PREDICTING COLLAPSE OF FRAME AND WALL STRUCTURES

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ABSTRACT :
This paper summarizes the state of knowledge in structural system modeling and tool development for predicting the collapse potential of buildings whose lateral load resisting system consists of moment-resisting frames or reinforced-concrete shear walls. In this context, collapse implies dynamic instability in a sidesway mode, triggered by large story drifts that are amplified by structure P-Δ effects and deterioration in strength and stiffness of the components of the system. The collapse capacity of a building is defined as the maximum ground motion intensity (often represented by the spectral acceleration at the first mode period) at which the structural system still maintains dynamic stability. A collapse fragility curve that incorporates aleatory dispersion due to record-to-record variability is obtained by ordering the collapse capacities for a representative set of ground motions. Additional dispersion of the collapse fragility curve is caused by epistemic uncertainties due to modeling assumptions on which analytical predictions are based. The collapse potential of a building can be expressed as the probability of collapse at a discrete hazard level or the mean annual frequency of collapse, both of which can be computed from the collapse fragility curve and the hazard curve for the site of the structure. Collapse fragility curves and estimates of the collapse potential for generic structures are discussed, together with the effects of different levels of detailing in modeling for nonlinear analysis. Results for generic structures are compared to assess the merits of different structural systems for controlling the collapse potential.

KEYWORDS:
Structural Collapse, Moment-resisting Frames, Shear Walls, Component Deterioration, Performance-based Design

1. INTRODUCTION

Building collapse in earthquakes has a dominant impact on casualties and loss of life and could be a major contributor to monetary losses and downtime losses. Current practice in structural design and codes associates collapse safety with an acceptable story drift or a component plastic deformation “capacity”, usually at plastic hinge regions. Such an approach ignores the ability of the structural system to redistribute damage after formation of few plastic hinges. This paper is a summative compilation of research conducted by the authors in which a methodology for reliability-based assessment of collapse potential and for design for collapse safety is developed. The proposed methodology takes advantage of recent advancements in structural components modeling to incorporate stiffness and strength deterioration, and in identifying the propagation of variability from ground motion and building modeling to collapse potential of building.

2. STRUCTURAL COMPONENT MODELS

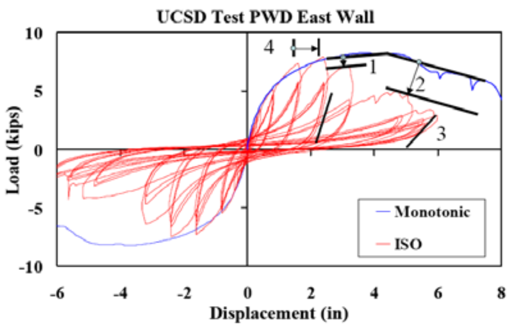
A fundamental building block of any structural model used in a seismic demand study that addresses structural collapse is a structural component model that incorporates stiffness and strength deterioration. Several studies in the past have addressed such phenomena (Hisada et al, 1962, Clough and Johnson, 1966; Mahin and Bertero, 1975; Takeda et al., 1970) in which stiffness and strength of a structural component is updated in each excursion according to the maximum deformation experienced in previous excursions. Foliente (1995) presented a summary of the widely known Bouc-Wen model and its modifications to account for component deterioration.

Others (Naeim et al., 1995, Sivaselvan and Reinhorn, 2000) modified the Bouc model into a versatile smooth hysteretic model with stiffness and strength degradation and pinching characteristics. Rahnama and Krawinkler (1993) model component deterioration rate in excursion *i*, β_i, as a function of the energy dissipated in that cycle, E_i, the sum of hysteretic energy dissipated in all excursions prior to excursion *i*, a reference hysteretic energy dissipation for the component, E_t, and an exponent parameter, *c*, that defines the rate of deterioration (Eqn. 2.1). The value of stiffness and strength (denoted generically as S_i) in excursion *i* is obtained as $S_i = (1-\beta_i)S_{i-1}$. Value of *c* and E_t are calibrated using experimental results.

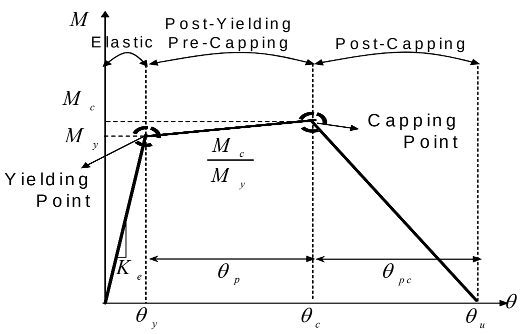
$$\beta_i = \left[E_i \Big/ \left(E_t - \sum_{j=1}^{i-1} E_j \right) \right]^c \tag{2.1}$$

Ibarra et al. (2005) extended this deterioration model and assumed that deterioration in structural components occurs in two modes: monotonic deterioration and cyclic deterioration. Monotonic deterioration is exemplified in the component backbone curve, and cyclic deterioration is accounted for by using a deterioration criterion similar to the model developed by Rahnama and Krawinkler. The characteristics of this phenomenological model, which is the basis of the deterioration model used in this study, are briefly explained next.

Figure 1a illustrates a monotonic load-displacement response and a superimposed quasi-static cyclic response of two identical plywood shear wall panels (Gatto and Uang, 2002). The monotonic test result shows that strength is “capped” and followed by a negative tangent stiffness. Ideally, this monotonic backbone curve can be divided into three regions: (1) elastic, (2) post-yielding pre-capping and (3) post-capping, as shown in Figure 1b. The backbone curve in Figure 1b is depicted in the form of moment-rotation but the shape of the curve is valid for any force-deformation response. The elastic region of the backbone curve is defined by the component stiffness, K_e, and yield moment M_y. In the post-yielding pre-capping region the rate of increase in moment is much smaller than in the elastic region. The extent of the post-yielding pre-capping region is defined by the post yielding deformation capacity θ_p (denoted as plastic hinge rotation capacity) and capping strength M_c, both under monotonic loading. After passing the Capping Point, the loading path enters the post-capping region where the component softens under monotonic loading. The deformation at which the strength drops to zero (this is a theoretical value that may never be attained because of residual strength) is denoted as θ_u, and the increase in deformation from θ_c to θ_u is denoted as post-capping deformation capacity θ_{pc}.



(a)



(b)

Figure 1: Structural component model that incorporates monotonic and cyclic deterioration: a) a typical experimental test result, b) component backbone curve definition.

The hysteretic model used in this study follows the rules of the peak-oriented model initially proposed by Clough and Johnson (1966). Other hysteretic models (e.g., pinching and bilinear) are implemented in the model but are not considered in the results presented here. It was shown by Medina and Krawinkler (2003) and Ibarra and Krawinkler (2005) that the collapse potential of structural systems is not very sensitive to component hysteresis behaviour, except for a pinching hysteretic model with severe stiffness degradation.

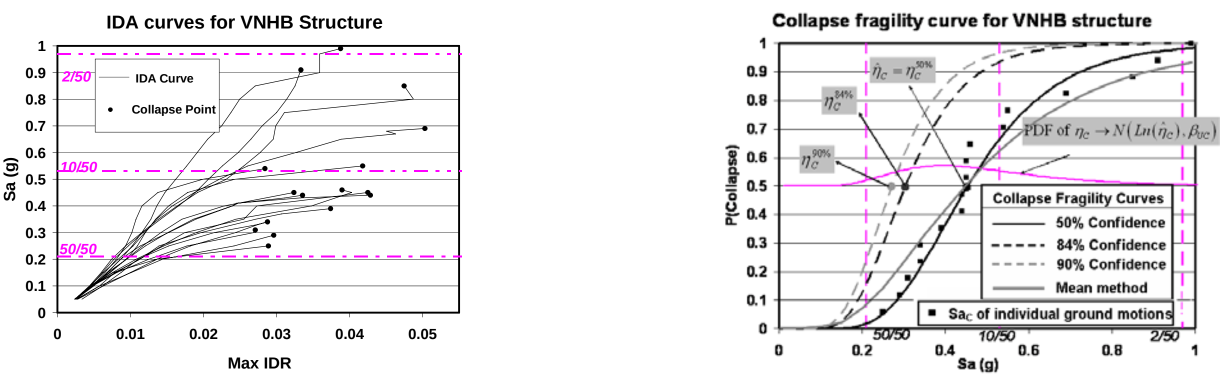
Cyclic deterioration is accounted for by using energy dissipation as a deterioration criterion as shown in Eqn. 2.1. Based on observations made from component cyclic test results (see Figure 1a), four modes of deterioration are identified: 1) basic strength, 2) post-capping strength, 3) unloading stiffness, and 4) accelerated reloading stiffness deterioration (Ibarra et. al., 2005). Basic strength and post-capping deterioration move the strain hardening and post-capping branch toward the origin, respectively. Unloading stiffness deterioration flattens the unloading stiffness after each reversal, whereas accelerated reloading stiffness deterioration increases the target maximum displacement. Deterioration parameters are calibrated to mimic the results obtained from experiments in which it is assumed that $E_t = \lambda M_y \theta_p$, where λ is a parameter that is calibrated using experimental results. In a modified version of the Ibarra-Krawinkler model (Lignos, 2008) the rate of cyclic deterioration is expressed by means of a reference cumulative plastic rotation capacity using a term $\Lambda = \theta_p.\lambda$, which can be easily visualized.

3. COLLAPSE FRAGILITY CURVES

Most current codes and guidelines today use a structural response parameter as an indicator for building collapse. Such an approach is inefficient and relatively inaccurate. It ignores the fact that the structural system can redistribute seismic demands once components pass the arbitrary deformation limit set by the code. Estimation of any structural response parameter near collapse is very sensitive to factors such as: assumptions made in developing the structural model, time step used in the structural analysis, and estimate of the ground motion intensity. A small variation in any of these parameters can change the estimates of building response drastically. We define “collapse capacity” of a given building as the ground motion intensity at which the building experiences dynamic lateral instability. This instability is due to P-Δ effects in one or combination of stories, which may be greatly accelerated by strength and stiffness deterioration in structural components.

In the study summarized here we use Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell, 2002) to obtain the collapse capacity for an individual ground motion record. In an IDA the intensity of the ground motion is incremented and applied to the structural model up to the point at which dynamic lateral instability occurs. We use spectral acceleration at the fundamental period of the building, S_a as the ground motion intensity measure. IDAs are performed for a number of representative ground motions that are likely to occur at the location of the building. The value of S_a at which dynamic instability occurs is denoted as S_{ac}.

Figure 2a shows IDA curves obtained by performing IDAs using a modified version of Drain-2DX (Prakash et. al, 1993) for a model of an old reinforced-concrete building (denoted as VNHB) using 17 ground motions (Zareian et. al., 2006). The building is located in Van Nuys, California, at 34.221° north latitude and 118.471° longitude, and the 17 ground motions used in the simulations are representative of the seismicity of this location (Krawinkler ed, 2005). Each IDA curve shows the relation between the maximum interstory drift ratio (Max IDR) and S_a at T = 1.5 sec. The solid circle at the end of each IDA curve shows the last point at which analytical convergence is obtained. The projection of each solid circle on the vertical axis represents the collapse capacity of this model for the individual record. The cumulative distribution function, assuming a lognormal distribution, of the S_{ac} values is defined as the “collapse fragility curve”. The collapse fragility curve for the VNHB building is shown in Figure 2b with a solid black line.



(a) (b)

Figure 2: Development of collapse fragility curve for VNHB with T = 1.5 sec.: a) IDAs of the structural model, b) collapse fragility curves with various representations of uncertainty (after Zareian et. al., 2006)

Loss of lateral dynamic stability is the only mode of collapse considered in this study. Other modes of collapse, such as loss of vertical load carrying capacity or cascading (progressive) collapse, can be incorporated in the estimation of the collapse fragility curve directly and indirectly. In the direct method, criteria associated with other modes of collapse need to be integrated in the structural model and the nonlinear analysis program used to perform IDA. Development of such criteria, and structural models and structural analysis programs that can utilize directly all possible collapse modes are the subject of current research (Mosalam and Talaat, 2007). In the indirect method, probabilistic representation of individual modes of collapse are developed using simulation or empirical techniques and are integrated with a Bayesian approach to reach the overall probability of collapse (Krawinkler ed., 2005).

Variability in collapse capacity can have various sources. We assume that these sources can be categorized into two types: *aleatory* and *epistemic*. In this approach we assume that the variability in collapse capacity due to unknown nature of the ground motion can be characterized to be random (*aleatory*). We further assume that all other sources of variability in estimation of collapse capacity are due to our lack of knowledge in developing a "real" model of the structure and, therefore, are characterized to be *epistemic*. Simultaneous consideration of effects of both types of variability in the estimation of collapse capacity necessitates an elaborate Monte Carlo simulation in which the probabilistic representation of each structural component in the structure model, plus a number of ground motions that are representative for the seismicity of the building location, are needed.

Two approximate methods for combining the effects of the two types of variability in the estimation of collapse capacity of a building can be utilized: "confidence level" method and "mean" method. In both methods we assume that the sources of variability of *epistemic* and *aleatory* nature are independent. The Bayesian approach is used to combine these types of variability to obtain the total variability in the estimation of collapse capacity (Cornell et. al. 2002). In both methods a model of the building with component properties set to their median values and a number of representative ground motion records are used to develop a median estimate of the collapse fragility curve. The dispersion in the median estimate fragility curve, β_{RC}, is assumed to be solely *aleatory*. In Figure 2b the fragility curve drawn with a black solid line is the collapse fragility curve obtained in this fashion for VNHB ($\hat{\eta}_C = 0.45g, \beta_{RC} = 0.37$). Then it is assumed that the median of the median estimate collapse fragility curve is by itself a random variable with a lognormal distribution whose median is $\hat{\eta}_C$ and its dispersion, β_{UC}, is solely *epistemic* and independent of the *aleatory* dispersion, β_{RC}.

We illustrate the "confidence level" method and "mean" method with aid of Figure 2b. In the "confidence level" method we assume that the "real" collapse fragility curve will have a dispersion equal to β_{RC} (*aleatory*), however, its median value is variable. We define Y as the probability that the "real" median value of collapse capacity is greater than $\hat{\eta}_C$ and is equal to η_C^Y. The fragility curve drawn with black dashed line in Figure 2b is associated with 84% confidence and assuming β_{UC} = 0.4 (Ibarra and Krawinkler, 2005.) In other words, the probability that the "real" median value of collapse capacity is more than the median value of the adjusted collapse fragility, $\eta_C^{84\%}$, is 84%. As the confidence level increases, for a given value of β_{UC}, the estimate of median of collapse capacity decreases (e.g., the fragility curve drawn with gray dashed line is associated with 90% confidence.) In the "mean" method, it is assumed that $\hat{\eta}_C$ is the median estimate of the median value of collapse capacity and the dispersion of the collapse fragility curve is inflated to represent both sources of variability. In the mean sense, this combined dispersion is equal to $\beta_C = \sqrt{\beta_{RC}^2 + \beta_{UC}^2}$. Figure 2b shows such a collapse fragility curve with a solid gray line ($\beta_C = \sqrt{0.37^2 + 0.40^2} = 0.54$).

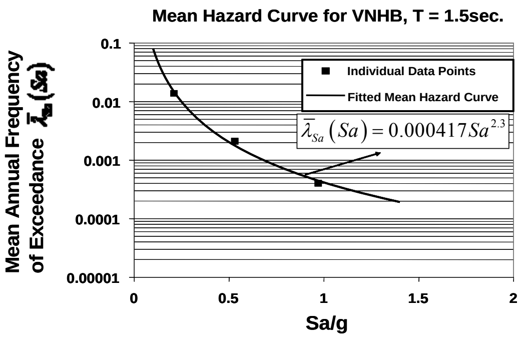
4. ASSESSMENT OF COLLAPSE POTENTIAL OF BUILDINGS

A meaningful assessment of the collapse potential of a building is obtained by combining the collapse fragility curve with a description of ground motion hazard at the location of the building. Two representations of collapse potential of buildings are addressed in this study: probability of collapse at discrete hazard levels, and mean annual frequency (MAF) of collapse.

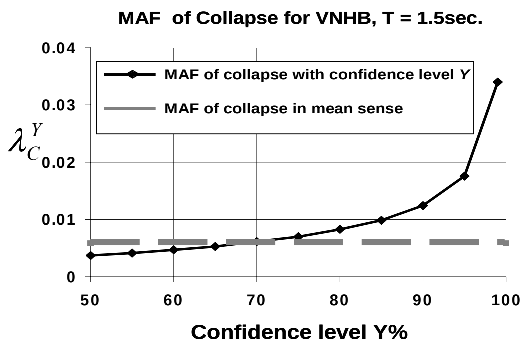
The two representations of collapse potential of buildings adhere to two dependent relations: the relation between the seismic hazard and ground motion intensity measure (i.e. seismic hazard curve), and the relation between the ground motion intensity measure and probability of collapse (i.e. collapse fragility curve). The seismic hazard curve contains the return period dependent description of the ground motion intensity. The variability in estimation of the hazard for given ground motion intensity can also be of *aleatory* and *epistemic* nature. It has been proposed by Cornell et. al (2002) that by using the *mean* hazard curve, denoted as $\bar{\lambda}_{Sa}(Sa)$, one can assume that both types of variability in hazard estimation have been considered.

Figure 3a shows the mean hazard curve for the period of 1.5 seconds at the location of the VNHB (Zareian et. al., 2006). The three data points correspond to three hazard levels with 50%, 10%, and 2% probability of exceedance, respectively, in 50 years (denoted as 50/50 level, 10/50 level, and 2/50 level). The hazard curve is obtained here by fitting a straight line in the log-log domain to the data points. The slope of this line is estimated as $k = 2.3$.

Assessing the probability of collapse of a building at a specific hazard level is possible by combining information from the collapse fragility curve (with confidence Y or in the mean sense) and the building's hazard curve. For the VNHB building, S_a at the 50/50, 10/50, and 2/50 hazard levels can be read from Figure 3a and is equal to 0.21g, 0.53g, and 0.97g, respectively. These intensities are marked with vertical gray (magenta in coloured version) dashed lines in Figure 2b. The intersection of these dashed lines with the collapse fragility curves shows the associated probability of collapse. For example, the probability of collapse of VNHB at the 50/50 hazard level is equal to 1.5% if no *epistemic* variability is considered (using the solid black fragility curve). If a confidence of 84% is sought for this estimation, the probability of collapse at the 50/50 hazard level is equal to 15.5% (using the dashed black fragility curve). In contrast, this probability of collapse is equal to 8% in the mean sense (using the solid gray fragility curve). Such large values for probability of collapse are due to rapid deterioration of the strength properties of the structural components in the old reinforced concrete VNHB, and in part due to the large value of *epistemic* variability ($\beta_{UC} = 0.4$).



(a)



(b)

Figure 3: a) Mean hazard curve for the location of VNHB, period of 1.5 seconds, and 5% critical damping (Zareian et. al., 2006), b) MAF of collapse obtained through numerical integration of collapse fragility curves and hazard curve, from "confidence level" method and "mean" method.

In order to estimate the MAF of collapse with confidence level Y or in mean sense, one needs to integrate the appropriate collapse fragility curve over the mean hazard curve. Eqn. 4.1 shows this integration in mathematical format, and Figure 3b shows the results of the integration. In this figure, each black diamond represents a pair of confidence level Y and associated MAF of collapse, and is obtained by integrating the collapse fragility curve with Y confidence (Figure 2b) over the mean hazard curve (Figure 3a). The gray dashed line in Figure 3b represents the MAF of collapse that is obtained by integrating the "mean" collapse fragility curve over the mean hazard curve. Inspection of Figure 3b shows an exponential increase in MAF of collapse for larger confidence levels.

$$\lambda_C^Y = \int_{Sa} P_Y(C \mid sa) \left| d\bar{\lambda}_{Sa}(sa) \right| \tag{4.1}$$

A closed form solution to Eqn. 4.1 has been developed by Jalayer (2003) and is shown in Eqn. 4.2. The expression on the right-hand side of Eqn. 4.1 contains the MAF of the spectral acceleration associated with the 50% probability of collapse, $\bar{\lambda}_{Sa}(\hat{\eta}_C)$, and two terms that account for *aleatory* and *epistemic* variability in the computation of the collapse capacity. K_Y is the standardized Gaussian variate associated with the probability Y of not being exceeded. For instance, the MAF of collapse for VNHB with 84% confidence can be obtained by substituting $\hat{\eta}_C = 0.45g$, $\bar{\lambda}_{Sa}(\hat{\eta}_C) = 0.0026$, $k = 2.3$, $\beta_{RC} = 0.37$, $\beta_{UC} = 0.4$, and $K_Y = 1.0$ into Eqn. 4.2 to obtain $\lambda_C^Y = 0.0094$. This value is comparable to the value obtained from numerical integration, which is equal to 0.0098.

$$\lambda_C^Y = \int_{Sa} P_Y(C \mid sa)\left|d\bar{\lambda}_{Sa}(sa)\right| = \left[\bar{\lambda}_{Sa}(\hat{\eta}_C)\right]\left[\exp\left(0.5k^2\beta_{RC}{}^2\right)\right]\left[\exp\left[K_Y\left(k\beta_{UC}\right)\right]\right] \tag{4.2}$$

5. CONCEPTUAL DESIGN OF BUILDINGS FOR COLLAPSE SAFETY

In contrast to the process of assessing the collapse potential of an existing or already designed building, designing a building for collapse safety involves decision making on its structural system and components. A good design is based on concepts that incorporate performance targets up front. For collapse safety, such performance targets could be expressed as a tolerable probability of collapse at discrete hazard levels or a tolerable MAF of collapse. Both representations can incorporate the two types of variability (*aleatory* and *epistemic*.) The foundation for designing buildings for collapse safety is similar to the assessment of collapse potential of an existing building and requires the development of collapse fragility curve of design alternatives and the hazard curve for the location of the building. Our focus is on selecting a proper combination of structural system and its structural parameters that conforms to a desired collapse performance objective. We assume that the building's hazard curve is dependent on the location of the structure.

A major issue in design for collapse safety is identifying the sensitivity of the collapse fragility curve of feasible structural systems to variations in their structural parameters. Such research has been performed on collapse capacity of generic moment-resisting frames and structural walls (Zareian and Krawinkler, 2007a and 2007b). These generic structures are representative of typical structural systems designed in the past and present, and utilize component hysteresis models that account for monotonic and cyclic deterioration in strength and stiffness. Using generic structures, a database of collapse fragility curves has been developed for various combinations of structural system parameters, using a set of 40 ground motions (denoted as LMSR-N) that represent ordinary ground motions in California (Zareian, 2006). Closed-form equations are derived to estimate the median of the collapse capacity, η_c, and the associated aleatory dispersion, β_{RC}, as a function of structural parameters by utilizing the comprehensive database of collapse fragility curves. Details of the multivariate regression analysis and the criteria for reducing regression models with several predictor variables to simpler models with fewer predictor variables are presented in Zareian (2006). Eqns. 5.1 and 5.2 were developed by utilizing a first-order regression model based on one predictor variable (i.e., $b_{0,MRF}(N, \alpha_T)$ or $b_{0,SW}(N, \alpha_T)$) for the median collapse capacities of generic moment-resisting frames and generic shear walls, respectively. In Eqns. 5.1 and 5.2 N is the number of stories, α_T is the period coefficient ($\alpha_T = T/N$), and γ is the yield base shear coefficient. Due to the small variation of β_{RC} across all structures, it is concluded that $\beta_{RC} = 0{\cdot}4$ for generic moment-resisting frames and $\beta_{RC} = 0{\cdot}5$ for generic shear walls are good estimates for aleatory uncertainty in the collapse capacity of these structural systems. Values of $b_{0,MRF}(N, \alpha_T)$ or $b_{0,SW}(N, \alpha_T)$ as a function of the number of storeys N and period coefficient α_T are shown in Figures 4a, and 4b for moment-resisting frames and shear walls, respectively. The reader is invited to visit the web site http://spee.eng.uci.edu/Ctool/, which presents visual tools for estimating the probability of collapse for the range of generic systems discussed here.

$$\hat{\eta}_c = \exp\left(b_{0,MRF}(N,\alpha_T)\right)\cdot\theta_p^{\,0.32}\cdot\left(\frac{\theta_{pc}}{\theta_p}\right)^{0.08}\cdot\lambda^{0.08}\cdot\gamma^{0.73} \tag{5.1}$$

$$\hat{\eta}_c = \exp\left(b_{0,SW}(N,\alpha_T)\right)\cdot\theta_p^{\,0.66}\cdot\left(\frac{\theta_{pc}}{\theta_p}\right)^{0.17}\cdot\lambda^{0.13}\cdot\gamma^{0.33} \tag{5.2}$$

Evaluation of the aforementioned database shows that for frame structures the collapse fragility curve is sensitive primarily to the combination of base shear strength and period, the column-to-girder strength ratio, and the plastic rotation capacity and post-capping deformation capacity of the structural components. For frame structures whose first mode period is in the constant velocity spectrum range, a base shear strength proportional to a constant S_a/T value (approximating the constant R-factor concept) will lead to much larger collapse probabilities for long period structures. The reason is that for long period frame structures P-delta becomes a predominant issue and relatively larger strength is required to reduce "ratcheting" (drifting) of response and subsequent incremental collapse. The importance of the column-to-beam strength ratio also became very evident. Results of this study have disclosed that increasing this ratio from 1.2 (ACI recommendations) to 2.4 can increase the median of collapse capacity by a factor of about 2. From the component deformation parameters, the plastic rotation capacity and post-capping deformation capacity were found to be more important than the cyclic deterioration characteristics.

Wall structures, when compared to frame structures, show very low probabilities of collapse under the conditions that (1) reasonable plastic rotation capacities are provided (larger than about 1.5-2.0%) and (2) wall shear failure is prevented.

(a) (b)

Figure 4: Values of $b_{0,MRF}(N,\alpha_T)$ as a function of number of stories N and period coefficient α_T for: a) moment-resisting frames, b) shear walls

As a matter of caution, it must be said that the validity of collapse fragility curves depends, sometimes strongly, on the selected intensity measure and the frequency characteristics of the ground motions chosen for dynamic analysis. Recent studies (Baker and Cornell, 2005; Tothong, 2006) have shown that vector intensity measures provide more accurate representations of the ground motion environment than scalars such as S_a. Moreover, if collapse is associated with long return period hazards, as will be the case for new structures, the ground motion selection process should consider the ε-dependence inherent in the hazard analysis (Baker and Cornell, 2005). Using the aforementioned database it can be shown that consideration of ε significantly affects the prediction of η_c for the case study generic moment-resisting frames. It is observed that for low probability hazard levels (rare events), which translates into positive values of ε, the median collapse capacity obtained by ignoring the effect of ε is underestimated considerably. The data show that ignoring the effect of ε in estimation of the median collapse capacity at a hazard level associated with $\varepsilon = 2$ leads to an underestimation of η_c by about 45% (Zareian and Krawinkler, 2007a).

6. CONCLUSION

This paper provides a summary of the authors' research on developing a reliability-based methodology for assessment of collapse potential and for conceptual design for collapse safety of structures. Two measures for gauging the collapse potential of buildings are proposed: probability of collapse at discrete hazard levels, and MAF of collapse. Essential tools for implementation of the proposed methodology are structural component models that can represent monotonic and cyclic deterioration of stiffness and strength, and a mathematical formulation that can efficiently incorporate different sources of variability in the process. Results of this study demonstrate and quantify the importance of P-Δ effects and component deterioration as the main reasons for building collapse. Enhancement in selected structural parameters, such as increasing the column-to-beam strength ratio in moment-resisting frames, can considerably postpone building collapse.

7. ACKNOWLEDGMENT

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