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SEISMIC PERFORMANCE OF REDUCED BEAM SECTION MOMENT FRAMES CONSIDERING RECORD-TO-RECORD UNCERTAINTIES

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ABSTRACT

Reduced Beam Section (RBS) moment frame is one of the most economical and practical types of steel moment resisting systems recommended by new advanced seismic standards in the aftermath of the 1994 Northridge earthquake. In this study three 4-, 8-, and 16-story reduced beam section perimeter frames are considered. Through comparison with test results, a connection model which accounts for cyclic deterioration is developed and validated. The calibrated connection is then implemented in a two dimensional frame model using OpenSees software. Two performance goals “Immediate Occupancy, IO”, and “Collapse Prevention, CP” are considered as objective of this study. To quantify performances regarding each of these goals, S_a values at which different records induce failure to the structures (S_a capacities) are measured through Incremental Dynamic Analysis (IDA) procedure. These capacities are then treated probabilistically to account for record-to-record uncertainties. That is, S_a capacities of structures are extracted with respect to different failure probabilities and corresponding to alternate performance levels. Structural performance is finally discussed regarding issues such as dominant failure mechanism of structures.

Introduction

Connections in a steel frame experience high force and deformation demands because of their critical position in structure. Furthermore, suffering from the abrupt discontinuity caused by structural detailing, makes connections susceptible to several unfavorable failure modes. Thus, performance of steel frames is greatly influenced by behavior and capacity of their connections. This fact was further highlighted after the widespread damage observed in steel structures in Northridge 1994 earthquake. Afterwards, an extensive research program was conducted by the US Federal Emergency Management Agency (FEMA) to address the issues stimulated by that event. The outcome of the wide investigations performed on these, improved engineering intuition for better understanding of connection performance. Subsequently, a set of state-of-the-art reports and several guidelines were published (FEMA 2000 and AISC 2002).

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In order to ensure providing adequate strength, stiffness, and ductility, two alternatives are included in connection provisions (AISC 2002). One choice is to adopt one of the prequalified suggested connections, and the other is to perform project-based testing that shows the appropriateness of the connection. The connections approved as prequalified by FEMA (2000) have an improved detailing so that the beam plastic hinge formation is shifted away from the column face. High demands at the beam-to-column interface (possibly the main reason for brittle failures during the Northridge earthquake), are therefore significantly decreased. These prequalified connections are of two main categories; reinforced detailing and Reduced Beam Section (RBS) detailing. In the former, the formation of plastic hinge in the beam-column interface is hindered by providing reinforcement, while in the latter, lower strength of RBS shields the connection from high demands. RBS connections require less welding and material than reinforced connections. Thinner doubler plates and continuity plates also make RBS connections advantageous from the economical stand point.

Many experimental programs have incorporated the concept of RBS such as Popov (1998) and Engelhardt (1996, 1998 and 2000). Three RBS types tapered cut, radius cut, and constant cut were the main focus of these studies; among them, better performance was evinced by tapered cut and radius cut connections. In general, the observed overall cyclic behavior of connection assemblies, has led investigators to unanimously declare that RBS connections are suitable for use in special moment resisting frames. Numerical investigations, also, deduced that RBS connections could achieve the target beam plastic rotation at lower strain demands than comparable traditional connections because strain concentrations in the critical connection region were alleviated.

Unlike the wide investigations performed on the performance of connection assemblies (Engelhardt 1996, 1998 and 2000), system level studies of RBS frames are still quite limited and their manner needs to be investigated through a probabilistic procedure for consideration of earthquake associated uncertainties. Therefore, in this study we adopted Incremental Dynamic Analysis (IDA) approach (Vamvatsikos 2002) to evaluate the uncertainties introduced by inherent randomness of earthquake hazard. To do this, a connection model that accounts for strength degradation within hysteretic loops, was first developed using OpenSees (PEER 2006) program, and then was calibrated and verified using laboratory data from several connection tests performed by Engelhardt (2000). This connection model was next incorporated in modeling of a perimeter frame which was designed by previous researchers (Jin 2002), and finally, the IDA procedure was applied and the capacity of structure for two different performance levels, Collapse Prevention (CP) and Immediate Occupancy (IO), was extracted. First mode-5% damped spectral acceleration ($S_a(T_1, 5\%)$) is used to represent the intensity measure of earthquakes. The structures' failure probability at different S_a levels is presented in form of fragility curves. Lastly, capacities of structures are tabulated in terms of earthquake intensities corresponding to structural failure. These values are reported for different failure probabilities and regarding aforementioned performance levels.

Modeling Procedure of Prototype Buildings

Three 4- , 8-, and 16-story RBS perimeter frames utilized in this study were formerly designed by Jin (2002) with reference to pertinent provisions in FEMA (2000) and AISC (2002).

These frames were standard office buildings located in a C class site near Los Angeles and were proportioned as special moment resisting frames with identical floor plans. Beam flange reduction of RBS connections was specified as 50% for the majority of connections. Member sections and dimensions of these frames are illustrated in Fig. 1. Detailed information about designing procedure and other structural details of these buildings is found in Jin (2002).

Establishment and Calibration of Cruci-form Subassembly

In this study a cruci-form assemblage is established and subsequently validated on the basis of experimental investigations performed by Engelhardt (2000) on several connection specimens. The employed subassembly model consists of beams, columns, and a panel zone as well as two rotational springs located at a distance X from beam-column interface (where the beam cross sectional properties are critical due to RBS detailing). A schematic of model is depicted in Fig. 2. Boundary conditions of subassembly are defined presuming that the points of inflection fall at the mid-length of beams and columns. Dimensions of subassembly are same as those of Engelhardt (2000) and coupon test results are used as member strengths. In OpenSees software (PEER 2006), beams are modeled as elastic elements with nominal properties of their sections because inelastic behavior is deemed to concentrate in critical section of RBS (as the philosophy of RBS connection provides and is also approved by experiment results). Columns of the assembly are modeled incorporating nonlinear beam-column elements that are based on flexibility formulation. Columns cross sections are defined using fiber section modeling feature of OpenSees. In this method the member section is discretized into a number of fibers that can have different stress-strain relationships, therefore, distributed plasticity can form at any point along section's height and width. A bilinear steel material with 3% kinematic hardening is assigned to each of the fibers of the cross-section. The panel zone behavior of the established assemblage is adopted from the trilinear model proposed by Krawinkler (1978). Panel zone deformation has also been controlled to comply with testing results from Engelhardt (2000).

The most important part of cruci-form modeling process is to properly define RBS springs. As a part of study by Jin (2002), an analytical solution is given for determining the moment of inertia of an equivalent beam (I_e) that could represent RBS portion. This formulation accounts for the continuous variation of cross section within RBS portion. Based on this I_e value and using a regression analysis, a closed-form equation is then derived for calculating initial stiffness and yielding moment of the rotational spring used as RBS representative. The mentioned equations are used herein in conjunction with an assumption for RBS plastic moment strength to be proportioned to 90 percent of beam's plastic modulus in its critical section. A vital characteristic of connection moment-rotation curve which has not been accounted for in the work by Jin (2002) is the cyclic deterioration evident in testing results. From perspective of determining structural capacity, limit state behavior of all elements must be modeled with a reasonable accuracy, thus, an appropriate consideration of deterioration is crucial to the purpose of this study. For modeling this property normalized peak damage model is utilized. This damage model is based on the maximum value of the response parameter of interest and is considered as a non-cumulative damage model. Where the response parameter of connection was set to maximum plastic rotation, this model was found to appropriately reflect the cyclic deterioration of RBS connection. Joining this damage model with a trilinear relationship formed the cyclic moment-rotation curve of RBS connection which satisfactorily fits the observed experimental curve. The cyclic “horizontal load vs. tip-column displacement” histories obtained from test and OpenSees model are compared for specimen DDBWC (Engelhardt 2000) in Fig. 3. The good agreement between results confirms the appropriateness of the established model.

Modeling of Structural Frames

The prototype buildings are modeled based on the calibrated materials and elements obtained from connection subassembly. The superior advantages of OpenSees software, such as, nonlinear materials and elements library, also, equation solution algorithms, are used for performing a fully nonlinear analysis (considering both material and geometric nonlinearities) on the defined structural model. The buildings masses include total dead load plus 25 percent of live load and are lumped at joints. Regarding the symmetry of buildings plans only one-half of each frame is considered in the analysis. Neglecting the orthogonal effect of earthquake loading, one perimeter moment resisting frame is modeled only in one direction. The p-delta effect imposed by internal gravity frames on perimeter moment frame is considered through defining leaning columns.

Incremental Dynamic Analysis (IDA)

Incremental Dynamic Analysis is a newly developed method for quantifying earthquake-induced uncertainties in structural reliability assessment (Vamvatsikos 2002). In order to perform the IDA, a set of ground motion acceleration records are selected. A response history analysis is performed using increasing scales of each record and the predicted response parameter (Demand Measure, DM) of interest is recorded for each analysis. The scaling action starts from a sufficiently low value of Intensity Measure (IM) (a parameter assumed as a suitable representative of earthquake intensity), such that the structural response is kept linear and continues until the “structural collapse” state is reached. A plot is then made of the predicted DM maxima against the IM value to which the ground motion record was scaled. One of the critical issues regarding determining structural capacity through IDA is to correctly trace structural collapse. To numerically quantify collapse state, two alternate (or supplementary) definitions can be made; those are, “softening of IM-DM curve”, and, “numerical instability”. The former, is a situation in which IDA curve (IM vs. DM) starts to flatten; rather, the latter occurs when solution algorithm fails to converge. Occurrence of softened behavior in concept, is the situation in which, structure has exhausted its reserves for resisting lateral displacements; hence, a small increment in input excitation may lead to excessive increase of structural response. Regarding IDA results, this phenomenon can be quantified by defining a slope limit for IM-DM curve. FEMA350 (2000) states that the occurrence of flattening coincides with the IM-DM slope becoming less than 20% of initial elastic slope. This definition, although straight, in most cases, leads to large values of DM that are not acceptable by practical evidences. In consequence, this definition is not applicable in absence of engineering judgment and experimental evidences. Therefore, relevant provisions often accompany the slope limit method by an utmost value for DM measure. FEMA350 (2000) specifies that maximum inter-story drift capacity of steel moment frames determined through flattening criterion shall not exceed the value of 10%. Alternatively, numerical instability is a state in which solution algorithm cannot converge to a compatible set of forces and deformations for current state of structure’s mathematical model. In other words, formation of several plastic deformation mechanisms has caused the structural model either to suffer from lack of redundancy (in case no material hardening is introduced to the model) or to have such small stiffness that solving the equilibrium equation even for a small increment of forces is very difficult. Incorporating this criterion for determination of structural capacity significantly relies upon the robustness of incorporated mathematical model and solution algorithms. This implies that, dynamic analysis algorithm must be able to accurately track abrupt changes in structural response that are as the result of nonlinearities associated with the model.

Methodology of This Article

The incorporated solution algorithm in this study, involves recursive shortening of analysis time-step in critical situations of structural model during time-history analyzes. Using this strategy the numerical instability of model was postponed until collapse criteria was governed by excessive response rather than non-convergence criterion. Through this, we could increase the robustness of collapse criterion with cost of increasing run-times. For performing an IDA one of the most important issues is suitable selection of intensity and demand measures. A better selection of IM can considerably reduce dispersion of predicted capacity; rather, election of DM must be based on the dominant failure mechanism of building. In this study, representative parameters for IM and DM are 5% damped first mode spectral acceleration ($S_a(T_1,5\%)$) and the maximum inter-story drift angle (θ_{max}), respectively. In order to establish an automated still time-efficient procedure for performing IDAs, the "hunt and fill" algorithm (Vamvatsikos 2002) is utilized. The maximum inter-story drift angle θ_{max} of 10%, along with reaching a slope equal to 20% of elastic region are chosen as collapse criterion. Numerical non-convergence (as has not governed collapse criterion) is not considered for determining collapse state. For tracking the immediate occupancy limit state, θ_{max} value is limited to the value of 2%.

Ground Motion Selection

A suit of 40 ground motion records selected by Medina (2004) are used in this study. Ground motion records are comprised of two bin strategies: Large Magnitude-Short Distance bin, LMSR, ($6.5 < Mw < 7.0$, 13 km $< R < 30$ km), and Large Magnitude-Long Distance bin, LMLR, ($6.5 < Mw <7.0$, 30 km $\leq R \leq 60$ km). Details of these ground motions are not included here for brevity.

IDA Results

IDA results for all 40 records along with summarized curves are depicted in Fig. 4. Median IDA curves of all buildings are compared in the same figure. As expected, with increase of building height, IDAs have a lower IM for the same DM value. This indicates that S_a capacity is lower for tall buildings compared to short buildings. The next step towards evaluation of failure probability (to meet the desired performance) of structures, irrespective of level of performance to be considered, is to calculate fragility curves. Fragility function gives the probability that any input ground shaking will cause failure to building at an IM value exceeding a specific value (complementary cumulative function). Collapse and IO-limit fragilities for all 3 buildings are illustrated in Fig. 5. The S_a values corresponding to 16%, 50%, and 84% failure probabilities for different buildings are tabulated in Table 1. These quantities are valuable for specification of design earthquakes considering different failure probabilities. They may also provide a critical statement about the correctness of suggested values by pertinent provisions. However evaluating these concepts is beyond the scope of this article and requires a separate study.

To investigate drift distribution over buildings height in different intensity levels, the median value of maximum story drift is computed over all records at three different S_a levels for each story. These S_a levels are those corresponding to 16%, 50%, and 84% collapse probability evaluated separately for each structure. The results are shown for all floor levels in Fig. 6 and Fig. 7. While illustrated curves show very similar trends for 8- and 16- story buildings, the distribution of drift over buildings heights is different for 4-story building. For this structure, story drift has a relatively linear distribution over building height, whereas for two taller buildings the roles of 2nd and 3rd floors are distinctly dominant. Regarding this observation, it is further clarified that the failure mechanism of tall buildings is governed by accumulation of p-delta effects in one or more stories. In literature this phenomenon is called "soft story formation." As depicted by the IDA curves in Fig. 4, collapse state of 8 and 16 story buildings, in most cases, is preceded by a flat segment in IDA curves, though, this is not the case for 4-story building. This property is again attributed to the formation of soft story in tall buildings that causes the failure mechanism of these buildings to be similar to buckling failure mode of a slender column.

Conclusions

In this study a RBS connection subassembly was established defining a nonlinear panel zone element as well as degrading rotational spring elements representative of RBS critical section. This model was subsequently validated versus experimental results (Engelhardt 2000). Utilizing this assemblage, three buildings representative of different heights, were modeled and analyzed featuring fully nonlinear assumptions for materials and geometry. These frames were subjected to IDA procedure for a suit of 40 ground motion records. The outcomes of analyzes were used for estimating buildings performance in terms of S_a capacities for two performance levels CP and IO. The S_a values corresponding to 16%, 50%, and 84% failure probabilities for different buildings were tabulated. These values can be used in future studies for either specification of design earthquakes considering different failure probabilities, or as a critical statement about the correctness of the suggested values by pertinent provisions. Furthermore, statistical interpretation was made of buildings responses over their height. The median values of maximum story drift at three different IM levels were extracted over all 40 time-history analyzes. Failure mechanism of buildings with respect to distribution of drift over their heights was also assessed.

Table 1. S_a values corresponding to different failure probabilities

Failure Probability	**4-story building**		**8-story building**		**16-story building**	
	IO	**CP**	**IO**	**CP**	**IO**	**CP**
16%	0.33g	1.77g	0.10g	0.55g	0.09g	0.35g
50%	0.47g	2.26g	0.15g	0.68g	0.16g	0.49g
84%	0.70g	4.2g	0.23g	1.04g	0.22g	0.71g

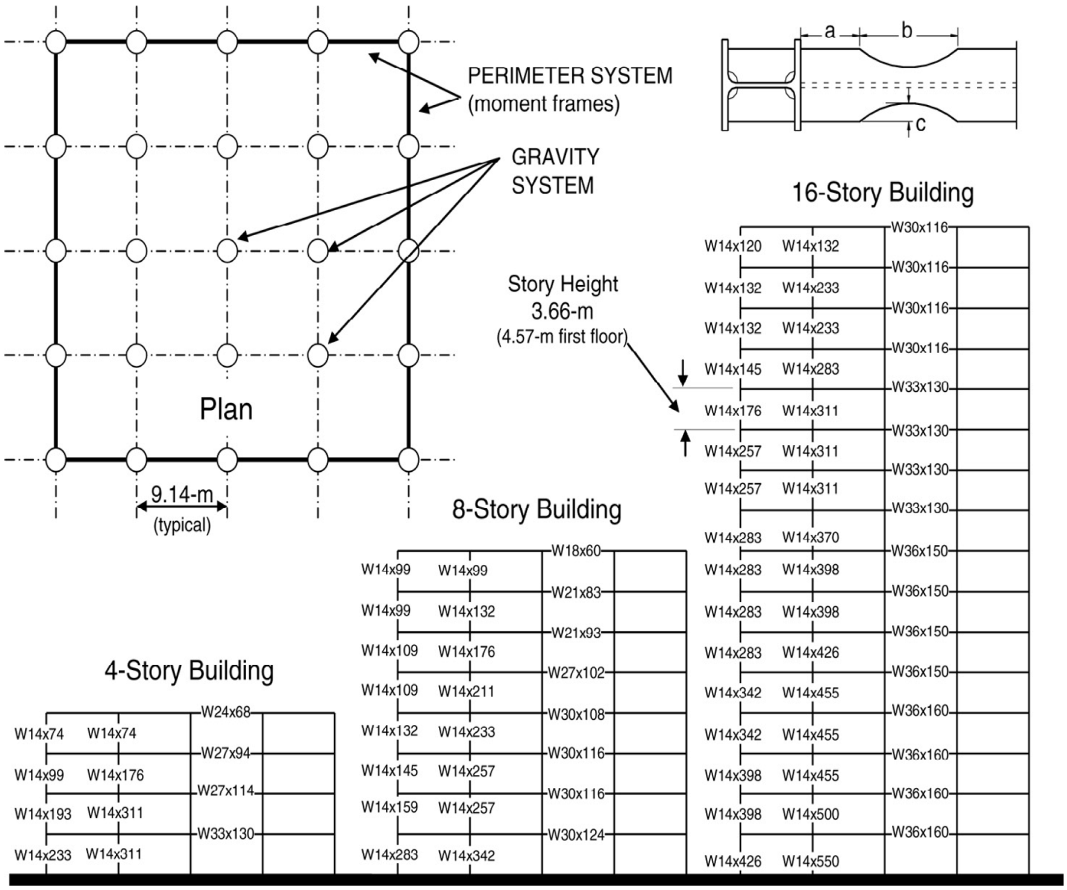

Figure 1. Design details of buildings (Jin 2002).

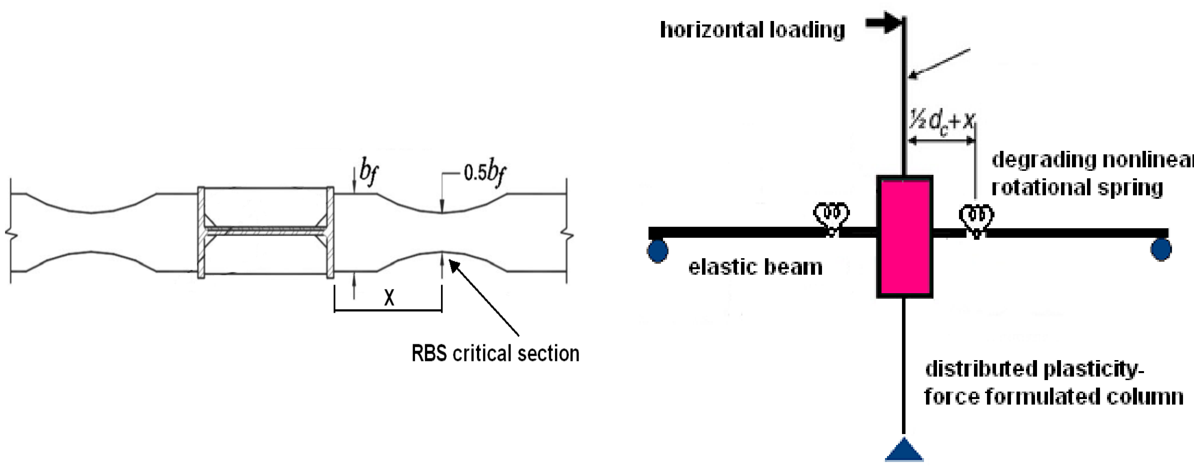

Figure 2. RBS connection and the cruci-form subassembly model.

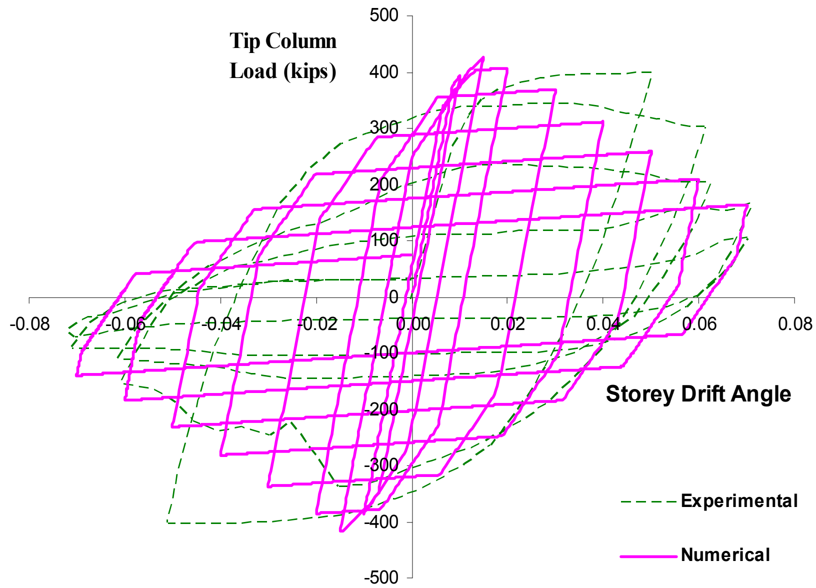

Figure 3. Experimental vs. numerical response (for specimen DBBWC of Engelhardt 2000).

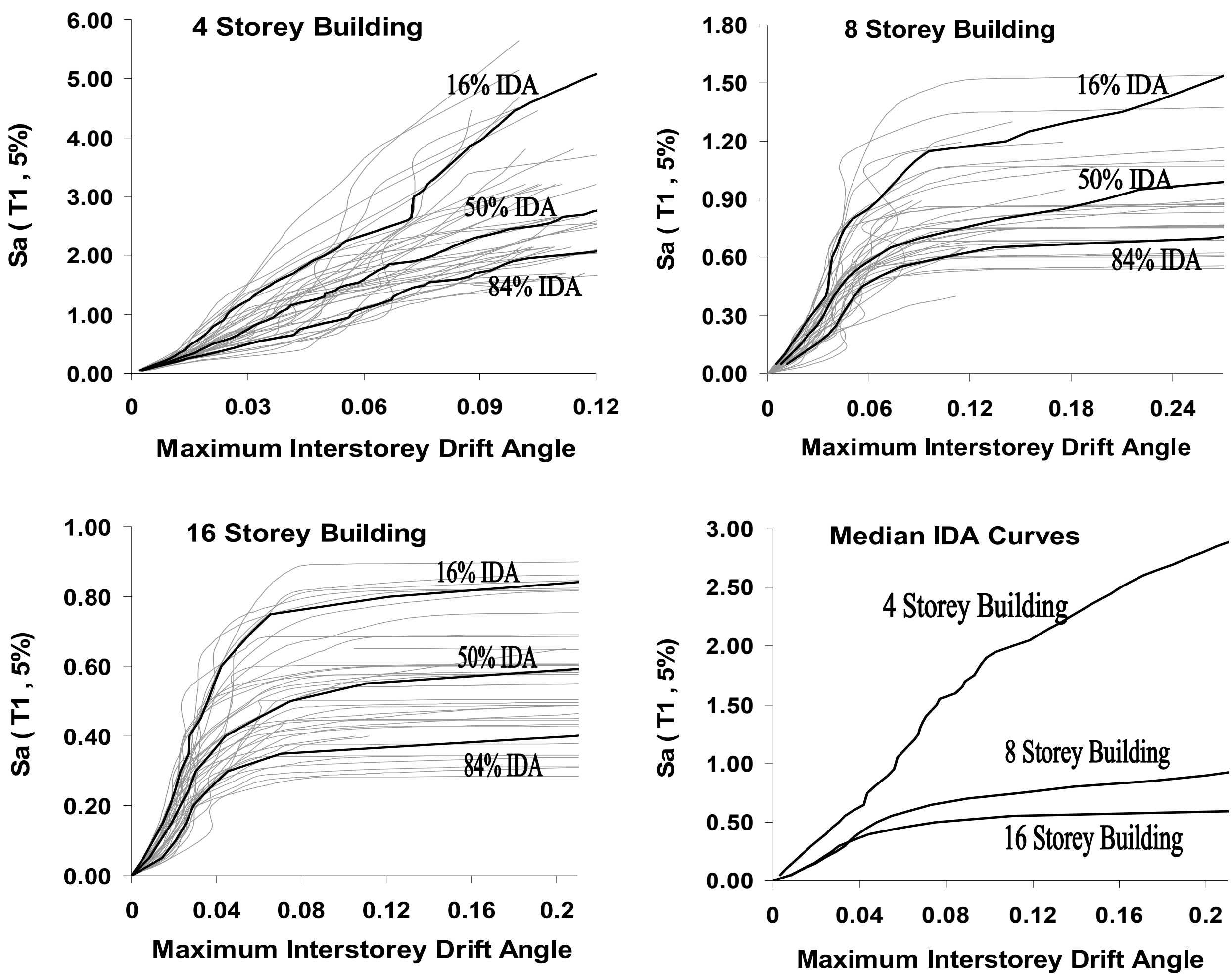

Figure 4. IDA results

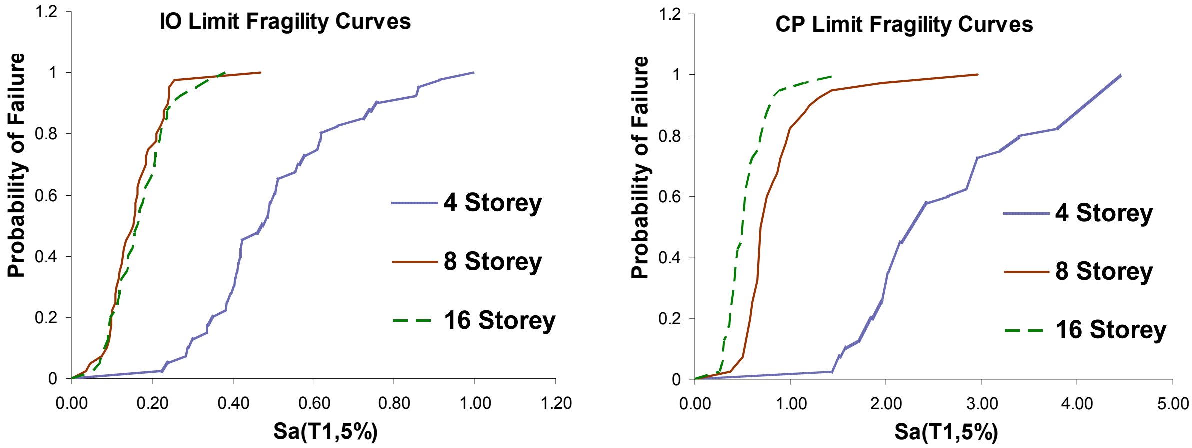

Figure 5. Fragility curves

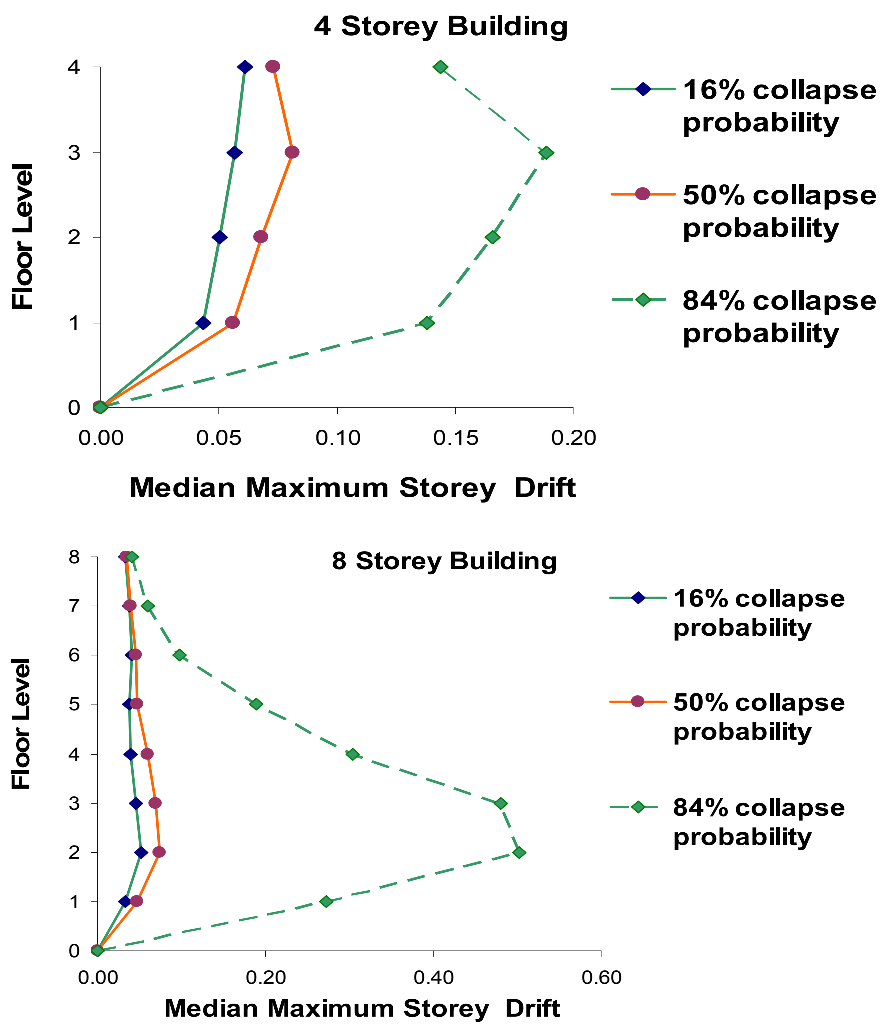

Figure 6. Median maximum story drifts over IDA results (4- and 8-story buildings)

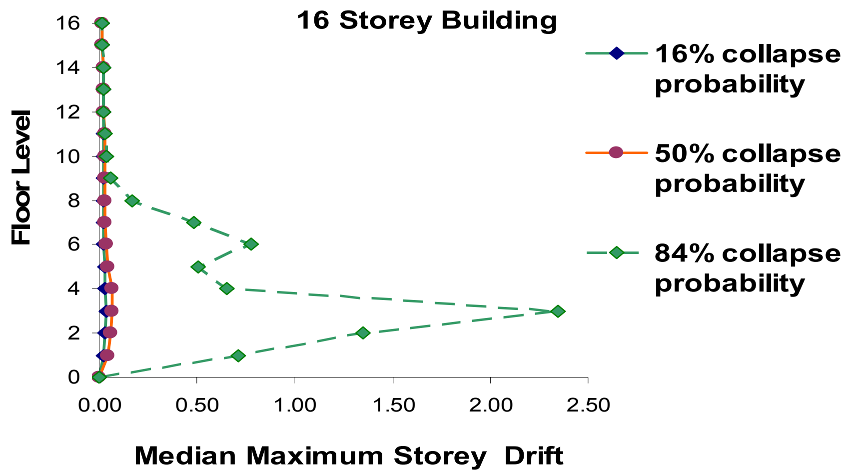

Figure 7. Median maximum story drifts over IDA results (16-story building)

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