Development of Collapse Indicators for Older-Type Reinforced Concrete Buildings

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SUMMARY:
This study examines the seismic collapse safety of older, non-ductile reinforced concrete building frames designed and constructed prior to 1970s in active seismic regions. The evaluation was performed by using a 6-story, 5-bay “benchmark” RC frame having properties similar to modern construction. Non-linear dynamic analysis simulated in structural analysis software was used to assess the seismic behavior of the structural frame. Consequently the “benchmark” frame was modified by varying two different structural parameters (the transverse reinforcement ratio and the column-to-beam moment strength ratios). The Incremental Dynamic Analysis method was used to determine the probability of collapse for each modified frame. The results indicate that simple engineering calculations determined by review of the engineering drawings could be utilized for an assessment of the behavior of RC structural frames.

Keywords: Incremental dynamic analysis; Reinforced concrete; Non-ductile concrete buildings; Seismic assessment

1. INTRODUCTION

A wide variety of concrete buildings were built in the 20th century in the United States and elsewhere, many of them before introduction of modern seismic design requirements. Many of these buildings were constructed with low base-shear strength and with details and proportions that result in low ductility/displacement capacity. Some of these buildings pose a high seismic risk to building occupants, and will be a major contributor to casualties in future earthquakes. An important goal is to be able to identify the highest risk buildings so that mitigation efforts can be directed to improve their safety.

One method for identifying high-risk buildings is to identify the codes to which they were designed. In the highly seismic western U.S., modern requirements for ductile design of concrete buildings were introduced in building codes starting in the mid-1970s. By 1980, these requirements were widely implemented. This benchmark year thus provides a date by which to classify older-type designs versus more modern-type designs. Unfortunately, in the counties of highest risk in California alone, over 20,000 such pre-1980 concrete buildings exist. Retrofitting all these buildings clearly is impossible given economic, social, and political constraints. Alternative procedures are required.

In this paper we report results of an exploratory study to identify characteristics of older-type concrete buildings having highest risk of collapse. The study begins with an idealized building frame that was detailed to comply with the current code provisions, but with strength typical of older-type construction. This benchmark building was sequentially weakened by modifying transverse reinforcement and column-to-beam moment strength ratios. For each case, the collapse risk was evaluated to identify combinations that result in sudden changes in collapse risk. The intent is to develop a set of “collapse indicators” whose presence in a building can be used to indicate a higher propensity for collapse compared with the background population of older-type concrete buildings.

2. DEVELOPMENT OF THE "BENCHMARK BUILDING"

The "benchmark building" corresponds to a 6-story, 5 bay reinforced concrete frame building. The idealized building consists of four perimeter moment-resisting frames, two in each direction, that were designed to resist gravity and earthquake forces while interior frames were designed to resist only gravity loads. Each perimeter frame was proportioned to have design strength sufficient to resist base shear of approximately 10% of half of the self-weight of the building.

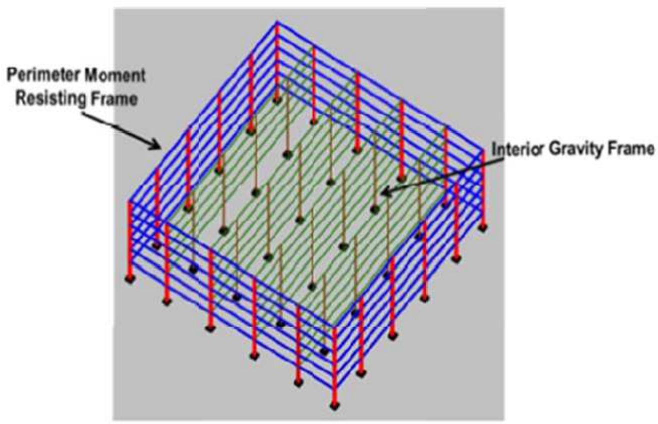

Figure 1. Three-dimensional view of the benchmark building

The building presented in Figure 1 does not correspond to a real, existing building, but instead it was designed solely for the needs of the current study. The "benchmark building" was designed to satisfy the detailing and proportioning provisions of ACI 318-08 (2008), including all requirement for configuration and spacing of transverse reinforcement, all requirements for development and splicing of reinforcement and the requirement that the sum of nominal moment strength of columns be at least 6/5 times the sum of nominal moment strength of beams at every beam-column connection except for those located at roof level. Accordingly, the "benchmark building" was expected to perform in a ductile manner without common performance deficiencies found in many older concrete buildings. This building was expected to serve as a reference point for a building type that would be unlikely to require any seismic rehabilitation and that it could be used as a pivot for every comparison regarding the collapse risk.

3. NON-LINEAR SIMULATION MODELS

The idealized building described in the previous section was assumed to have symmetric plan, such that the building responds to earthquake ground shaking with minimal plan torsion. Furthermore, it was assumed that the perimeter frames provide the majority of resistance to lateral forces. Therefore, to simplify the analysis approach, the building was modeled using a two-dimensional (2D) structural frame simulating only the vertical and lateral resistance of the perimeter frame. The decision to model only the vertical and lateral resistance of the perimeter frames necessitated making additional assumptions about distribution of seismic mass and gravity loads. The perimeter frames in each direction were assumed to carry the total seismic mass of the structure. Thus, the single frame, shown in Figure 2, was assigned to carry half the building seismic mass with masses lumped at floor levels. To account for P-Delta effect, a "leaning column" supporting assigned gravity loads was included in the model. At each level, the leaning column supports applied vertical load equal to 67% of half of the weight of the structure. This ratio was selected according to the judgment of the authors as to how much resistance would be provided by the interior framing, and was not a result of any detailed calculation.

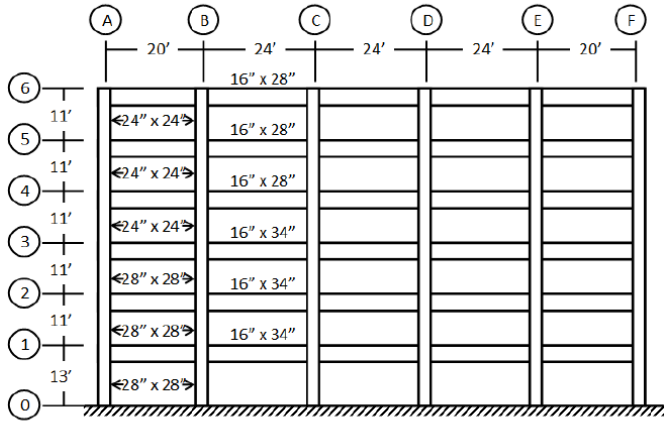

Figure 2. Elevation view of the perimeter frame

The structural analysis model is an assemblage of line elements representing the flexibilities of beams and columns connected to zero-length joints. The structural analysis model was assumed to have fixed supports at the foundation, except the leaning column which was pin-supported. The diaphragm was assumed rigid; therefore, all the joints at a given level were constrained to have equal horizontal displacements. For dynamic analyses, damping was assumed equal to 2% of the critical damping.

To model the non-linear flexural behavior of the structure, a lumped plasticity approach was used for both beams and column. By this approach, all the elements consist of three parts; a linear elastic element and rotational springs at each end. The linear elastic elements have flexural stiffness properties calculated from the members cross section characteristics. The flexural stiffness of the structural members was reduced in accordance with ASCE-41 Supplement 1 stiffness modification factors to account for concrete cracking and reinforcement slip from connections. The rotational spring element behavior was simulated in OpenSees based on the Clough model, using hysteresis implemented in OpenSees by Ibarra et al. (2005). The most important aspect of this model is the post-capping negative slope, which enables modeling of the strain-softening behavior associated with concrete crushing, reinforcement buckling, and reinforcement fracture. The model also incorporates cyclic strength degradation. The parametric values for the calibration of the spring elements were calculated according to the equations presented in Haselton et al. (2007/03).

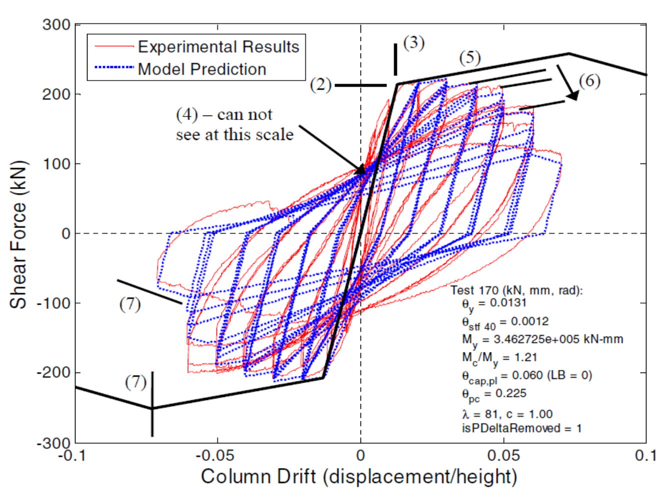

Figure 4. Response of a calibrated spring-column element (Haselton et al. PEER 2007/03)

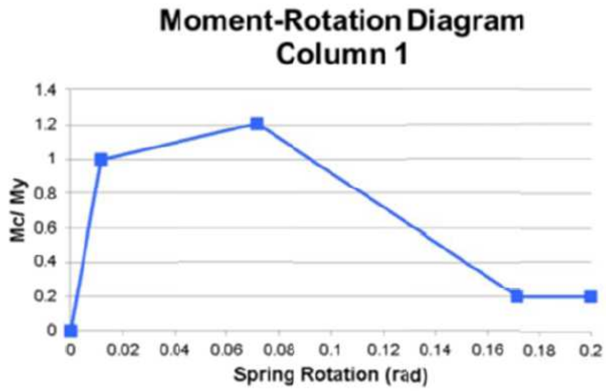

Figure 5. Backbone moment-rotation curve of the spring element of a typical column member of the "benchmark" building

As mentioned above, in addition to the benchmark building, additional designs of the same building were considered in which the spacing of transverse reinforcement was increased so that it covers the expected range in design and performance in California's older RC frame buildings. To measure the variation of shear demand over the shear resistance, an easily calculated parameter V_p/V_n was employed. In this ratio, the plastic shear demand V_p was assumed to be equal to $2*M_p/H_{column}$, where M_p corresponds to the plastic bending moment strength of a column member and H_{column} to the column height; the shear strength V_n was assumed to be equal to the nominal shear strength as suggested by ASCE 41-06. Consequently the V_p/V_n ratio was calculated for each column and an average value for the structure was computed.

The benchmark building model, which corresponds to $V_p/V_n = 0.4$ (a typical column of the benchmark building has 4-legged hooks with spacing equal to 4 in.), was assumed to model only flexural failure explicitly, so it corresponds to a purely flexural failure model. However, in accordance with ASCE-41, as V_p/V_n becomes higher than 0.7 the possibility of having a shear failure that follows or even precedes flexural yielding becomes likely. Table 1 presents the relation between the spacing of the transverse reinforcement and the V_p/V_n ratio (the value of V_p/V_n presented in the table corresponds to an average estimated value).

Table 1. Relation between spacing of transverse reinforcement and average V_p/V_n ratio

Spacing(in.) – No of hoop legs	**4-4 legs**	**8 – 4 legs**	**16 – 4 legs**	**14 – 2 legs**
V_p/V_n	0.4	0.7	1.0	1.3

To model shear failure the limit-state material as developed by Elwood and Moehle (2008) was incorporated for the column members of the model. Thus, models with increased spacing of transverse reinforcement correspond to flexural-shear models where shear and axial failure is modeled explicitly. The limit-state material introduces horizontal and vertical springs at the top of each column member that allows modeling shear and axial failure. The limit state material is a uniaxial material that monitors the response of the beam-column element to detect the onset of shear failure. The material consists of two branches: a) Linear elastic branch prior to shear failure b) Linear degrading branch after shear failure has occurred.

In the structural analysis, the uniaxial material model queries the column element for its force and deformation and then checks whether these demands exceed the relevant limit curve. If they do, then shear or axial failure is triggered and the shear or axial force correspondingly begins to degrade.

The limit-state material developed by Elwood and Moehle (2008) assumes that shear failure initiates following onset of flexural yielding. The model was updated such that shear failure could be initiated prior to flexural yielding. For this purpose, the model checks at every instance whether the applied shear exceeds the initial shear strength. If it does for any instance, shear failure initiates according to the Elwood and Moehle model.

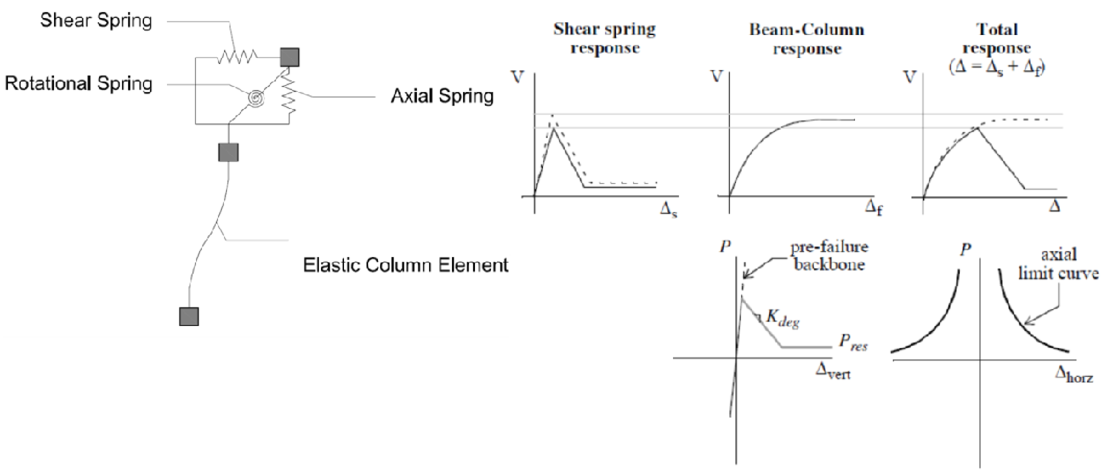

Figure 6. Limit State Material (developed by Elwood and Moehle)

The calculated elastic fundamental period of the benchmark building (Table 2) corresponds to the effective "cracked" stiffness of the beams and columns (30% of EI_g for both the column and beam members). The computed period is significantly larger than the values calculated from empirical formulas in ASCE or other standards due to modeling assumptions (these include, use of effective stiffness for the eigenvalue analysis and exclusion of the gravity resisting system from the analysis model).

Table 2. Modal analysis of "benchmark" building

Modes	Modal Period(sec)
1st	1.59
2nd	0.58
3rd	0.33

Nonlinear static (pushover) analysis was performed for both the benchmark building (which corresponds to a purely flexural model) and for the same building model but with wider spacing of transverse reinforcement so that the buildings can potentially fail in shear or flexure (flexural-shear models). The results of the analysis are presented in Figure 7.

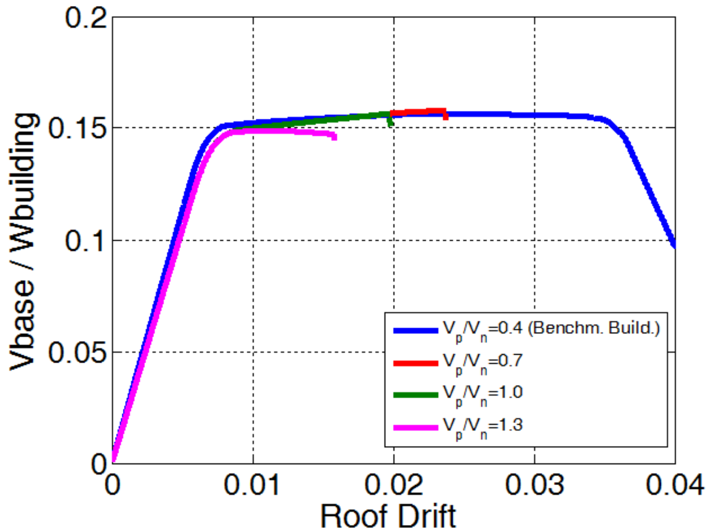

Figure 7. Pushover Analysis of model building for different V_p/V_n ratios

As illustrated in Figure 7, the "benchmark building" model exhibits a ductile response, which is not unexpected considering that its detailing and proportioning correspond to a modern building. Conversely, as the spacing of the transverse reinforcement increases, the response of the other buildings becomes more brittle and shear failure occurs at relatively low roof drift values.

4. ASSESMENT OF SEISMIC BEHAVIOR

The assessment of the seismic behavior of the studied building models was performed using the incremental dynamic analysis method (IDA). According to this method each studied non-linear model is subjected to numerous dynamic analyses under multiple ground motions scaled gradually to increasing acceleration amplitude. Two types of collapse were considered. Sideway collapse was defined as maximum interstory drift exceeding 10% of story-height, and vertical collapse defined as axial failure of more than 50% the columns in one story. A suite of 22 pairs of ground motions were selected. The set of ground motions was selected to be the same with the one used for FEMA P-695 (ATC-63). Each of these ground motions was scaled and the spectral acceleration level causing collapse of the building was measured. The spectral acceleration was evaluated at the first-mode period of the building (T_1). In Figure 8 each line represents the response of the structure to a single ground motion record scaled to increasing intensity.

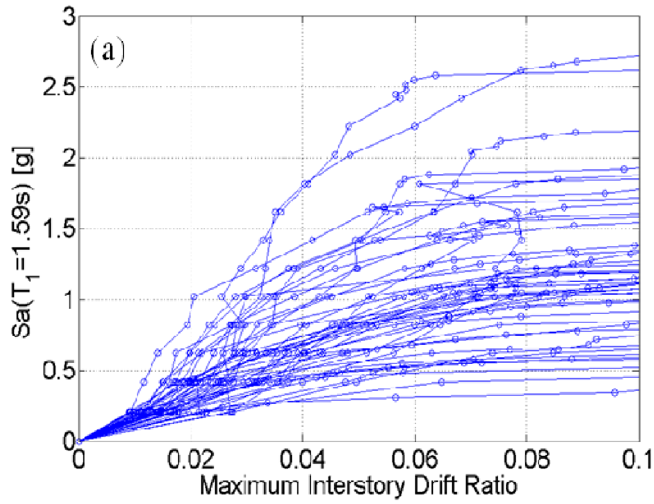

Figure 8. Incremental Dynamic Analysis (IDA) Results for "benchmark" building

The collapse risk of each of the structural analysis model was obtained from statistics on the IDA results. In this study, collapse performance was evaluated using the probability of collapse at a specified level of ground motion intensity, the Maximum Considered Earthquake level for a site in California (in this study it was assumed that $S_a(T_1) = 0.9g/T_1$). The collapse fragility function represents the probability of collapse, as a function of the ground motion intensity level, defined in terms of $S_a(T_1)$. The collapse probabilities in terms of $S_a(T_1)$ were assumed to be long-normally distributed. Figure 9 presents the collapse fragility curves for the benchmark building and for the same model but with wider spacing of transverse reinforcement.

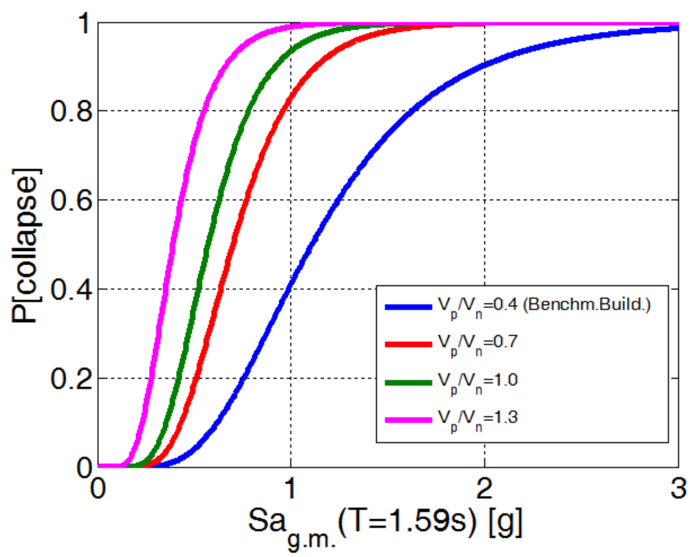

Figure 9. Fragility curves for different V_p/V_n ratios

As expected the benchmark building exhibits a lower collapse risk compared to the same building but with wider spacing of transverse reinforcement. It is recalled that the benchmark building satisfies all the requirements of the current codes, including the requirement that the sum of column nominal moment strengths be at least (6/5) times the sum of beam nominal moment strengths at every beam-column joint. This requirement is commonly referred to as the "strong-column-weak-beam" requirement. Its purpose is to promote beam yielding rather than column yielding, thereby spreading flexural yielding over multiple stories as the building responds to strong earthquake shaking. To study the effect of weak column (or stories) on the collapse risk of these buildings, this study defined and investigated the effect of $\Sigma M_c/\Sigma M_b$ ratio in the seismic response. The benchmark building corresponds to an average of $\Sigma M_c/\Sigma M_b = 1.2$. To modify the studied $\Sigma M_c/\Sigma M_b$ ratio, the bending moment strengths were scaled accordingly. The results of the IDA analysis for the case where the effect of the combination of two structural parameters are varied, V_p/V_n and $\Sigma M_c/\Sigma M_b$, are shown in Figure 10.

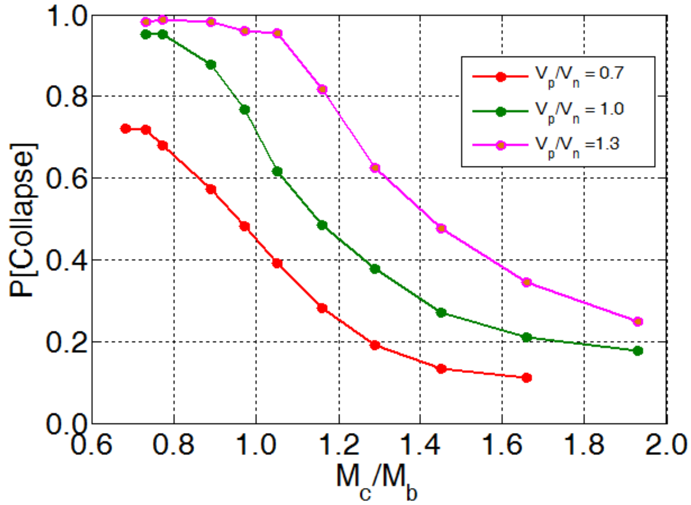

Figure 10. Probability of collapse at the MCE level for different V_p/V_n and $\Sigma M_c/\Sigma M_b$ ratios

From Figure 10 it can be observed that V_p/V_n and $\Sigma M_c/\Sigma M_b$ exhibit a significant correlation with collapse potential. For Vp/Vn=0.7 a significant change of slope in the curve is observed for $\Sigma M_c/\Sigma M_b$ in the range 1.2 to 1.4. This suggests that $\Sigma M_c/\Sigma M_b \sim 1.2$ to 1.4 is an optimal ratio for relatively ductile frames, as increases in the ratio come at significant column expense without significant payoff in reduction of collapse probability. However, as V_p/V_n increases, the optimal point shifts to larger values of $\Sigma M_c/\Sigma M_b$. Put simply, this result reflects that fact that an existing building is more susceptible to story mechanisms as columns become more highly shear-critical.

5. CONCLUSIONS

This study explored the effect of two structural parameters, specifically, ratios $\Sigma M_c/\Sigma M_b$ and V_p/V_n, on the collapse potential of a frame structure. The selected parameters are easily calculated without performing any complicated analysis and correspond to key concepts of the capacity design principle of modern seismic codes. It is found that both (a) the ratio of column-to-beam moment strength and (b) the ratio of column plastic shear demand-to-shear capacity exhibit a strong correlation with the collapse risk of the building system. Consequently, the values of these quantities can serve as important collapse indicators to identify buildings especially prone to structural collapse. The study shows that the two ratios interact, such that both need to be considered together to appreciate the collapse risk of a building.

The current study is limited to single frame geometry. Its specific conclusions, therefore, must be limited to the frame studied. The results, however, show the potential of being able to identify collapse indicators for buildings that would enable the rapid identification of buildings having highest safety risk. Additional studies are required to generalize the results reported here.

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