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Keynote Paper

Advances in Collapse Simulation of Reinforced Concrete Frame Buildings under Seismic Loads

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ABSTRACT

Past and current efforts to simulate the collapse response of reinforced concrete components and buildings subjected to seismic loads are summarized. Both detailed continuum models of single components and frame idealizations of building frames are discussed. Particular emphasis is placed on incremental dynamic analysis (IDA) and the development of collapse fragility curves. Some key factors that influence the generation of IDAs and consequently the collapse fragility of a building are highlighted.

1. INTRODUCTION

The partial or complete collapse of reinforced concrete (RC) buildings is not uncommon during major seismic events as evidenced by failures in recent earthquakes. While most of the observed failure occurred in older RC buildings with non-ductile detailing, there are also instances when RC buildings designed to modern code provisions have suffered significant damage. Hence, the quantification of the collapse risk of existing (and new) RC structures in a future major earthquake is of significant importance to stakeholders. A range of research effort has been directed towards assessing the collapse potential of RC buildings and identifying earthquake characteristics and model parameters that influence the seismic response of buildings.

2. SIMPLIFIED APPROACHES

Early studies employed single-degree-of-freedom (SDOF) idealizations to investigate collapse and dynamic instability. For example, Takizawa and Jennings (1980) used equivalent SDOF models to estimate the ultimate capacity of ductile RC Frames and concluded that strong motion duration and period of the system (stiff vs. flexible structures) played a role in the imposed ductility demands. Later studies by other researchers have contradicted some of these findings indicating that the differences in the conclusions are likely a result of differences in modeling approaches and inherent assumptions in how certain characteristics (ground motion duration, for example) are defined.

Other SDOF studies (Bernal 1987; MacRae 1994) have examined P-Δ effects in seismic simulations. Bernal proposed a simplified method to assess the dynamic instability of two-dimensional frames whereas MacRae concluded that the post-yield ratio assigned to bilinear force-deformation models has a significant impact on the maximum plastic deformation of SDOF oscillators. Miranda and Akkar (2003) developed an empirical expression through statistical analysis of a large set of response data from nonlinear simulations of SDOF models to predict the minimum lateral strength required to prevent dynamic instability.

Finally, nonlinear static (or pushover) analysis can provide useful information on the collapse resistance of a building. It can identify, for example, if the strong-column weak-beam principle in seismic design has been achieved or if the building has a soft story. It provides a fairly good estimate of the building force capacity but not the deformation capacity. Moreover, a pushover analysis is a static method where the loads are applied monotonically and cannot account for several other important effects such as the influence of higher modes and degradation of strength and/or stiffness due to cyclic effects. Despite numerous advances in pushover methods to somehow account for higher modes and cyclic effects, they cannot replace fully nonlinear dynamic simulations.

3. HIGH FIDELITY ANALYSIS OF RC COMPONENTS

The literature on nonlinear finite element modeling of RC components and connections is extensive. However, despite decades of research and advances in nonlinear modeling of RC structures, many challenges remain in predicting the failure of RC components under extreme loads. In many early studies, material and model parameters had to be tuned to experiments hence limiting the application of such models.

The author and co-workers (Lucchini et al. 2017) investigated the possibility of employing available advanced FE models to complement expensive experimental testing with virtual or numerical experiments. The goal was to evaluate the effectiveness of current finite element software, without model tuning, to simulate the failure of reinforced concrete components under lateral loads. The particular case of a shear-critical non-ductile column was considered since experimental data was available to validate the modeling. Findings from the work could form the basis for developing simpler modeling strategies using a combination of continuum and frame elements to enable full 3D seismic simulations of non-ductile buildings under seismic excitations. This effort is described in the following section.

Among the available software for the 3D analysis of reinforced concrete members, the general-purpose finite element code LS-DYNA (Hallquist 2012) with an explicit solver was used. It should be noted that explicit methods are conditionally stable and require a step size based on the highest eigenvalue of the model. Concrete was modeled with an 8-node solid element with one integration point and viscous hourglass control. The main features of the model include: isotropic constitutive equations, a yield surface formulated in terms of three stress invariants, a hardening cap that expands and contracts, a damage-based formulation to degrade the stiffness and a regularization technique to deal with mesh sensitivity during softening. Reinforcing steel was modeled using a nonlinear beam-column element that can simulate compression buckling. The most important aspect of the model was the development of interface models to simulate the steel-concrete interaction. Duplicate nodes were specified along the reinforcement bars at locations where concrete and bar nodes intersect. At locations where both transverse and longitudinal reinforcement cross, three nodes were specified at the same location. The interface was represented by springs and unique sets of constraints to best represent the interaction at that location. Penetration of the reinforcing bars into the concrete core was prevented through contact constraints specified by penalty functions. Kinematic constraints were also used to model the restraints provided by the transverse reinforcement against lateral deformations of the longitudinal bars. On each column face, constraints are defined normal to the interface whereas at corners, the nodes were constrained in both transverse directions of the bar. Spring elements were used to model bond at the steel-concrete interface. The constitutive behavior of the spring was defined using a monotonic bond-slip law calibrated based on recommendations in fib (2013).

As indicated earlier, a focus of this research study was to analyze older non-ductile RC members. For the particular specimen simulated in the study, the transverse reinforcement contained 90-degree end hooks that was susceptible to opening under large lateral deformations. Considerable effort was directed to simulating the possible opening of the stirrups. To accomplish this, the two beam elements of the stirrup at the hook location were not connected directly with the node of the longitudinal bar, but instead, were connected through the cover. Allowing concrete elements in the cover to erode at ultimate strain, the longitudinal bar located at the corner of the cross section was therefore free to laterally deform and eventually to buckle.

A sample subset of the simulated response of a non-ductile column tested by Sezen (2000) is shown in Fig. 1. Complete details of the modeling and simulated responses can be found in Lucchini et al. (2017). It was demonstrated that the above-referenced strategy adopted to model a reinforced concrete column was successful in capturing the interaction between concrete, longitudinal and transverse reinforcement, as well as local phenomena such as confinement effects on concrete due to transverse reinforcement, buckling of longitudinal bars, opening of stirrups, and bond effects.

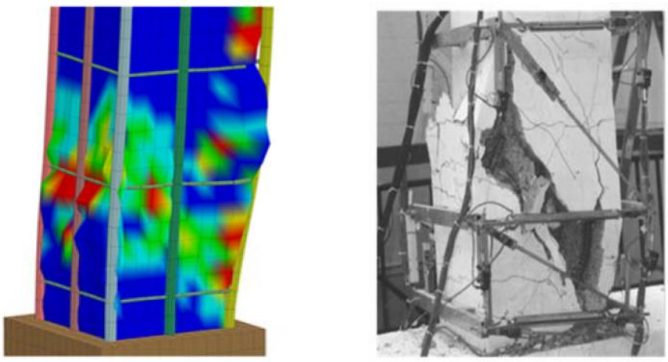


Fig. 1 Shear strain contours obtained with the FE model compared to photo from Sezen (2000) showing damage state at the end of the test.

4. DEVELOPMENT OF COLLAPSE FRAGILITIES

Collapse fragility curves are widely used for assessing seismic vulnerability of structures. Haselton et al. (2011) examined the collapse safety of thirty different ductile RC moment frame buildings of varying height and bay widths using this approach. Additionally, nonductile frames were also studied for comparison (Liel et al. 2011). Besides, collapse fragility curves are useful to examine uncertainties affecting building collapse. By comparing collapse fragility functions for short and long duration ground motions of different buildings, Raghunandan and Liel (2013) established the significance of ground motion duration in collapse resistance. All these studies were based on frame models with concentrated nonlinear springs at element ends incorporating degrading strength and stiffness.

4.1 Incremental Dynamic Analysis

Incremental dynamic analysis (IDA) is a parametric analysis method that is used to generate collapse fragility curves. It involves subjecting a structural model to several ground motion records, each scaled to multiple levels of intensity, thus producing curves of response parameterized versus a ground motion intensity level (Vamvatsikos et al. 2002). Each horizontal component of the selected ground motions is individually applied to the structural model. Ground motion records are typically amplitude scaled according to the spectral acceleration at the first-mode period, $S_a(T_1)$. The ground motions are incrementally scaled until collapse occurs. One of the challenges using the IDA approach is the definition of collapse. In a nonlinear seismic analysis, collapse is typically defined as the point of dynamic instability, where the lateral story drifts of the building increase without bounds (often called sideway collapse). This occurs when the IDA curve becomes flat. It is often necessary to actually plot the story drift histories to check if a non-converged solution is a result of excessive displacements leading to collapse.

4.2 Modeling sensitivity

Existing studies in the literature on assessing the collapse probability of RC frames are based primarily on concentrated plastic hinges introduced at the ends of each element. Limited studies employ fiber-based cross-section integration that included axial-moment interaction. Hence, a sensitivity study was undertaken to compare collapse fragilities using different element models as well as different constitutive models for reinforcing steel when the frame element is modeled using fiber-based sections.

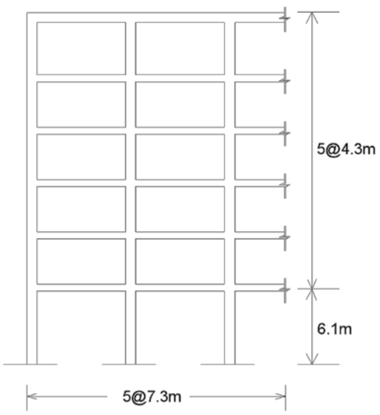
Element models: Two types of element models were considered: Model A – frame elements modeled with concentrated hinges at the element ends - in this case, the spring response was modeled as using a trilinear envelope with post-peak softening to incorporate cyclic degrading effects; and Model B – frame elements modeled using fiber-sections at each integration point along the element length - in this case, different cyclic constitutive models were used for the reinforcing steel as described in the next section.

<u>Constitutive models for reinforcing steel</u>: Two primary modeling options were considered: Model B1 – the uniaxial cyclic steel material (*Steel02* in OpenSees (2018)) with isotropic strain hardening; and Model B2 – a trilinear envelope with post-peak softening beyond the ultimate stress (achieved with the *Hysteretic Material* in OpenSees).

The effect of the above variables on the collapse probability of a typical 6-story reinforced concrete (RC) frame building was examined. To evaluate the difference in the response of the building to the different modeling choices, the building model was subjected to a set of near-fault ground motions which contain strong coherent long period pulses and permanent ground displacements caused by rupture directivity effects.

4.2.1 Building Detail and Ground Motions

The building considered for the study was designed for a site in San Francisco in accordance with the requirements of ASCE/SEI 7-16 (2016) and ACI 318-14 (2014). The following spectral values were used to establish the design base shear: S_s = 1.7 g and S_1 = 0.79 g. The elevation view of a typical frame as well as section sizes and reinforcing details is shown in Fig. 2. The building is symmetric in plan, hence only a typical interior frame was considered in the analysis. An eigenvalue analysis of the frame indicated a first mode period of 1.1 sec and a second mode period of 0.37 sec.



	Beams				Columns		
Floor	Size, b x h (in x in)	Long. Reinf. Bott.	Long. Reinf. Top	Trans. Reinf.	Size, h	Long. Reinf.	Trans. Reinf.
1	26x30	8 #9	8 #10	#3 @ 6	30	14#12	#4 @ 5
2	22x30	6 #8	6 #9	#3 @4.8	30	14#12	#4 @ 5
3-5	22x28	6 #8	6 #9	#3 @4.8	28	11#12	#4 @ 5
6	20x24	3 #8	3 #8	#3 @4.8	24	8#12	#4 @ 5

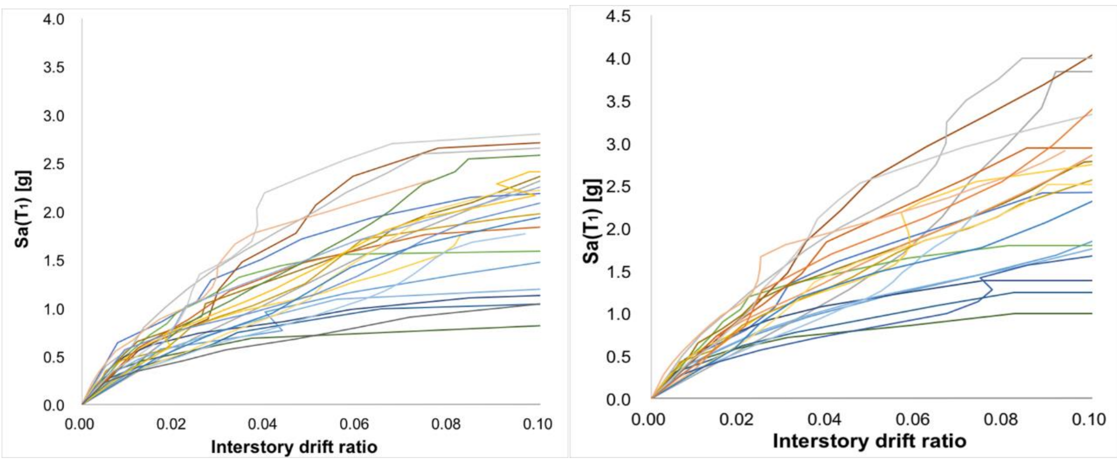
Fig. 2 Elevation of 6-story RC frame with section and reinforcement details

The building was subjected to 20 ground motions extracted from the PEER Strong Motion database. Criteria used in the selection were: magnitude 5.0 – 8.0 and fault distance 0 – 20 km and soil sites with shear wave velocity 200 – 400 m/s.

4.2.2 Simulation Results and Findings

Incremental Dynamic Analysis (IDA) was used to establish the collapse fragility curves. Two-dimensional nonlinear response history analysis (NRHA) for generating the IDA curves was performed on the frame model using the OpenSees (2017) platform. As described previously, the following modeling choices were investigated: Model A – frame with concentrated springs; Model B1 – fiber section model where the reinforcing steel was modeled using *Steel02*; and Model B2 – fiber section model where the reinforcing steel was modeled using the *Hysteretic Material* with post-peak softening. The fiber-section based models use force-based nonlinear beam-column elements for all members, with four and five integration points along beam and column elements, respectively. The material used to define the behavior of concrete is *Concrete 02* material which utilizes the well-known Kent & Park model in compression and linear elastic behavior in tension up to tension cracking followed by linear softening.

The generated IDA curves for Model A and Model B1 are shown in Fig. 3. Based on the IDA curves, the collapse drift was specified as 8% so as to develop the collapse fragilities. Since the *Steel02* material only permits strain hardening without limits, higher spectral demands are needed to cause collapse and the dispersion at this limit state is also higher than Model A. Given this drawback in using a non-softening constitutive model for collapse analysis, it is necessary to impose limiting strains to signal the fracture or buckling of reinforcing steel. One of the options in OpenSees allows the user to cap the strain/deformation limits of a constitutive model with the command *minmax*. In this study the following limiting strains were used: tension max: 0.12 and compression min: 0.04. The effect of adding these capping values is shown in the comparison of fragility curves presented in Fig. 4a (where Model B3 refers to *Steel02* with capping limits specified through *minmax*. A comparison of all the generated fragility curves using different models is displayed in Fig. 4b.



(a) (b)

Fig. 3 IDA curves for: (a) Model A; (b) Model B1

It is observed from Fig. 4a that the collapse probability increases slightly when the capping strains are specified. Next, examining the collapse probabilities shown in

Fig. 4b, it is seen that the most conservative estimates of the collapse probability is obtained when using the concentrated plastic hinges with nonlinear behavior specified in terms of the moment-rotation response of the hinge. Using a non-softening model for the reinforcing steel behavior obviously results in the least conservative estimate. Adding the limiting strains is recommended when using such a model since the predicted responses at the collapse limit state are more reliable.

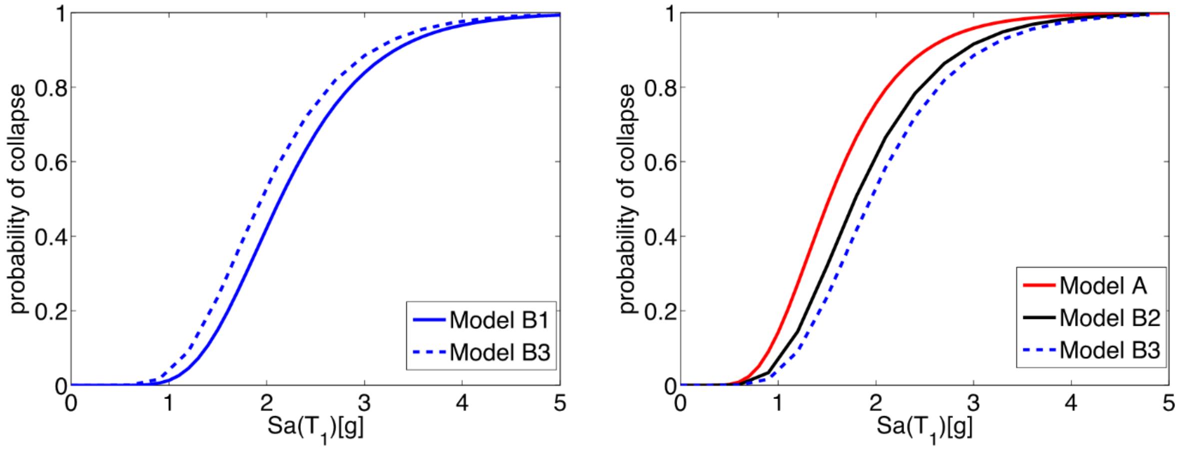


Fig 4 Comparison of collapse fragilities for different models

5 CONCLUSION

Advances in performance-based seismic engineering have promoted the need to assess the collapse risk of buildings in a future seismic event. However, modeling choices can influence the estimation of the collapse probability of an RC building. The information presented in this paper provide some insights into issues that can have an impact on collapse assessment of structures in general and RC buildings in particular.

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