

# Assessing Building System Collapse Performance and Associated Requirements for Seismic Design

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## Abstract

Methods to assess collapse safety using inelastic time history analyses are employed in an Applied Technology Council project (ATC-63) to develop a "Recommended Methodology for Quantification of Building System Performance and Response Parameters". Supported by the Federal Emergency Management Agency and utilizing simulation techniques developed by the Pacific Earthquake Engineering Research Center, the ATC-63 methodology establishes a consistent basis to assess seismic collapse safety of buildings and, thereby, the underlying design requirements. Key features of the ATC-63 methodology are reviewed, including its application to assess the collapse performance of reinforced concrete moment frame buildings designed according to modern building code provisions. The assessment indicates that modern ductile moment frames have a probability of collapse of up to 20% when subjected to ground motions whose intensity corresponds to that of the Maximum Considered Earthquake (MCE) design spectra. Comparisons are included to illustrate how the methodology can be used to evaluate the influence of specific design parameters, such as minimum base shear requirements in tall buildings on collapse behavior.

## Introduction

Building code provisions for earthquake safety have largely been developed based on judgments informed by observed performance of buildings in past earthquakes, laboratory testing of structural components, and analysis of idealized models. While building codes imply there to be a low chance of collapse, codes are generally silent on ways to evaluate the collapse risk. Thus, the actual safety of modern code-conforming buildings is unknown, as is the relative safety

between different building systems and materials. The lack of methods to quantify collapse safety is an impediment to (a) the improvement of existing seismic design requirements for buildings, and (b) the development and adoption of new innovative seismic force resisting systems and components.

Advancements of the past two decades in earthquake risk assessment and performance-based engineering are making it possible to rigorously evaluate the collapse safety of buildings under earthquake ground motions. Among the major advancements in the United States are those associated with HAZUS, FEMA 273/356, the SAC Joint Venture, the CUREE-Caltech Woodframe Project, PEER's research on performance-based engineering, and complementary research of MAE and MCEER. With a focus on assessing large inventories of existing buildings of various construction types and quality, HAZUS introduced a probabilistic framework where building fragility curves were used to relate building performance to the intensity of ground shaking (e.g., Kircher et al. 1997). Around the same time, FEMA 273 (1997) introduced detailed guidance on applying nonlinear analysis methods and acceptance criteria to assess nonlinear performance of structural components as a function of earthquake demands. As part of a large project to address damage to steel-framed buildings in the Northridge earthquake, the SAC Joint Venture introduced a probabilistic approach to assess building collapse using nonlinear time-history analyses, considering the inherent uncertainties in ground motions and nonlinear structural behavior (FEMA 2000, Cornell et al. 2002, Hamburger et al. 2003). The CUREE-Caltech project employed similar concepts and developed models to assess the performance of residential wood-frame buildings (Porter et al. 2001, CUREE 2004). More recently, PEER has integrated and extended these concepts further to develop a comprehensive performance-

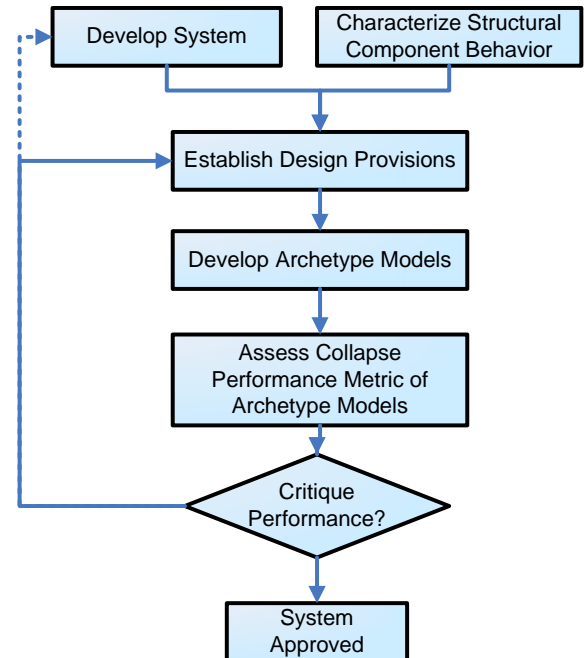
based framework that incorporates improved nonlinear analysis models and criteria to simulate building performance from the onset of damage up to collapse (Krawinkler et al. 2004, Deierlein 2004).

In a similar vein to these previous initiatives, the ATC-63 project provides a systematic method to assess collapse safety for the purpose of assessing the adequacy of structural design standards and building codes (ATC-63 2007). The resulting guidelines represent contributions of practicing engineers, researchers, industry and government representatives who served in various capacities on the ATC-63 project. Among the distinguishing aspects of the ATC-63 approach are (a) the introduction of building archetypes to assess the collapse safety of general classes of building seismic systems, (b) integration of nonlinear analysis and reliability concepts to quantify appropriate capacity margins, measured relative to the maximum considered earthquake intensity, (d) quantifying uncertainty parameters in building code provisions for seismic resisting systems, and (c) specification of a set of ground motions and scaling procedures to represent extreme (rare) ground motions. The ATC-63 procedure is demonstrated herein to assess the performance of code-conforming reinforced concrete special moment frames (RC-SMF), but its real value is to facilitate the assessment and building code adoption of new seismic resisting systems.

## Overview of ATC-63 Methodology

The overall process for the ATC-63 acceptance procedure is illustrated in Figure 1. As suggested by the top three boxes, the procedure presumes that the structural system under evaluation is clearly defined and has design provisions that are substantiated by test data to characterize structural behavior. For existing systems, such as RC-SMFs, the design provisions are envisioned to encompass the general seismic design requirements of ASCE 7 (2005) combined with the detailed requirements of ACI-318 (2005). For newly proposed systems, the system configurations and details would need to be clearly defined along with appropriate design criteria. The design criteria would typically include the general requirements of ASCE 7 (2005) along with the proposed system-specific seismic response factors ( $R$ ,  $C_d$ ,  $\Omega_o$ ), drift limits, height and usage restrictions (if any), and structural design and detailing requirements.

With the system design requirements defined, the next step is to develop a series of archetype models of the structural system, which reflect the range of applications and seismic behavioral aspects of the system. Development of the archetype models begins with definition of an idealized model that reflects salient features that impact the collapse response of the structural system. For example, in the case of

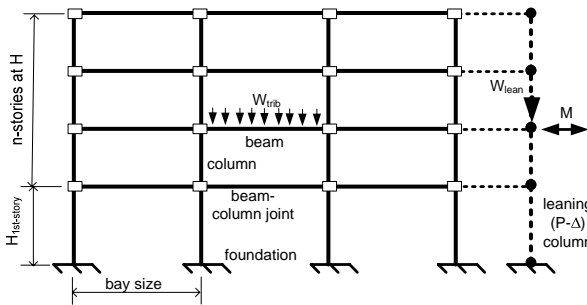


**Figure 1 – Schematic Flowchart of ATC-63 Methodology for System Performance Assessment**

ductile moment resisting frames, the three-bay multi-story frame model shown in Figure 2 is considered to capture the important aspects of beam, column and beam-column joint response. The three-bay configuration is judged to be the minimum number of bays necessary to capture effects such as overturning forces in columns and a mix of interior and exterior columns and joints. Multiple realizations of the idealized archetype models are then created to represent the expected range of building heights, bay widths, gravity load ratios, and seismic design categories for the archetype system model. Depending on the structural system type, the archetype study is expected to include about twenty to thirty independent design realizations of the idealized archetype analysis model.

The collapse capacity of each of the archetype models is evaluated by nonlinear time-history analyses under a prescribed set of ground motions, which are scaled to reflect specified earthquake ground shaking intensities. The nonlinear assessment must account for all likely modes of strength and stiffness degradation that can lead to earthquake-induced collapse. Ideally, all of the likely modes of failure are incorporated directly in the time history simulation; however, the method makes provision to check collapse modes that are not simulated directly in the analysis.

Once the nonlinear time history analyses are complete, the calculated collapse capacities of the archetype models are



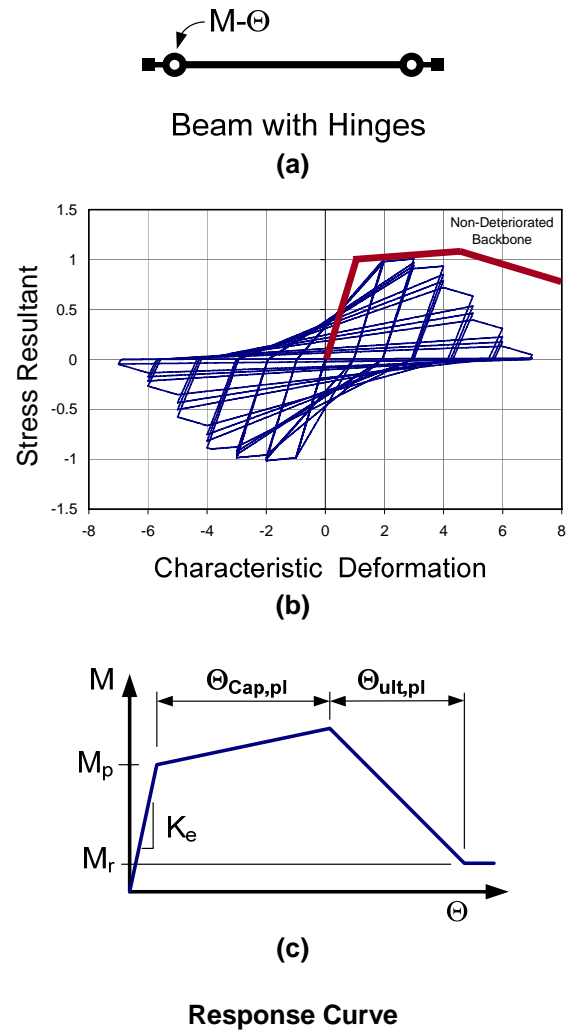
**Figure 2 – Idealized Analysis Model for Special Moment Frame Archetype**

evaluated to determine whether the proposed system and design requirements provide adequate collapse safety. The acceptance criterion is expressed as a minimum required collapse margin between the median value of the collapse capacity and the intensity of the Maximum Considered Earthquake (MCE). The assessment incorporates statistics from the nonlinear analysis and other factors that account for uncertainties in the nonlinear structural component and system behavior and how accurately the archetype models represent actual conditions. As shown by the return arrow in Figure 1, where the system does not meet the required performance, one can iterate on the system definition and/or design provisions to increase the collapse capacity. For example, one could revise the seismic response factor, i.e.,  $R$ -value, to adjust the design force requirements, or alternatively, revisions could be made to other parameters (such as capacity design or detailing requirements) to adjust the strength and deformation capacity of the structure.

### Nonlinear Analysis Model

Nonlinear time-history analysis is a central ingredient of the collapse assessment, the accuracy of which depends on how faithfully the model captures the strength and stiffness degradation that can lead to structural collapse. To characterize the modeling requirements, it is useful to distinguish between sidesway and vertical modes of collapse. Sidesway collapse occurs when the lateral strength and stiffness become insufficient to resist destabilizing  $P-\Delta$  effects, leading to large interstory drifts. Vertical collapse can arise due to loss in vertical load carrying capacity of one or more components in the structure, such as punching failure at a slab-column joint or loss in axial capacity of a column. Assessment of sidesway collapse of indeterminate systems is best accomplished through nonlinear response analysis of the entire system to account for inelastic force redistribution. On the other hand, vertical collapse, which is generally more difficult to simulate directly, tends to be more localized and can be evaluated on a component-wise basis using imposed deformations and forces from the overall analysis.

Using as an example reinforced-concrete moment frames, accurate modeling of strength and stiffness degradation leading to sideways collapse can be achieved by integrating degrading hinge-type models of the type shown in Figure 3 into the archetype analysis models of Figure 2. A few key aspects of this model are the characterization of the post-peak softening branch of the monotonic backbone curve (Figure 3c) and the degrading hysteretic response (Figure 3b). As shown by Ibarra et al (2003, 2005), the post-cap degrading portion of the monotonic backbone curve is essential to simulating collapse due to the combined effects of inelastic softening and  $P-\Delta$  effects. For the collapse analysis results presented later, Haselton and Deierlein (2007) have calibrated the concentrated spring model of Figure 3 to capture the



**Figure 3 – Example of Concentrated Hinge-Type Model or Nonlinear Frame Analysis (a) Beam with Hinge Model, (b) Hysteretic Response, and (c) Monotonic Backbone Curve**

nonlinear response of reinforced concrete beams, columns, and beam-column joints. The calibration is done to match the characteristic (or median) values of the reinforced concrete components based on their expected strengths. Uncertainties in the model parameters and their effect on the collapse assessment are incorporated either through adjustments to the collapse fragility.

## Input Ground Motions

In concept, the ground motions for the collapse analysis of a *specific* building should reflect the characteristics of the ground motion hazard and geotechnical conditions of the specific building site. However, since the ATC-63 method is geared toward assessing building code design provisions that can be generally applied to any site, the input ground motions and hazard information are developed in a generic sense. To this end, the ATC-63 guidelines include two suites of ground motions along with specific rules for scaling these. One set, termed the “Far-Field” record set includes twenty-two ground motion pairs recorded at sites located greater than 10 km from fault rupture. The second, termed the “Near-Field” set includes twenty-eight pairs of motions recorded at sites located within 10km of fault rupture. Records in each set were selected based on a set of criteria to help ensure an unbiased suite of motions that represent strong ground motion shaking with earthquake magnitudes of 6.5 to 7.9. Records in each set are normalized by their peak ground velocities to reduce the scatter between records while preserving variations that are consistent with variations observed in ground motion attenuation functions at large earthquake intensities. Shown in Figure 4 is plot of the normalized response spectra of the Far-Field record set. Further details on the record selection and normalization procedure are given in ATC-63 (2007).

To assess the collapse capacity using the incremental dynamic analysis procedure, the amplitude of the ground motion records are scaled based on the fundamental vibration period of the building under consideration. The scaling procedure is illustrated in Figure 5, where the average response spectra of the Far-Field record set is overlaid with spectra for two MCE level ground motions.

The “low” and “high” MCE demand spectra shown in Figure 5 reflect hazard levels associated with Seismic Design Categories C and D of ASCE 7 (2005). The “low” level is based on an MCE intensity of  $S_{MS} = 0.56g$  (short period) and  $S_{MI} = 0.24g$  (1 second period); and the “high” level is associated with  $S_{MS} = 1.5g$  (short period) and  $S_{MI} = 0.90g$ . As shown in the figure, the median spectrum of the unscaled Far-Field record set is slightly higher than the low seismic MCE. Superimposed on the MCE demand spectra is the scaled median of the Far-Field set, where the record set is

scaled (anchored) to the demand spectra at a period of about 0.8 to 0.9 seconds, corresponding to the natural period of the 4-story archetype frames. In this example, records in the Far-Field set are scaled by a factor of 0.65 to match the low hazard and 2.4 to match the high hazard. For analyses of other buildings, the record set would be re-scaled based on the first-mode period of the building in question.

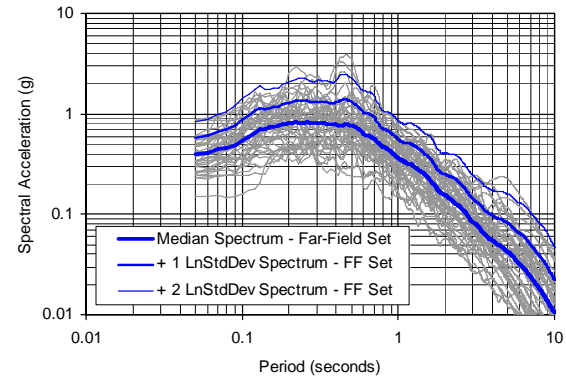


Figure 4 –Normalized Far-Field Record Set

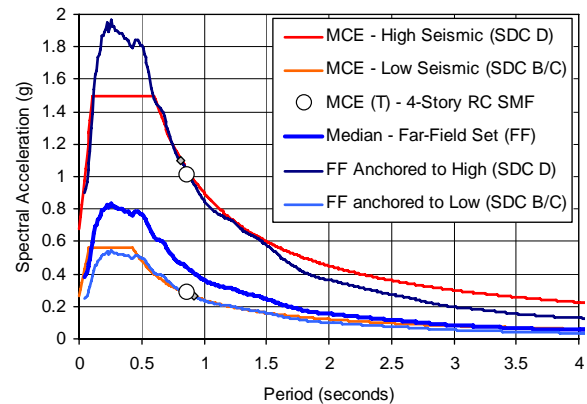
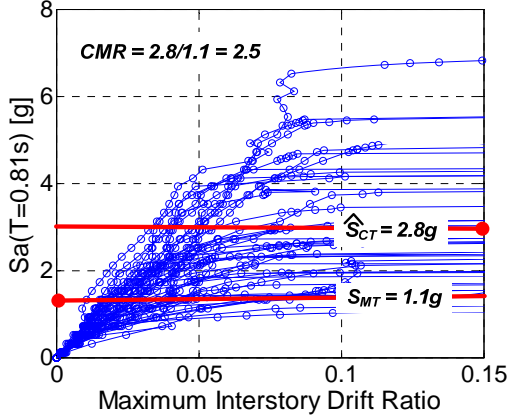


Figure 5 – Comparison of MCE Demand Spectra and Median Spectra for Far-Field Record Set Scaled to Demand Spectra at a Period of 0.8 Seconds

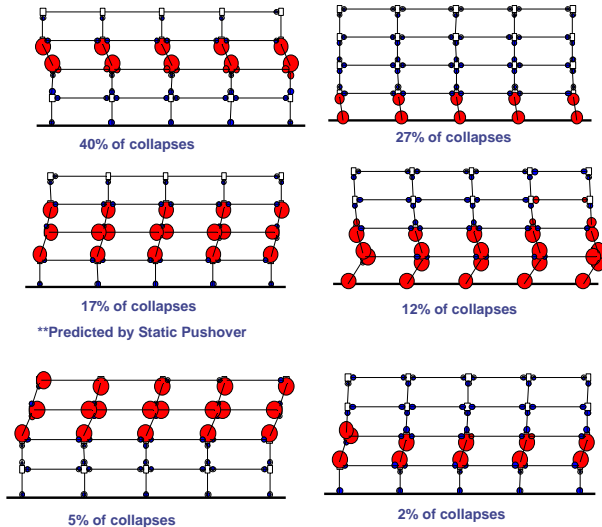
## Incremental Dynamic Analysis

Incremental Dynamic Analysis (IDA) is a technique to systematically process the effects of increasing earthquake ground motion intensity on structural response up to the point of collapse (Vamvatsikos and Cornell, 2002). Shown in Figure 6 are results of IDA for a four-story reinforced concrete special moment frame that was analyzed for each of the 44 ground motion records of the Far-Field record set at increasing intensity. Each point in the figure represents the results of one nonlinear time-history analysis – relating the spectral intensity of the record to the peak interstory drift recorded during the analysis. Each curve represents the response of the structure to a single ground motion whose

intensity is increased until a collapse mechanism is observed. Collapse is detected when excessive drifts occur under small increases in ground motion intensity.



**Figure 6 –Incremental Dynamic Analysis (IDA) Collapse Results**



**Figure 7 – Frame Collapse Modes**

Implicit to the IDA is the definition of a ground motion intensity measure, which is used as a metric to define ground motion amplitude scaling (i.e., the vertical axis of Figure 6). While it is generally agreed that ground motion intensity is best described by some type of spectral value (typically acceleration or velocity), the appropriate choice of ground motion intensity measure is a subject of ongoing debate. One popular intensity measure is the spectral acceleration at the period of the first (fundamental) mode of vibration of a structure. This has the advantage of being simple to define (assuming that one knows the fundamental period of the structure) and it provides a convenient way to integrate with standard seismic hazard curves (which are also defined in

terms of spectra at specific vibration periods). However, when the goal is to evaluate the response for specific intensity scenarios, e.g., the MCE intensity, scaling of each record based on the first-mode spectral value of the structure tends to artificially constrain the inherent uncertainty in the intensity measure assessment. Scaling to a single period also tends to bias the response to the elastic first-mode behavior, as compared to higher modes and the change in modes due to inelastic softening.

The ATC-63 methodology adopts a strategy of scaling the ground motions on a set-wise basis, where the intensity measure for all of the records in the set is based on the median response spectra of the entire record set. Further, the scaling of the spectral average is done at the period of the first (or natural) mode of vibration of the structure. Referring back to the case shown in Figure 5, this is why the scaled average spectra are shown “anchored” to the demand spectra at a period of about 0.8 seconds, which is the natural period of the four-story reinforced concrete frame model. Thus, referring to Figure 6, the spectral index on the vertical axis, corresponds to the spectral acceleration value (at  $T = 0.8$  seconds) for the median (anchor point) of the Far-Field record set. In this sense, one can think of the entire record set as representing a ground motion earthquake scenario that is scaled about the median of the set.

The large variability in the IDA response plots reflects both the spectral variability of each record about the median (see Figure 4) and other features of the ground motions, e.g., duration and frequency content, which are not fully reflected in the spectral acceleration intensity. The inherent variability that different records have on the structural response is further evident in Figure 7, which shows the various collapse modes observed from the IDA study of a four-story RC SMF. In spite of the fact that the RC SMF design conforms to current design standards (ASCE 7-05 and ACI 318-05), collapse occurred due to a single story mechanism for over two-thirds of the records (40% caused a third story mechanism, 27% a first story mechanism, and 2% a second story mechanism). Only 17% of the records led to collapse by the multi-story mechanism that was observed in a static pushover analysis of the frame.

Despite occurrence of story mechanisms, the 4-story RC SMF frame performed well in the sense that the calculated collapse intensities generally exceeded the MCE intensity by a large margin. Referring to Figure 6, the median collapse capacity of  $S_{CT} = 2.8g$  is about 2.5 times the MCE intensity of  $S_{MT} = 1.1g$ . The IDA collapse statistics are illustrated in the collapse fragility curve of Figure 8, which relates the probability of collapse to the spectral intensity of the ground motion. Characterized by a lognormal distribution, the collapse fragility curve is a cumulative distribution function

defined by the median collapse intensity ( $S_{CT} = 2.8g$ ) and the dispersion given by the standard deviation of the natural log,  $\sigma(\ln(Sa))=0.45$ , both of which are obtained from the IDA data. Referring to Figure 8, this collapse fragility curve indicates that the probability of collapse at the MCE intensity is less than 2% (i.e.,  $P_{collapse}[Sa=S_{MT}] < 0.02$ ). However, the plot of Figure 8 is an interim collapse fragility curve that does not account for modeling uncertainties and other aspects of the ground motions that are important for accurate characterization of collapse. These are considered next.

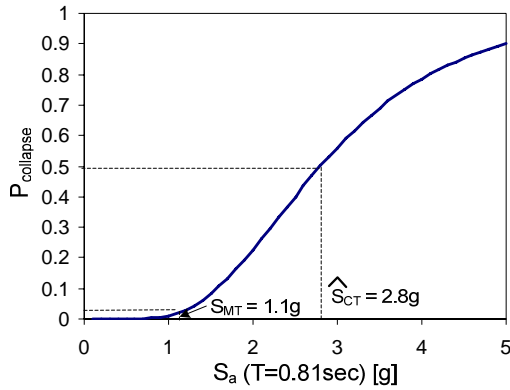


Figure 8 – Collapse Fragility Curve

### Collapse Fragility Curve

Three adjustments need to be applied to the IDA collapse data before the collapse assessment is considered complete. The first modification is to adjust for collapse mechanisms that are not captured directly by the nonlinear time history analysis. The second is to adjust for uncertainties in the analysis model, so-called modeling uncertainties. The third is to adjust for the unique spectral shape effects of extreme (rare) ground motions that cause collapse.

**Non-simulated Collapse Modes:** In the event that certain deterioration or collapse modes are not modeled explicitly in the nonlinear analysis, they are incorporated by adjusting the calculated collapse point in the nonlinear IDA assessment. For example, consider the case of a RC ordinary moment frame (RC-OMF), where the design provisions do not enforce capacity design requirements to prevent shear failure of RC columns and where the nonlinear frame analysis models do not directly simulate column shear failure. In this case, a practical approach to adjusting the IDA results to account for column shear failure is illustrated for one ground motion IDA curve in Figure 9, where the sideways collapses that are simulated directly in the nonlinear analysis is denoted “SC” (for simulated collapse) and the check for column shear failure is denoted “NSC” (for non-simulated collapse).

The SC point in Figure 9 is the same as the limit points shown previously in Figure 6, where the nonlinear IDA model simulates directly the deterioration and collapse. Where other “non-simulated” modes of failure are possible (e.g., collapse triggered by column shear failure), then the response quantities or demand parameters from the analysis are monitored and checked against criteria for the component limit state criteria. For shear critical RC columns, such models would likely include some combination of column forces (axial and shear) and deformations (e.g., Elwood & Moehle, 2005). As shown in Figure 9, if the NSC limit state is detected before the SC point, then the predicted collapse capacity is scaled back from  $S_{CT(SC)}$  to  $S_{CT(NSC)}$ .

Using the approach illustrated in Figure 9, the NSC collapse modes would be directly incorporated in the IDA results. In other words, the collapse statistics for the IDA results (median  $S_{CT}$  and dispersion  $\sigma(\ln(Sa))$ ), would be reported based on the plots whose collapse value is controlled by the lesser of the SC or NSC limits. Note that for consistency with the SC assessment, the criteria used to judge the NSC limit state should be based on the median capacity for that limit.

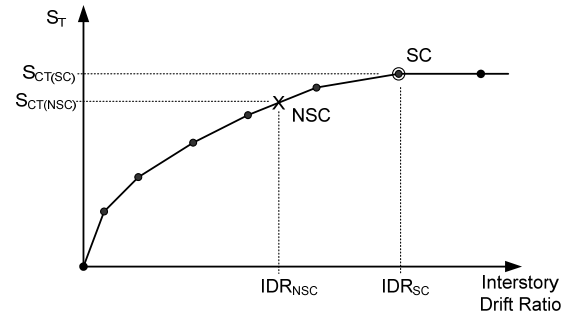
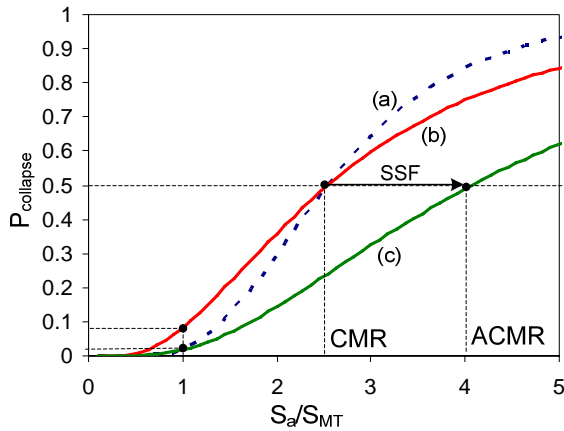


Figure 9 – Illustration of adjustment to IDA collapse analysis for “non-simulated” collapse modes

**Modeling Uncertainty:** As described until this point, the nonlinear analysis model has been based on the average (or characteristic) properties of the structure, such that the only uncertainties in the collapse assessment are those associated with the variations in response for alternative ground motions. Studies by Ibarra and Krawinkler (2003) and Haselton and Deierlein (2007) and have shown that certain model parameters, such as the inelastic capping rotation and post capping slope (see Figure 3) can have a significant effect on the collapse performance. They have further shown that the additional variability in response introduced by uncertainties in the structural parameters (so-called “modeling uncertainty”) can be reasonably accounted for by adjusting the dispersion ( $\sigma(\ln(Sa))$ ) of the collapse fragility curve. Graphically, the increased dispersion is shown by comparing curves (a) and (b) in Figure 10, where the dashed curve (a) is the same as the IDA results in Figure 8, reflecting

uncertainties due to the ground motions, and the solid curve (b) includes the additional modeling uncertainty.

From detailed studies of nonlinear RC component response and its effect on the four story frame example, Haselton and Deierlein (2007) have determined the modeling uncertainty to increase the dispersion in the collapse fragility curve from  $\sigma(\ln(S_a)) = 0.45$  (for record uncertainties) to  $\sigma(\ln(S_a)) = 0.65$  (for combined record and modeling uncertainties). This reflects, for example, variability in the capping point rotation (see Figure 3) of  $\sigma(\ln(\theta_{cap})) = 0.6$ , which is carried through the nonlinear time history and IDA analyses along with other factors to result in the cumulative collapse intensity uncertainty of  $\sigma(\ln(S_a)) = 0.65$ .



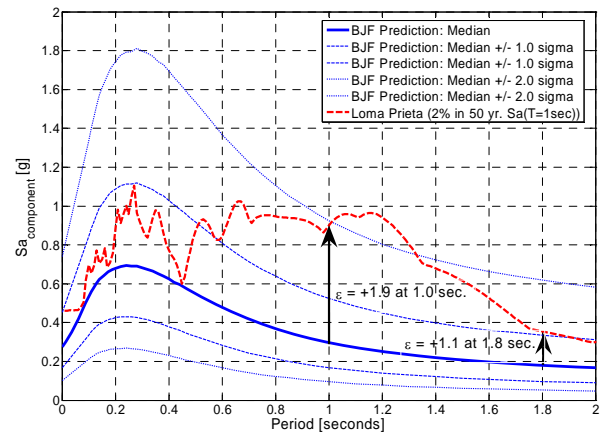
**Figure 10 – Collapse Fragility with Adjustments for Modeling Uncertainty and Spectral Shape**

The horizontal axis in Figure 10, corresponding to earthquake intensity, has been normalized by the intensity of the MCE. Thus, the horizontal index represents the ratio between the earthquake intensity that causes collapse and the MCE intensity for which the structure has been designed. The collapse ratio at the median point is termed the Collapse Margin Ratio (CMR). The CMR = 2.5 for the median in Figure 10 is the same median margin as in Figure 8 and is unchanged by adjustments due to the uncertainty. While the median margin is unchanged, the modeling uncertainties increase the probability of collapse at the MCE (at  $S_a/S_{MT} = 1$ ) by about four times, from less than 2% to about 8%.

**Spectral Shape Effect:** Recent research (Baker and Cornell 2006, Haselton and Baker 2006) has demonstrated the importance of considering the unique spectral shape of extreme ground motions when evaluating collapse at ground motion intensities for rare earthquakes. For example, as shown in Figures 8-10, code-conforming structural systems are expected to resist ground motions that are scaled to earthquake intensities on the order of one to three times the MCE. Assuming that the MCE intensity for coastal

California has a return period of 1000 to 2500 years, the median collapse intensities are very rare.

Generally speaking, high intensity MCE ground motions in coastal California (and similar regions) are *infrequent* ground motions that can occur under rather *frequent* earthquakes. For example, shown in Figure 11 are the hazard spectra calculated from the Boore/Joyner/Fumar (BJF) attenuation function for a magnitude 6.9 earthquake at a distance of 11 km. Return periods of such earthquakes on active faults are on the order of 150 to 500 years. The thick line corresponds to the median value from the BJF attenuation functions, and each of the lines above and below represent extreme values, corresponding to  $\pm 1\epsilon$  and  $\pm 2\epsilon$ , where  $\epsilon$  is a standard measure of the variability in ground motions. Superimposed on this plot is a response spectra for a ground motion recorded from the Loma Prieta earthquake, which is consistent with the M6.9 and 11 km distance used in the attenuation estimates. At a period of  $T=1$  second, the Loma Prieta record has a spectral acceleration of about 0.9g, which corresponds to an intensity prediction that is  $\pm 2\epsilon$  above the BJF median. The 0.9g intensity is also representative of the 2500 year return period intensities in a Seismic Design Category D site.



**Figure 11 - Comparison of an observed ground motion spectrum with spectra predicted by an attenuation function (Haselton and Baker 2006)**

The Loma Prieta record shown in Figure 11 can be referred to as a “ $+2\epsilon$  record at  $T=1$  seconds”. It is in fact extreme spectral values like this that govern the high intensity long return period hazard values. An important feature of these extreme motions is that they generally do not have extreme high values at all periods. As is evident from the record in Figure 11, motions that have high intensities at one period tend to have lower intensities at other periods. For example, at  $T=0.45$  seconds this Loma Prieta motion has a neutral “zero  $\epsilon$ ” intensity and at  $T=2$  seconds has a  $+1\epsilon$  intensity. Thus, the  $\epsilon$  parameter is a function of both the ground motion and the period at which the spectral quantity is evaluated.

If one were to collect a bin of records, similar to the Loma Prieta record, with high  $\varepsilon$  at  $T=1$  second, the average spectra for that bin would tend to be peaked near  $T=1$  second and drop off at higher and lower periods. Thus, the spectra of ground motions that drive the high hazard at  $T=1$  second has a different shape than the spectra given by typical “equal hazard” curves. This is important, since ground motions whose spectral shape drops off at longer periods are less damaging than other records whose intensities do not reduce.

When scaling ground motions to represent extreme (rare) shaking intensities for a certain period range (typically near the fundamental vibration mode), it is important to consider this so-called “ $\varepsilon$ -effect” or “spectral shape” effect. In nonlinear IDA simulations, this effect can be included by either (a) choosing ground motions that have positive  $\varepsilon$  values at the predominate period that defines the ground motion hazard, or (b) adjusting the collapse fragility to account for the spectral shape effect (Haselton and Deierlein 2007). The latter approach has been adopted for the ATC-63 methodology since it can be implemented with a single set of ground motions that can be used and scaled at different periods, depending on the structural system.

For ductile systems in coastal California, where the MCE hazard tends to be dominated by positive  $\varepsilon$  records (on the order of  $+1.5\varepsilon$  to  $+2\varepsilon$ ), the ATC 63 method permits a 1.6X shift in the median collapse intensity. For the four-story RC SMF example, this spectral shape factor (SSF) shift is illustrated by fragility curve (c) in Figure 10. The median point of this curve is termed the Adjusted Collapse Margin Ratio (ACMR). As indicated, this SSF shift results in a significant reduction in the probability of collapse at the MCE – on the order of a 4X reduction in this case.

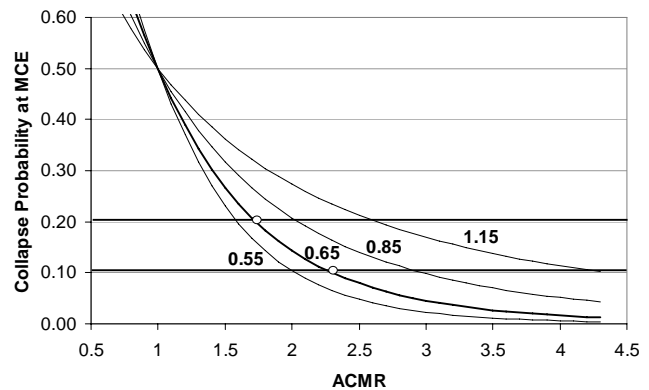
It should be emphasized that the SSF shift depends on both the system ductility and site hazards dominated by positive  $\varepsilon$  records. Systems with minimal ductility do not benefit from the SSF effect, since they tend to reach their collapse condition without appreciable period elongation. Locations, such as the eastern US, where the MCE hazard is dominated by the recurrence of the earthquake event, as opposed to variability in the attenuation functions, will tend to be dominated by “zero- $\varepsilon$ ” records and, thus, do not qualify for the SSF adjustment. The ATC-63 report provides guidance on applying SSF shift, which has multipliers that range from 1.0 (no effect) to 1.6 (maximum benefit).

### Relating ACMR to Collapse Probabilities

As illustrated in Figure 10, the adjusted collapse margin ratio, ACMR, is related to the probability of collapse at the MCE ground motion intensity ( $ACMR = 1.0$ ) by the assumed

lognormal form of the collapse fragility relationship and the variability (dispersion) in the collapse assessment. The ATC 63 procedure provides guidelines on assessing the composite uncertainties as a function of (i) record-to-record uncertainty in the ground motions, (ii) quality of the nonlinear analysis model, (iii) quality of the available test data to calibrate the nonlinear analysis and component limit states, and (iv) comprehensiveness and quality of the design requirements for the system. The composite uncertainties range from a low value of  $\sigma(\ln) = 0.55$  (superior systems and data) to a high value of  $\sigma(\ln) = 1.15$  (for less well-defined systems and data). The low value of 0.55 is largely controlled by the inherent variability in the nonlinear response of structures to randomness in the earthquake ground motions. On the other hand, the added uncertainties above this value are largely a function of the quality and certainty of design/construction provisions and our confidence in predicting the structural behavior and simulating collapse.

Shown in Figure 12 are plots of how the probability of collapse at the MCE relates to the ACMR as a function of the dispersion. Based on benchmark studies of buildings designed per current building codes, the ATC 63 method is targeting maximum permitted MCE collapse probabilities of 10% to 20% (or, conversely, 90% and 80% probabilities of non-collapse). For systems such as RC SMFs, where the design provisions and collapse assessment methods are well established, the dispersion is  $\sigma(\ln) = 0.65$ . Accordingly, the minimum permissible ACMRs are 2.3 and 1.73, respectively, to meet the MCE collapse probabilities of 10% and 20%. In other words, the *median* collapse probabilities (as determined by the nonlinear analysis and adjusted using the SSF factor) need to be at least 2.3 and 1.73 times the intensity of the MCE hazard. The plots in Figure 12 show how reducing the uncertainty to  $\sigma(\ln) = 0.55$  will reduce the minimum required ACMRs to 2.0 and 1.6. At the other extreme, the larger uncertainty of  $\sigma(\ln) = 1.15$  would increase the minimum required margins to ACMRs of 4.3 and 2.6.



**Figure 12 – Relationship Between Collapse Probability at MCE to the ACMR as a Function of Dispersion in Collapse Fragility**

## Evaluation of Acceptance Criteria

The collapse fragility curve (c) of Figure 10 is the best estimate of the collapse safety for the four-story RC SMF example. In this case, the probability of collapse at the MCE intensity is roughly 2%. Measured relative to the MCE intensity, the fragility curve can be described by the ACMR ( $=4$ ) and the dispersion (uncertainty) of  $\sigma(\ln(Sa))$  ( $= 0.65$ ). Thus, to achieve a target failure probability at the MCE, one can specify a combination of a minimum ACMR for a given dispersion. The ATC-63 provisions adopt this as a strategy, where the acceptance criteria are specified in terms of specified limits on the ACMR and the median collapse point is calculated from the nonlinear analysis IDA procedure, including the SSF adjustment.

Referring back to Figure 1, the system acceptance criteria are evaluated in the context of a series of assessments of archetypical structures, which capture the salient seismic design aspects of the proposed system. Each of the archetype models are subjected to a collapse assessment, along the lines outlined previously for the four-story RC SMF frame. The key features in the development of the fragility curve for a given single archetype model are as follows:

1. Develop a nonlinear analysis model that captures the median response of the structural archetype, including all significant strength and stiffness deterioration that can lead to collapse.
2. Perform IDA analyses to determine the median collapse capacity and collapse margin ratios of the structural archetype for the ATC 63 specified set of ground motions, i.e., point  $\hat{S}_{CT}$  in Figures 6 and 8 and CMR in Figure 10. The median is simply the spectral intensity at which point half of the ground motions cause collapse. Potential failure modes that are not directly simulated in the analysis should be incorporated in the median collapse assessment by supplemental checks on component limit states (see Figure 9).
3. Adjust the CMR, as calculated by the IDA of the ATC 63 records, by the spectral shape adjustment to determine the ACMR (Figures 10 and 11). The ATC 63 procedure specifies the SSF adjustment factor as a function of the system ductility and the primary seismic hazard region for which the systems are intended.
4. Determine the composite uncertainty in the collapse assessment. The ATC 63 report outlines procedures to evaluate each of these constituent components of the uncertainty. As shown in Figure 12, the range of composite uncertainty can range from a low of 0.55

(superior systems and data) to a high of 1.15 (for less well-defined systems and data).

5. Finally, given the composite uncertainty, the ATC 63 procedures specify lower limits on the ACMR that are predicated on limiting the probability of collapse for the MCE spectral intensity. Lower limits on the ACMR are specified both for averages from the entire archetype study and for any individual archetype model. The limits are loosely based on calculated collapse probabilities for special moment frame systems that are permitted in current design practice and considered to provide acceptable safety. The procedures are calibrated to limit the probability of collapse under MCE input motions to an average of 10% for the entire archetype group and 20% for any single archetype model.

## RC SMF Archetype Study

The ATC 63 collapse safety assessment methodology is applied to RC SMF systems with the goal to both illustrate the method and to determine the expected performance of buildings designed per current building code requirements of ASCE 7 (2005) and ACI-318 (2005). The frames were designed using a seismic response factor of  $R = 8$  and met all the minimum strength, stiffness (deflections), and capacity and ductile detailing design requirements.

Summarized in Table 1 are key parameters and assessment results of the twenty-two RC SMF archetypes that were designed and evaluated to represent the typical range of design parameters encountered in practice. All of the archetypes are based on the three-bay multistory frame configuration shown in Figure 2. The primary design variables used to articulate the archetype design space are building height, perimeter- versus space-frame configurations, framing bay width, Seismic Design Category D and C, and alternative minimum base shear requirements. The frames were designed assuming a typical office building occupancy with an 8-inch thick flat slab gravity system. The perimeter frame configurations are designed for a tributary seismic mass floor area of about 6,750 sq. ft. per frame at each floor and a tributary gravity area of 1,080 sq.ft. (a ratio of tributary gravity to seismic mass of 0.16). The space frames had equal seismic and tributary floor areas of 2,700 sq.ft. For further details of the frame designs see ATC 63 (2007) and Haselton and Deierlein (2007).

For purposes of establishing acceptance limits, ATC-63 provides guidelines on subdividing the archetype design parameter space into appropriate subsets. Referring to Table 1, in the RC SMF example, the primary groupings were distinguished based on perimeter and space-frame configurations (Sets 1 and 2 in Table 1) with variations in

building height (1 to 20 stories) within each set. Select additional archetypes were investigated to evaluate the trend with respect to seismic design category (Set 3) and framing bay spacing (Set 4). All of the frames in Sets 1-4 were designed according to ASCE 7-05. Frames in Set 5 were re-designed to investigate the effect of alternative minimum base shear strength requirements of ASCE 7-02.

The following is a summary of key variables of Table 1:

T	natural period used to determine the minimum base shear according to the upper limit values permitted by the ASCE 7
V/W	design base shear coefficient
S <sub>MT</sub>	spectral acceleration corresponding to MCE ground motions at the design period, T.
Ω <sub>o</sub>	static overstrength calculated by a static nonlinear

pushover analysis using expected material and component strengths

S<sub>CT</sub> spectral collapse capacity of frame as measured by the average spectral acceleration of the ATC-63 ground motion record set as scaled at the design period T to the point where half of the records cause collapse of the frame (see Fig. 6)

CMR Collapse Margin Ratio, calculated as the ratio of the median spectral collapse capacity to the spectral demand, S<sub>CT</sub>/S<sub>MT</sub>. (see Fig. 10)

SSF Spectral Shape Factor multiplier to account for the spectral shape of extreme ground motions

ACMR Adjusted Collapse Margin Ratio, calculated as the product of CMR\*SSF (see Fig. 10).

To further explain these values, consider the first entry (archetype design ID #2069) in Table 1, which is a one-story

**Table 1 - Summary of RC SMF Archetype Parameters and Assessment Results**

Design ID Number	No. of Stories	Archetype Design Parameters						Analysis Results		Collapse Margin Ratios			Acceptance Check	
		Framing/ Gravity Loads	Seismic Design Criteria											
			SDC	R	T(sec.)	V/W (g)	S <sub>MT</sub> [T] (g)	Static Ω <sub>o</sub>	S <sub>CT</sub> [T] (g)	CMR	SSF	ACMR	Acceptable ACMR	Pass /Fail
Set 1: High Seismic and Low Gravity (Perimeter Frame) Designs, 20' Bay Width														
2069	1	P	D	8	0.26	0.125	1.50	1.6	1.77	1.18	1.6	1.89	1.73	Pass
2064	2	P	D	8	0.45	0.125	1.50	1.8	2.25	1.50	1.6	2.40	1.73	Pass
1003	4	P	D	8	0.81	0.092	1.11	1.6	1.79	1.61	1.6	2.58	1.73	Pass
1011	8	P	D	8	1.49	0.050	0.60	1.6	0.76	1.25	1.6	2.00	1.73	Pass
5013	12	P	D	8	2.13	0.035	0.42	1.7	0.46	1.10	1.6	1.75	1.73	Pass
5020	20	P	D	8	3.36	0.022	0.27	2.6	0.20	0.74	1.6	1.19	1.73	Fail
Mean:	-	-	-	-	-	-	-	1.8	-	-	-	1.97	2.30	Fail
Set 2: High Seismic and High Gravity (Space Frame) Designs, 20' Bay Width														
2061	1	S	D	8	0.26	0.125	1.50	4	2.94	1.96	1.6	3.14	1.73	Pass
1001	2	S	D	8	0.45	0.125	1.50	3.5	3.09	2.06	1.6	3.30	1.73	Pass
1008	4	S	D	8	0.81	0.092	1.11	2.7	1.97	1.78	1.6	2.85	1.73	Pass
1012	8	S	D	8	1.49	0.050	0.60	2.3	0.98	1.63	1.6	2.61	1.73	Pass
5014	12	S	D	8	2.13	0.035	0.42	2.8	0.66	1.57	1.6	2.51	1.73	Pass
5021	20	S	D	8	3.36	0.022	0.27	3.5	0.33	1.22	1.6	1.96	1.73	Pass
Mean:	-	-	-	-	-	-	-	3.1	-	-	-	2.73	2.30	Pass
Set 3: Low Seismic Designs, 20' Bay Width														
6011	8	P	B/C	8	1.49	0.013	0.16	1.8	0.34	2.12	1.2	2.54	1.73	Pass
6013	12	P	B/C	8	2.13	0.010	0.11	1.8	0.23	2	1.2	2.40	1.73	Pass
6020	20	S	B/C	8	3.36	0.010	0.07	1.8	0.26	3.7	1.2	4.44	1.73	Pass
6021	20	S	B/C	8	3.36	0.010	0.07	3.4	0.12	1.73	1.2	2.08	1.73	Pass
Set 4: 30' Bay Width														
1009	4	P	D	8	0.81	0.092	1.11	1.6	2.20	1.98	1.6	3.17	1.73	Pass
1010	4	S	D	8	0.81	0.092	1.11	3.3	2.77	2.5	1.6	4.00	1.73	Pass
Set 5: ASCE 7-02 Designs of 12 and 20 Story From Sets 1 and 2														
5013-R	12	P	D	8	2.13	0.044	0.42	1.7	0.61	1.45	1.6	2.32	1.73	Pass
5020-R	20	P	D	8	3.36	0.044	0.27	1.6	0.45	1.66	1.6	2.66	1.73	Pass
5014-R	12	S	D	8	2.13	0.044	0.42	2.1	0.67	1.59	1.6	2.54	1.73	Pass
5021-R	20	S	D	8	3.36	0.044	0.27	2	0.53	1.98	1.6	3.17	1.73	Pass

perimeter frame system designed for Seismic Design Category D. The frame has a period of  $T = 0.26 \text{ sec}$  and design base shear coefficient of  $V/W = 0.125g$ , based on the MCE spectral value of  $S_{MT}=1.5g$ , the 2/3 adjustment to the design intensity and  $R = 8$ . From the nonlinear pushover analysis, the frame has a static over strength of  $\Omega_o=1.6$ , which is in line with expectations given the typical material overstrength of about 25%, modest redundancy, discrete member sizing, and light gravity loads on the perimeter frame. The median sidesway collapse capacity of  $S_{CT}=1.77g$ , as determined from the nonlinear time history analyses using the ATC-63 ground motion record set, is 1.18 times larger than the MCE demand of  $S_{MT}=1.5g$ . The resulting capacity/demand ratio ( $CMR = 1.18$ ) is then multiplied by the spectral shape factor ( $SSF = 1.6$  for ductile frames in regions where the MCE ground motion is dominated by +ε records) to determine the adjusted ratio of  $ACMR = 1.89$ .

Referring back to Figures 10 and 12, the ACMR ratio is related to the probability of collapse at the MCE ground motion intensity (equivalent to an  $ACMR = 1$ ) based on the assumed lognormal fragility curve (cumulative distribution function) and the uncertainty in the fragility curve. For the RC SMF systems, the dispersion is equal to  $\sigma(\ln) = 0.65$ , as determined by the ATC 63 guidelines considering the variability in the collapse assessment process and quality of the design provisions and supporting test data. From the relationships plotted in Figure 12, the  $ACMR = 1.89$  of archetype design ID 2069 corresponds to a probability of collapse of 16% at the MCE ground motion intensity.

The last two columns of Table 1 compare the calculated ACMRs to the acceptance limits of 1.73 and 2.30. For systems with  $\sigma(\ln) = 0.65$ , the limit of 1.73 corresponds to an MCE collapse probability of 20%, which is the maximum permitted value for any individual archetype design. The limit of 2.30 corresponds to the MCE collapse probability of 10%, which is the maximum permitted value for the average of representative sub-sets of buildings. In the RC SMF example, the representative archetype subsets are the perimeter and space frame sets 1 and 2.

As indicated by comparing the ACMR to the acceptable ACMR (the 2<sup>nd</sup>- and 3<sup>rd</sup>- columns from the left in Table 1), all of the individual frames in Sets 1 and 2 pass the minimum acceptance criteria except for archetype ID #5020 (the 20-story perimeter frame). Due to low values in several frames, the set of perimeter frames also fails to pass the minimum average ACMR value of 2.3. Thus, according to the criteria, the RC SMF frame Set 1 reveals a deficiency in the design provisions for RC SMF systems. Checks of the acceptable ACMR values for frames designed for a lower SDC B/C (Set 3) indicate that these are generally less critical than those in SDC D, and thus the checks for SDC D will control the design. Similarly, comparisons between the frames in Set 3

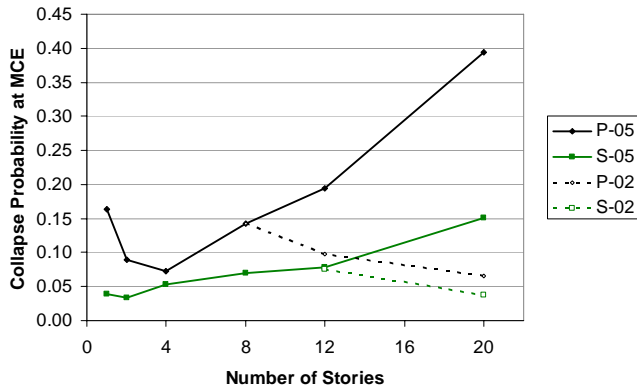
(with 30 foot framing spans) indicate that they are less critical than the corresponding frames in Sets 1 and 2 with 20 foot framing spans.

Differences in safety (as judged by the ACMR values) of the archetype frames can result from a variety of factors that influence the design base shear capacity and ultimately the seismic resistance of the systems. The seismic response factor (or R-factor) assumed in design is one important component of the seismic design provisions, but there are other factors that may be equally important. For example, consider the 4-story perimeter frame (ID #1003), which has a rather high  $ACMR = 2.58$ , in comparison to the 20-story perimeter frame (ID #5020), which has a low  $ACMR = 1.19$ . Interestingly, this trend in ACMR and collapse capacity is opposite to the trend in static overstrength, where the  $\Omega_o = 1.6$  for the 4-story frame versus  $\Omega_o = 2.6$  for the 20-story frame. It turns out that one major difference between the two lies in the design base shear, i.e.,  $V/W = 0.092$  for the 4-story frame versus only  $V/W = 0.022$  for the 20-story frame.

The significance of the low base shear (2.2%) in the 20-story frame is most apparent by contrasting the performance with an alternative design (archetype ID 5020-R in Set 5), which was redesigned to meet the minimum base shear requirement of the 2002 edition of ASCE 7. The 2002 ASCE 7 included a minimum base shear strength requirement, which for long period (tall) structures, requires a larger base shear than that one would otherwise calculate based on the MCE hazard. For the 20-story building (with a design period of  $T=3.36 \text{ sec}$ ), the minimum ASCE 7-02 base shear of  $V/W = 0.044$  is twice that required by value required by ASCE 7-05. The large difference is due to the elimination of the minimum base shear requirement in the 2005 edition of ASCE 7-05. [Note – based on this ATC 63 project finding, the ASCE 7 committee has recently issued an addendum to reinstitute the minimum base shear requirement of the previous 2002 edition].

Comparing results for the 20-story perimeter frames with the alternative base shear strengths (ID #5020 and #5020-R), the increase in design base shear from 0.022 to 0.044 increases the ACMR from 1.19 to 2.66. Note that the change in the base shear requirement actually reduces the calculated static overstrength from  $\Omega_o = 2.6$  to  $\Omega_o = 1.6$ , thus demonstrating how the static overstrength can be a misleading indicator of collapse safety.

The relative safety of the perimeter and space frames, including the effect of the minimum base shear requirement for long-period buildings, is compared in Figure 13. The MCE collapse probabilities (on the vertical axis) are directly related to the ACMRs through the relationship shown previously in Figure 12. Quite apparent from Figure 13 is the consistent difference in collapse safety between the perimeter



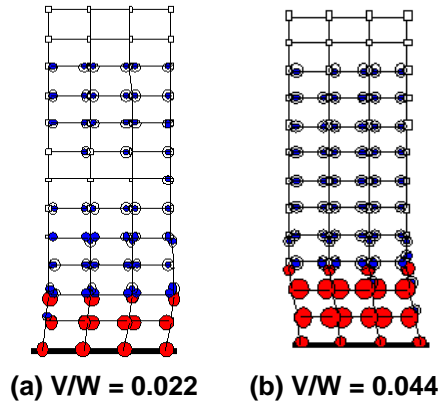
**Figure 13 – Conditional MCE Collapse Probabilities for Perimeter and Space Frame Buildings Designed using Alternative Minimum Base Shear Coefficients of ASCE 7-05 and ASCE 7-02**

and space frame systems, where on average the space frames have about half the collapse risk. Further, the plot shows the dramatic reduction in collapse risk for frames over 8 to 12 stories tall when the minimum base shear strength requirement of ASCE 7-02 is applied in the design.

While some of the differences in collapse risk between different buildings are the result of building-specific aspects of the individual archetype designs, the trends between perimeter versus space frames and minimum base shear requirements are clearly tied to underlying features in design and behavior. For example, shown in Figure 14 is a comparison of the representative inelastic deformations at the onset of collapse in the two 20-story frames, designed with different base shear strengths. The plots clearly show how the larger base shear strength results in greater distribution of inelastic effects and energy dissipation in the frame. Thus, the effect of the base shear is not simply to increase the frame strength, but rather to affect the distribution of inelastic deformations and thereby increase the system ductility. One can show, for example, that the drift displacement capacity of ductile systems is highly dependent on the base shear capacity through destabilizing  $P-\Delta$  effects. Further comparisons of this sort for other design parameters and provisions (e.g., space versus perimeter frame, strong-column weak-beam capacity design, etc.) are examined by Haselton and Deierlein (2007).

### Concluding Remarks

This paper describes the results of ongoing efforts to develop more consistent and scientifically-based methods to assess the collapse safety of buildings with the ultimate aim toward improving seismic design standards and practice. The methods are enabled by research to improve understanding of



**Figure 14 – Comparison of Inelastic Deformations in 20-Story SMF Designed for Different Base Shears**

ground motions and their effects on structural response, nonlinear behavior and computer response simulation of structures, and practical probabilistic approaches to account for the inherent uncertainties in design and analysis. The ATC 63 project has implemented these advancements in a practical framework to assess the collapse safety of buildings and their underlying design basis. While the immediate focus of the ATC 63 project is on assessing building system response factors for seismic building code provisions, the method and concepts can readily be applied more broadly for the performance assessment and design.

Structural design for seismic collapse safety requires structures sufficient strength, stiffness, and robustness (ductility and ability to distribute inelastic effects). While straightforward in concept, it is not always clear how these attributes are provided for in modern building code provisions. For example, whereas seismic response factors, i.e., R-factor, are prominent in the building code and subject to considerable debate and scrutiny, the R-factors are just one of many contributors to the structural strength requirements of current codes. As shown for long period buildings, minimum base shear requirements can have an equally if not more significant effect on design, as can other factors such as (i) differences between theoretical and code-specified periods for determining the design base shear, (ii) limits on deflections and how they are calculated and enforced, and (iii) minimum member sizes as required by capacity design requirements or other constraints. It is for these reasons that the concepts of the “building archetype” and the design/assessment of multiple archetype realizations are central aspects of the ATC 63 assessment procedure.

While the proposed procedures seek to minimize reliance on “tradition”, “past-practice” and “engineering intuition”

through the application of rigorous scientific analysis, they nonetheless still heavily depend on informed engineering judgment and interpretation. This is most apparent in the large effect that uncertainties have in the assessment criteria (e.g., see Figure 12) and the role that judgment plays in quantifying uncertainties arising due to the application of building code requirements in design and construction practice. Large uncertainties also occur in assessing the extent to which mathematical analysis models realistically capture the collapse behavior in actual buildings. In this regard, it is important to emphasize that while the procedures give the impression of providing precise collapse probabilities, their larger value is to promote more consistency in comparing the relative safety between alternative systems and the effectiveness of various design provisions. For example, the proposed method has already demonstrated its value in helping to resolve questions concerning the effectiveness and need for minimum base shear strength requirements for long period structures (i.e., tall buildings), where the force requirements otherwise calculated by spectral acceleration hazards are quite low.

Finally, as the ATC 63 project is still ongoing, some aspects of the methodology are subject to change and may ultimately differ from the information presented herein. Therefore, readers are cautioned that the information presented herein may differ and not fully reflect the implementation as will be reported in the final ATC 63 report.

## Acknowledgements

This work reported in this paper has been supported in large part by the Applied Technology Council through funding of the ATC-63 project by the Department of Homeland Security's Federal Emergency Management Agency and through related research of the Pacific Earthquake Engineering Research (PEER) Center of the National Science Foundation (award number EEC-9701568).

The ATC-63 project is led by the Project Management Committee consisting of C. Kircher (Technical Director), M. Constantinou, G. Deierlein, J. Harris, and A. Porush; the ATC Director of Projects, J. Heintz; the ATC Project Technical Monitor, W. Holmes; the FEMA Technical Monitor, R. Hanson; the ATC Executive Director, C. Rojahn; and the FEMA Project Officer, M. Mahoney. Other members of the project team include S. Cranford, A. Filiatrault, H. Krawinkler, K. Cobeen, C. Haselton, A. Liel, J. Christovasilis, J. Chou, B. Dean, K. Haas, J. Takagi, A. Wanitkorkul, and F. Zareian. Contributions of ATC 63 Project Review Panel, chaired by M. Phipps, and the ATC staff are gratefully acknowledged. The comments and conclusions made in this paper are those of the authors and

do not necessarily represent those of the project sponsors or other ATC 63 team members.

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