapse margins decreased (by up to -15%).

Since the one-story strength-irregular designs performed better than the baseline designs, one might surmise that the strength-irregularities did not change the collapse mechanism. However, the strength-irregular designs exhibit better performance in spite of experiencing a higher percentage of first-story collapse mechanisms, as illustrated in Figure 10. In the baseline design (Figure 10a), the collapse mechanism typically involves the first and second stories (for 86% of ground motions). For the strength-irregular structures with relatively weaker first-stories, the collapse mechanism is more likely to occur in the 1st-story. Even so, the collapse capacity is higher for this strength-irregular design, presumably due to the increased overstrength in upper stories, and there is also considerable inelastic energy dissipation in other stories of the building prior to collapse. Note that these are the fully developed collapse mechanisms, which tend to exaggerate the deformations in the critical first story.

Effects of Bay Spacing and Foundation Fixity Assumptions

As indicated in Table 1, increasing the bay spacing from 20' to 30' spans in the 4-story and 12-story designs increased the static overstrengths by 12% to 22% and increased the collapse margins by 23% to 33%. Thus the differences in span lengths and associated design changes can significantly affect the collapse safety, largely due to the interrelationships between design parameters. For RC SMFs, we observed that changes in column spacing can affect the joint shear capacity design requirement, which in turn may necessitate larger column sizes. When this occurs, the larger columns tend to reduce the column axial stresses, thereby increasing plastic rotation capacities and collapse performance.

Similarly, the design foundation fixity assumptions can lead to changes in the first-story column sizes, which can affect collapse capacity. As indicated in Table 1, the change from a standard grade beam condition to a fixed or pinned base condition can lead to variations of up to 25% for static overstrength and 16% for collapse margins for the 1-2 story buildings. The increased column size associated with the pinned base assumption tends to increase the assessed collapse performance, whereas the fixed base assumption has the opposite effect.

IMPACTS OF POSSIBLE CHANGES TO SEISMIC DESIGN REQUIREMENTS ON COLLAPSE SAFETY

Historically, the development of seismic design requirements in building codes have been based on experience from past earthquakes and on engineering judgment. The findings presented here suggest that direct collapse modeling and performance-based earthquake engineering methods are now mature enough to deliver reasonable estimates of collapse

risk, and that such estimates can be used to inform important code and policy decisions. This section illustrates how performance-based collapse assessment can be used to inform decisions about building code provisions. Specifically, the impact of two altered seismic design requirements are evaluated: (1) a reduced minimum base shear requirement included in ASCE 7-05, and (2) the SCWB design criteria in ACI 318.

Reduced Minimum Base Shear Requirement, as in ASCE 7-05

The results presented thus far indicate that collapse risk is fairly uniform for buildings ranging in height from 1-20 stories at high seismic sites. These buildings were designed by the 2002 edition of the ASCE 7 provisions, which includes a minimum design base shear coefficient (C_s) requirement of 0.044g, based on the site hazard used in this study. This minimum base shear requirement controls the design of longer period buildings (i.e. the 12- and 20-story buildings), such that the requirement increases the strength of the building relative to the design ground motion intensity.

The minimum base shear requirement was reduced in the 2005 edition of ASCE 7 for sites with $S_1 < 0.6g$, leading to a significant decrease in the design base shear for taller buildings. Figure 11 illustrates this modification to the minimum base shear requirement more generally by showing an effective design R-factor for the buildings investigated in this study. This effective design R-factor is defined as $S_{D1}I / C_sT$ for buildings in the velocity domain (i.e. $T_S < T < T_L$), and this value differs from the code-defined design R-factor when the minimum base shear requirement controls the value of C_s. This shows that the minimum base shear requirement in ASCE 7-02 effectively introduces period-dependency in the design R-factor. These differences in the minimum design base shear can result in large changes to building design strength; for example, the design base shear

coefficient of a 20-story building designed for $S_1 = 0.599g$ is $C_s = 0.044g$ using ASCE 7-02 and is only $C_s = 0.022g$ using ASCE 7-05.

To examine the impacts of the minimum base shear requirement on seismic collapse, we redesigned the 12- and 20-story buildings according to ASCE 7-05 criteria with the reduced minimum base shear limit. The findings, presented in Table 2 and Figure 12, indicate that the ASCE 7-05 20-story perimeter frame building has a conditional probability of collapse that is 4 times larger and a mean annual frequency of collapse that is 11 times larger than that of the same ASCE 7-02 design. The 12-story building collapse probabilities are also affected, but to a lesser degree (note that the similar results for the 12-story space frames are just an oddity causes by design decisions). The reduced lateral strength of the ASCE 7-05 designs also introduces a trend whereby collapse risk increases with building height and leads to collapse mechanisms involving fewer stories. These results show that the ASCE 7-02 shear requirement is critical for ensuring relatively consistent collapse risk over height for flexible frame structures. Elimination of this requirement in the ASCE 7-05 provisions makes taller frame buildings significantly more vulnerable to collapse.

This observed increase in collapse risk for tall buildings with insufficient lateral design strength is consistent with findings from other researchers (e.g., Krawinkler et al. 2007, 2003). For frame buildings, a reduction in lateral design strength leads to more localization of damage in fewer stories of the building due to the P-Δ effects. This damage localization reduces the inelastic deformation capacity of the building, which results in a lower building collapse capacity. Krawinkler and Zareian (2007) have found that these localization effects are more pronounced in frame buildings than flexurally-

dominated shear wall buildings. Strength demands for taller buildings of various types as well as the governing failure modes, should be more thoroughly investigated.

These findings have important implications for the design of tall frame buildings and suggest that achieving uniform collapse safety for buildings of different heights requires that taller frames be designed with relatively more strength (as compared with shorter buildings). ASCE 7-02 provisions provided this additional strength through the minimum base shear requirement ($C_s = 0.044S_{DS}I$), which effectively reduces the design strength reduction factor for taller buildings. These research findings were made available to the ASCE 7 Seismic Committee, who subsequently passed a special amendment to ASCE 7-05 (Supplement No. 2) which reinstated the older minimum base shear requirement from ASCE 7-02 (ASCE 7-02 equation 9.5.5.2.1-3). This minimum base shear requirement solved the safety problem for taller frame buildings, but this is not the only viable approach; for example, a period-dependent R factor could have been created, more stringent strong-column requirements could have been used for taller buildings, or other similar approaches could have been pursued.

Effect of Strong-Column Weak-Beam (SCWB) Design Requirements

This section examines the relationship between the minimum column to beam strength ratio, as required in ACI-318 and comparable design provisions, and assessed collapse capacity. Increasing the required SCWB ratio delays column hinging, and thereby increases both (1) the number of stories involved in the collapse mechanism and (2) the inelastic deformation capacity of the entire frame.

To quantify the effects of the SCWB ratio, we designed two additional sets of 4-story space frames and 12-story perimeter frames (all designed otherwise according ASCE 7-

02). SCWB ratios were assumed to vary from the current standard for RC SMFs in ACI (2005) of 1.2 (ACI 2005) to a maximum of 3.0. In addition, values of the SCWB ratio smaller than the current requirement were evaluated. The minimum SCWB ratio considered corresponds to the ratio at which the majority of column designs were governed by flexural strength requirements rather than adjacent beam strength. This limit occurred at a SCWB ratio of 0.4 for the 4-story buildings and 0.9 for the 12-story buildings.

The assessed conditional collapse probabilities for these 4- and 12-story frames are summarized in Figure 13. These results show that the SCWB ratio has a dramatic impact on the collapse capacity of the 4-story building up to a ratio of about 1.0, beyond which additional increases do not significantly improve collapse performance. Whether coincidence or by design, this data suggests that the current ACI 318 SCWB ratio of 1.2 is appropriate for low-rise buildings. In contrast, the collapse capacity of the 12-story building consistently improves for all increases in the SCWB ratio.

Physically, increasing the SCWB ratio implies that the resulting building design has stronger columns, spreading damage over more stories of the building and improving the collapse capacity of the structure. Figure 14 illustrates the dominant collapse mechanisms for the 4-story buildings. As the SCWB ratio increases, the collapse mechanism involves a larger number of stories, distributing damage. A SCWB ratio of 2.0 is sufficient to achieve formation of an ideal collapse mechanism involving all stories of the building. Further increases in the SCWB ratio do not change the collapse mechanism and, accordingly, do not significantly increase the assessed collapse safety. The 12-story buildings, illustrated in Figure 15, show a similar trend with SCWB ratio,

except that the benefit does not saturate, even up to a SCWB ratio of 3.0. It is expected that increasing the SCWB ratio would continue to improve the collapse resistance of the structure, as more stories are engaged in the sidesway collapse mechanism.

The results shown in Figures 14 and 15 illustrate that the current SCWB capacity design requirements in the ACI provisions (i.e. SCWB = 1.2) do not result in a collapse mechanism that involves all stories of the building. Even so, the SCWB ratio of 1.2 may be roughly appropriate for the 4-story example based on the observation that the larger SCWB values do increase the number of stories involved in the final collapse mechanism but do not substantially increase the collapse capacity. In stark contrast, the taller 12-story building has much different behavior and illustrates (1) the important role of strong columns for distributing damage throughout the height of tall buildings, and (2) that significant benefits in terms of collapse performance could be gained by increasing the minimum SCWB ratio up to values of 3.0 and perhaps even larger. This finding is consistent with much previous research on this topic (e.g. Ibarra et al. 2003, Park and Paulay 1975, etc.). Based on these observations, design provisions could be modified to develop a building height-dependent SCWB requirement to achieve more consistent risk levels. It may also be beneficial to vary the required SCWB ratio over the height of the building, with larger ratios in the lower stories of the building where the collapse mechanism is forming under the largest P-Δ effects. A more complete analytical study, including more buildings of various heights, would be useful to better investigate these possible modifications to design requirements.

SUMMARY OF MAJOR FINDINGS

Collapse safety predictions were presented for a set of 30 representative RC SMF buildings designed according to ASCE 7-02 (ASCE 2002) and ranging in height from 1 to 20 stories. This assessment found that the probability of collapse, conditioned on occurrence of a ground motion with spectral intensity exceeded with 2% likelihood in 50 years, ranged from 3 to 20% with an average of 11%. The predicted mean annual frequency of collapse varies from 0.7×10^{-4} to 7.0×10^{-4} with an average of 3.1×10^{-4} collapses/year, depending on the building considered. Among the buildings considered, collapse risk is relatively consistent over building height and perimeter frame structures tend to be more vulnerable than space frame structures.

Studies of strength-irregular design variations that involved over-design of upper story members to create story strength irregularities in the first- and second-stories indicate that, when limiting story strength irregularities to the maximum values permissible by ASCE 7-02, the benefits of increased strength in the upper stories tend to offset the negative effects of localized damage in the lower stories.

This paper further examines the use of direct collapse simulation to quantify the effects that modifications to seismic design requirements would have on collapse safety. Comparative studies of 12-story and 20-story buildings demonstrate that the reduction of the minimum base shear requirements between the 2002 and 2005 editions of the ASCE-7 provisions dramatically increased the collapse risk in tall (long-period) buildings. This increased risk corresponds to an increase in the mean annual frequency of collapse by more than an order of magnitude (a factor of 11). As a result of this study, the ASCE 7 Seismic Committee has reinstated the minimum base shear requirements into the 2005

edition. It would be useful to extend this effort to systematically investigate strength demands for buildings with other types of structural systems and failure modes. Such additional studies could be used to determine the lateral design strengths required for consistent collapse safety between buildings of various structural systems and heights.

The study also investigated the influence of SCWB requirements on the collapse safety of buildings of different heights. Comparative studies of 4-story and 12-story buildings demonstrate that the current requirements of ACI-318 that column flexural strength exceed beam flexural strength by a factor of 1.2 is reasonable to control collapse safety in low-rise buildings. On the other hand, it may be possible to achieve significant reductions in collapse risk of taller buildings by increasing the SCWB ratio, thereby engaging more stories in the collapse mechanism. A more complete study of ACI 318 SCWB requirements would be useful to improve understanding of how such provisions affect collapse safety by investigating a larger number of buildings of various heights, the effect of a story-dependent SCWB ratio and the costs and benefits associated with each alternative.

The collapse evaluations presented in this paper include a variety of approximations, such as characterizing future ground motions that we have never experienced to date and which are intense enough to collapse a modern building (the current state-of-the-art methods are entirely approximate), identifying representative structural configurations for modern RC frames, and developing analytical models which are simplifications of the true behavior (e.g. employed models do not capture axial-flexural interaction effects). For perimeter frame buildings, both the beneficial effects of the slab system (added strength and stiffness) and the possible detrimental effects (possible vertical collapse of the slab) are not accounted for. In the future, experimental data are needed where slab-column connections with modern

detailing are tested to large levels of deformation, in order to verify that damage at the slab-column will not result in loss of vertical load carrying ability. The large uncertainties in ground motion characterization and structural collapse modeling result in a relatively high level of uncertainty in the (absolute) measures of collapse risk reported in this study. Even so, relative comparisons of collapse risk (e.g. the impacts of the minimum base shear requirement) are still useful for understanding seismic safety and for improving our structural design methods.

Overall, the studies presented herein demonstrate the capabilities of nonlinear dynamic analysis to assess collapse, and the important role that such assessments can play in establishing and refining building code requirements to provide more consistent building safety. Moreover, quantitative risk-based measures of collapse safety offer opportunities to engage public officials and other stakeholders in setting policies and standards for seismic design. A further application of these collapse safety assessments to assist in crafting policy development for non-ductile reinforced concrete buildings is presented in a companion paper by Liel et al. (2010).

interpretations contained in this publication, and no warranty is offered with regard to the results, findings and recommendations herein, either by PEER, FEMA, or ATC (its directors, members, or employees). These organizations and individuals do not assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any of the information, product or processes included in this publication.

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