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SYSTEM-LEVEL ACCEPTANCE CRITERIA FOR SEISMIC ASSESSMENTS OF PRE – 1970s REINFORCED CONCRETE BUILDINGS

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ABSTRACT

The objective of this ongoing research project is to determine system-level acceptance criteria for seismic assessments of pre 1970 reinforced concrete buildings. These system-level acceptance criteria could be based on strength or stiffness degradation considering the global performance of the structure. Current seismic rehabilitation standards, including ASCE/SEI 41 Supplement 1, are based on component acceptance criteria which do not consider the interaction between the components and the ability of the system to redistribute loads and resist collapse. Field observations of structures after severe earthquakes have revealed that intense damage to primary components has not necessarily resulted in structural collapse as would have been concluded by current seismic assessment procedures. A methodology similar to the one presented in FEMA 695 has been utilized in this research to perform probabilistic assessment of collapse risk for pre-1970s reinforced concrete frame structures. The first step is to determine collapse fragility curves based on global/interstory drifts of these non-ductile structures. Probabilistic analysis (sampling method) is used to determine the probability of collapse. Uncertainty modeling of the material and geometric parameters (directly represented in the probabilistic models for shear/axial failure) are considered in the analysis. Based on the probability of collapse at different drift ratios, the system-level acceptance criteria are extracted.

Introduction

Reinforced concrete frames constructed prior to the introduction of seismic provisions in modern building codes are susceptible to irrecoverable damage during lateral loads imposed by earthquakes. Modern seismic codes enforce ductile detailing of reinforcement and strong column – weak beam mechanism which older codes lack. As a result, several structures designed according to old provisions have undergone severe failures, including collapse. Rehabilitation plays an important role in order to prevent such disasters. In order to decide on the most appropriate and economical rehabilitation strategy for an existing structure and to design the rehabilitation system, it is necessary to assess accurately the lateral load resistance and the potential collapse modes.

Current rehabilitation standards, including the latest version of rehabilitation standards,

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ASCE/SEI 41 Supplement 1, assists engineers with the seismic assessment of existing buildings based on component acceptance criteria. In this standard the structure is first pushed statically to a specified target displacement to obtain the demands on structural components. After which, the component's demand is compared with its capacity curve (i.e. acceptance criteria) defined in the Standard. The comparison will define an exclusive performance level for each element, *immediate occupancy* (IO), *life safety* (LS), and *collapse prevention* (CP). The component with the highest performance level will define the state of the structure. As an example, the severe performance level of collapse prevention, which is also the performance state addressed in this study, of only one component will result in the system being considered to be in this collapse prevention state. In other words, component-based criteria do not take into account global behaviour and the capability of structures to redistribute gravity and lateral forces after one component reaches the collapse state. Field observations of structures after severe earthquakes (e.g. Sezen et al. 2000) have also revealed that intense damage to primary components has not necessarily resulted in structural collapse as would have been concluded by current seismic assessment procedures. Therefore, these component-based criteria lack the ability to differentiate system-level limit states based on the response of the critical component, and as a result, tend to err on the conservative side.

Recently published FEMA 695 report (previously known as ATC-63) titled "Quantification of building system performance and response parameters" introduces state-of-the-art research on building behaviour at the collapse limit state and quantification of this behavior for new design (FEMA 2009). A methodology similar to the one presented in FEMA 695 has been utilized in this study to perform a system-level probabilistic assessment of collapse risk for pre-1970s reinforced concrete structures. This different approach is required to resolve the technical issues in the FEMA 695 methodology and to expand the approach for older concrete structures, explained in detail in the next section. The current research focuses on concrete frame building structures. Although other systems are prevalent in the inventory of older concrete buildings, this class of structures has been shown to be particularly vulnerable to collapse in past earthquakes (Sezen et al. 2000).

Review of FEMA 695

FEMA 695 has recommended a methodology to reliably quantify building system performance. This report consists of a framework for establishing seismic performance factors (SPFs), such as force modification (R) and over strength (Ω_0) factors. The approach involves the development of detailed system design information and probabilistic assessment of collapse risk. By applying the incremental dynamic analysis (IDA) approach and probabilistic assessment of collapse risk the SPFs for a proposed system are established. The methodology only applies to the seismic-force-resisting system of new buildings. It utilizes nonlinear analysis techniques, and discretely considers uncertainties in ground motion, modeling, design, and test data (FEMA 2009).

FEMA 695 is a state-of-the-art method for system assessment and has highlighted many difficult technical issues in assessing the collapse performance of building structures. The current study will attempt to address some of these technical issues, specifically for existing reinforced concrete frames. These include: (1) incorporation of additional uncertainty by adjusting the collapse capacity due to the effects of spectral shape; (2) evaluation of non-simulated collapse modes by limit state checks without explicit consideration of uncertainty in the ability of models to capture the limit states; (3) focus on sidesway collapse as the only collapse mode for reinforced concrete frames; and (4) discretely considering uncertainty in ground motion, modeling, design, and test data.

In the incremental dynamic analysis (IDA) approach, response spectra of selected ground motion records are scaled in order to reach the collapse state. For rare ground motions the spectral shape is different from the uniform design spectrum. In order to compensate for this difference, an adjustment factor is applied to the collapse margin ratio, CMR (the ratio of the median spectral acceleration of the collapse level ground motions to the spectral acceleration of the MCE ground motions at the fundamental period). This modification factor depends on the period and the ductility capacity of the structure. The simplified spectral shape factor will adjust the collapse margin ratio and, as a result, may lead to an increase in the (aleatory) uncertainty associated with the collapse capacity.

In the approach recommended in FEMA 695, collapse modes are assessed through either explicit simulation in the nonlinear analyses or evaluation of non-simulated collapse modes using alternative limit-state checks on demand quantities from the nonlinear analyses. In non-ductile reinforced concrete frames, shear failure and subsequent axial failure of concrete columns are considered as non-simulated collapse modes and are dealt with by limit-state checks. These limit-state checks will generally result in a low estimate of the median collapse point, because non-simulated collapse modes are usually associated with a component failure mode. The inherent assumption is that the first occurrence of this non-simulated failure mode will lead to collapse of the structure, which may not always be the case. Furthermore, the impact of the non-simulated collapse modes on the rest of the structure is not directly accounted for in the analysis. For this reason, local failure modes should also be directly simulated in order to more accurately reflect the behaviour of the structure.

In incremental dynamic analysis, sidesway collapse is the governing mechanism, and collapse prediction is based on dynamic instability or excessive lateral displacements, (i.e. the primary expected collapse mode is flexural hinging leading to sidesway collapse). However, in non-ductile reinforced concrete frames, it is expected that columns will frequently lose the capacity to support gravity loads due to shear and axial load failures prior to the development of flexural mechanism necessary for a sidesway collapse, and as a result, they will likely form other collapse mechanisms (e.g. loss in vertical load carrying capacity collapse).

The recommended methodology described in this paper for existing concrete frame structures, constructed before the introduction of modern seismic codes, is a refinement of the aforementioned methodology introduced in FEMA 695. In the current study it is assumed that the collapse of such structures is mainly due to the degradation of axial capacity after column shear failure. This dominant non-ductile collapse mode has not been directly implemented in FEMA 695. The methodology introduced in this paper has the main objective of resolving the four primary deficiencies noted above when FEMA 695 is applied to existing concrete buildings.

Methodology

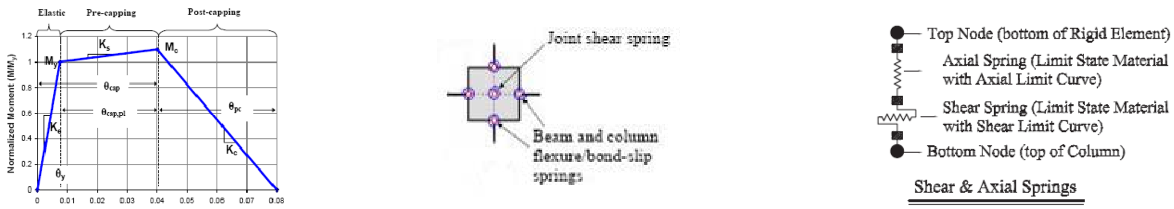
The main objective for this ongoing research is to determine system-level acceptance criteria for the seismic assessment of pre-1970s reinforced concrete buildings. This paper outlines the general framework of the methodology and describes the overall process. In this methodology nonlinear analysis is coupled with probabilistic methods to provide system-level criteria which, when used in the seismic assessment of existing concrete frame buildings, can provide the seismic demands expected to lead to a selected probability of collapse. It is our goal that these system-level acceptance criteria could be integrated into an ASCE 41-type assessment procedure, enabling practicing engineers to consider the overall system response when evaluating the "Collapse Prevention" performance level.

This paper introduces the key elements of the methodology, including numerical analysis of reinforced concrete frames based on probabilistic shear and axial-failure models, development of probabilistic analysis to determine the probability of collapse, definition of uncertainty of the material and geometric parameters (directly represented in the probabilistic models of shear and axial failures), and coupling of the resulting fragility curves with the pushover response to investigate trends between the probability of collapse and the global behaviour of the structure. The numerical analysis includes two main features: 1) the non-ductile behavior expected in older frame buildings is directly included in the simulation, and 2) consider collapses other than sidesway collapse. Moreover, probabilistic analyses are conducted to explicitly incorporate uncertainty in the numerical model in contrast to the discrete approach adopted in FEMA 695. These elements are specified in more detail in the sections that follow.

Numerical Analysis

The first step in this methodology is to perform a detailed and robust nonlinear analysis on the non-ductile reinforced concrete frame. Accurate modeling of inelastic behavior in beam and column elements is an essential component of collapse modeling of these structures. Non-ductile behaviour originating from column shear and subsequent axial failure plays an important role in these structures; and the analytical model must have the ability to capture this behaviour. Overall the model should encompass the following key features:

- The models should be able to capture post-peak behaviour which will affect the collapse response of these structures. Capturing post-peak response requires modeling the strain-softening behavior associated with flexural response such as concrete crushing, rebar buckling and fracture, and/or bond failure (e.g. Figure 1a).
- To account for the degradation of strength and stiffness associated with large deformations, the analysis should utilize suitable geometric transformations to take into account P-Δ effects.
- Although recent tests on non-ductile RC frames have shown that collapse of such frames is less likely to occur due to joint failure than column failure (Yavari et al. 2009) , in order to capture the degradation in strength and stiffness originating from possible joint softening, a joint model (e.g. 2D joint model developed by Lowes and Altoontash 2003) should be included which will account for non linear shear deformations of the joint and bond-slip behavior (Figure 1b).
- Shear and axial failures should be modeled by probabilistic "Limit State Material" models which will detect the respective failures using drift capacity models (Koduru et al. 2007).These models define shear and axial failures as a function of inter-story drift, as well as geometric, material and design parameters (Figure 1c).



(a) Flexural model in able to capture the post – peak behaviour (Ibarra 2005)

(b) Joint model in able to capture the shear panel and the bond slip behaviour (Liel 2008)

(c) Limit state material in able to capture the non-ductile behaviour (Elwood 2003)

Figure 1. Detailed numerical model utilized to simulate collapse behaviour.

Numerical analysis always involves a balance between complexity to achieve accuracy and simplicity to ensure efficiency of the analysis. For example, choices which must be made in this study between 2D and 3D models and between simplified static pushover analysis and detailed nonlinear dynamic analysis.

When an older concrete structure is laterally deformed, local failure is likely to occur in vertical load-carrying components (e.g. non-ductile columns) as a result of shear and subsequent axial failure. As the gravity loads carried by these elements transfer to neighboring elements, the local failure can propagate until the structure reaches a state where it loses its ability to resist the gravity loads. This state is referred to collapse. This progression of damage can be tracked on a pushover curve through strength and stiffness degradation of the structure mainly as a result of component strain-softening (ductile) and non-ductile failure in addition to second-order P-delta effects. In numerical simulation, there is always the challenge of defining the collapse state during the analysis and differentiating structural collapse from numerical non-convergence. In collapse analysis used in FEMA 695, the criterion is based on maximum interstory drift ratios and there is an assumption that the structure is ductile enough to reach this state. However, because of the limited ductility of older reinforced concrete structures, the collapse state should be defined with different criteria. One possible criterion for gravity-load collapse of reinforced concrete frames sustaining shear and axial failures could be when the total axial capacity for a floor drops below the total gravity load supported by the structure at the respective floor (Elwood 2008). This definition of collapse will be adopted in the current study. The selection of the collapse criteria can significantly alter the final results.

Probabilistic Analysis

Reliability analyses are utilized in order to determine the probability of collapse of a prescribed performance objective considering a limit-state function, uncertainties associated with each random variable, and using a suitable method to perform the analysis. The reliability method could be coupled with the numerical analysis to evaluate the probability of collapse for a selected structural performance state, such as collapse. In this methodology, all essential uncertainties are explicitly accounted for in the probabilistic analysis. This approach contrasts to that used in FEMA 695 where uncertainties are discretely accounted for and combined at the end of the approach to obtain the total uncertainty.

Uncertainties are inherent in material properties, geometry, loading and modeling of the structure. Different reliability methods could be applied to perform this task, but due to technical difficulties arising when applying the first- and second-order (FORM and SORM) reliability methods to nonlinear models, the sampling method is considered a more appropriate approach in this methodology. Using the distributions defined for each random variable, realizations of each random variable is generated and inputted into the numerical model. This process is repeated for thousands of sets of realizations and the result for each simulation is used to obtain the collapse fragility defined through a cumulative distribution function (CDF). These CDF curves relate the different response measures to the probability of collapse. In this methodology the uncertainty for each random variable is explicitly considered in the analysis and reflected in the final probabilities of collapse. The random variables selected with the respective probability distribution should have the capability of capturing the major uncertainties inherent in non-ductile reinforced concrete frames. Uncertainty in the shear and axial failure models for non-ductile columns are considered the main sources of uncertainty in this study.

Response Measures

The last step in this approach is to generate cumulative distribution functions (CDF curves) for different response measures. The selected response measures should provide insight on the response of the system as a whole, in order to facilitate the selection of system-level acceptance criteria.

The methodology should be applied to a range of different concrete-frame buildings to seek trends between the values of response measures resulting in large increases in the probability of collapse. The methodology is applied to one such frame building below, one frame of a four-story four-bay non-ductile concrete space frame, to elucidate the process in detail.

Example

The non-ductile reinforced concrete frame structure, designed for a study by Liel and Deierlein (2008) and shown in Figure 2, is used as a prototype to demonstrate the methodology presented in this paper. The structure is fully designed according to the requirements of the 1967 UBC (ICBO 1967). The requirements include maximum and minimum reinforcement ratios, maximum stirrup spacing, and requirements on hooks, bar spacing and anchorage, etc.

Numerical Model

The numerical analyses are conducted using a three-bay two-dimensional frame modeled in OpenSEES, an open-source, object-oriented software platform developed at the Pacific Earthquake Engineering Research Center (PEER 2006). For simplicity, only the lateral resisting system is modeled, neglecting the contributions of elements designed primarily for gravity loads or nonstructural elements. The analytical model incorporates all the important features required for collapse analysis as explained in the methodology.

The numerical model of the 4-story building is subjected to static pushover analysis, to determine the global response of the structure. Figure 3 includes the results for two different models: first the results for the complete model briefly explained above (labeled as limit state material) and the results for a model which does not include the shear and axial zero-length springs, similar to the models employed in the FEMA 695 approach (labeled as w/o limit state material). This figure shows that the drift at full shear strength degradation for the model with non-ductile material is almost half the corresponding drift of the "ductile" model without limit-state materials incorporated.

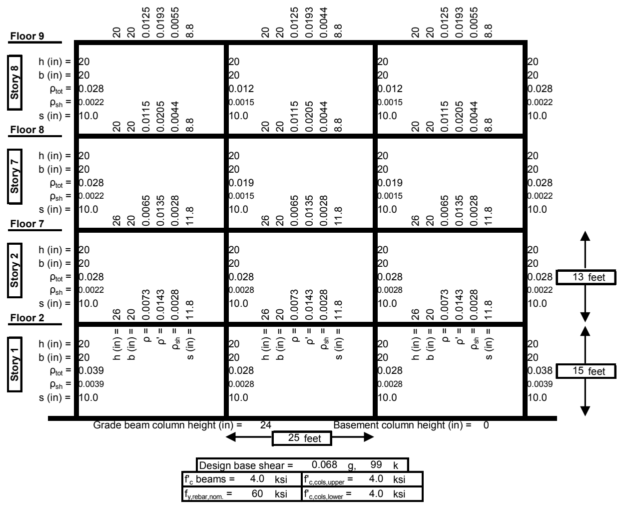


Figure 2. Four storey non – ductile RC concrete frame (Liel and Deierlein 2008).

Probabilistic Analysis

The random variables chosen to represent the uncertainty in this non-ductile frame and incorporated in the probabilistic analysis are listed in Table 1. Past research has indicated that modeling uncertainties associated with damping, mass, and material strengths have a relatively small effect on the overall uncertainty in seismic performance predictions, and will primarily have an influence on pre-collapse performance of structures (Lee and Mosalam 2005). In addition, Ibarra and Krawinkler (2003) have shown that the uncertainty associated with modeling deformation capacity and post-peak softening response of component element models can have a significant influence on the predicted collapse performance. Therefore, the parameters chosen as random variables in this example are related to modeling uncertainties associated with component deformation capacity and other parameters critical to collapse prediction of reinforced concrete (RC) moment frame buildings.

Response Measures

The last step is to construct CDF curves for different response measures using the sampling method. The selected response measures include maximum interstory drift ratio, roof drift ratio, first story drift ratio, strength degradation, stiffness degradation and maximum number of columns which sustain shear and subsequent axial failure for each story. The CDF curves for the aforementioned response measures obtained for the example building frame are

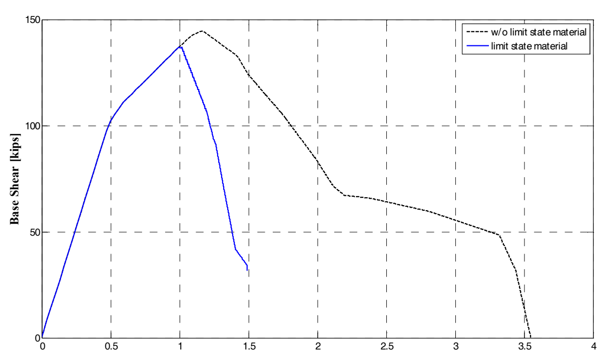


Figure 3. Static pushover curve for the 4-story reinforced concrete building (the response at the median realization of all random variables).

Table 1. Uncertainty modeling of the random variables.

			Random Variables
Material	Flexural Model	Lumped Plasticity Model	$EI_{elasric}$, M_y, M_c, θ_{cap}, θ_{pc}
	Non-Ductile Model	Shear & Axial Model	Coefficients in the probabilistic capacity models in (Koduru et al. 2007)
		Shear Panel	$EI_{elasric}$, M_y
Geometrical	Frame Properties		b,h (column and beams)

the selected response measures increase. In each of these figures, the probability of collapse has a sudden increase in a limited range of the response measure (e.g. 0.026 – 0.029 maximum interstory drift in Figure 4b) which highlights the dominant behaviour of the building frame for that response measure. These values could be used as trial values for the system-level acceptance criteria (Table 2a). CDF curves without consideration of axial failure have been also included in Figure 4 (labeled as lateral load carrying capacity). In these models, collapse is defined as the stage when the base shear resistance is reduced to zero, that is complete loss of lateral load carrying capacity.

The CDF curves could also be coupled with the pushover curve (the response at the median outcome of all random variables) to track the system behaviour for a chosen probability of collapse. For this matter Figures 3 and 4 (a) are coupled and the result is illustrated in Figure 5. In this figure, a preferred 10% probability of collapse (CDF = 0.10 on the CDF curve) can be tracked on the pushover curve. The system behaviour at this point can be used for a system level acceptance criteria. These values could also be used as trial values for the system-level acceptance criteria (Table 2b). This example suggests that a system-level acceptance criteria (based on this specific example) could be a global drift of 1%, a maximum interstory drift of 2.35%, shear strength degradation divided by peak strength of 0.04%, or maximum number of shear failures in each story should be limited to one column.

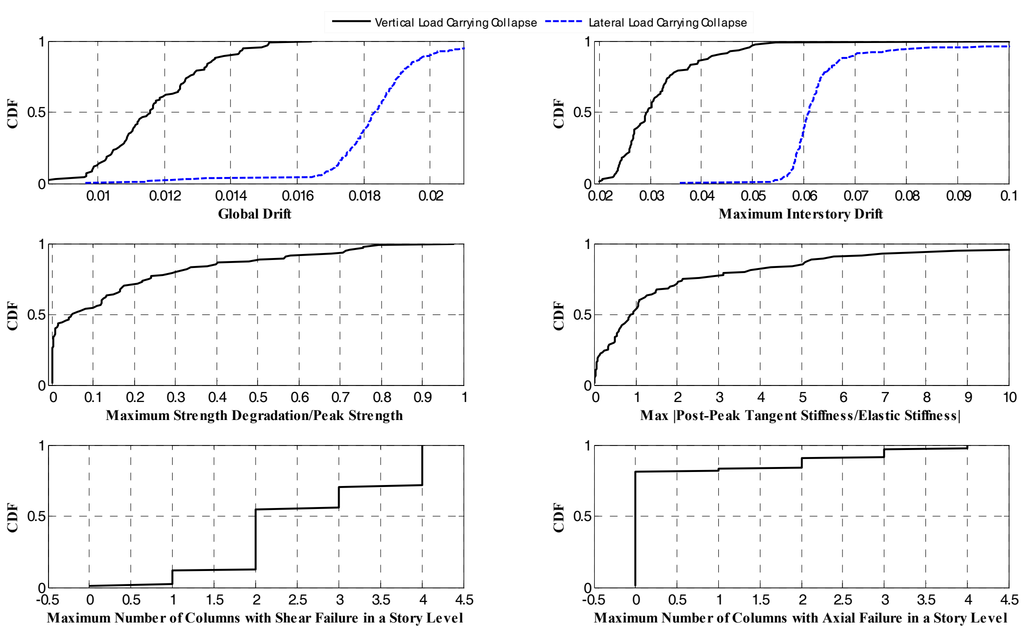


Figure 4. CDF vs. Response Measures.

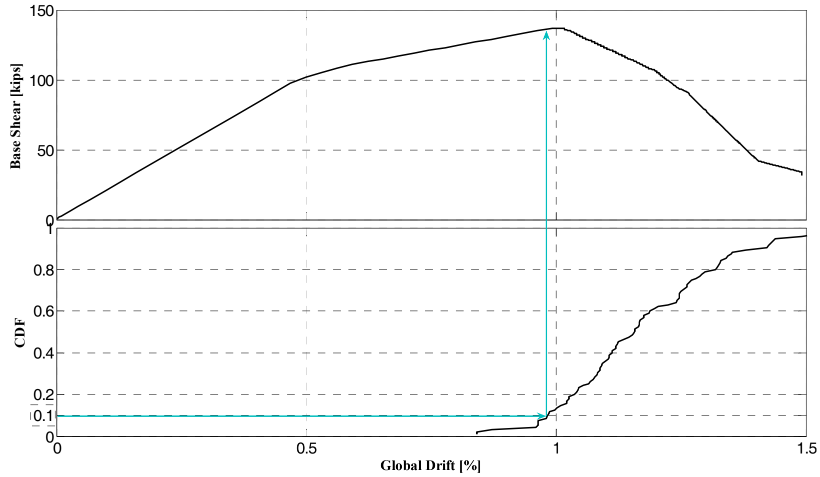


Figure 5. CDF curve (for global drift) coupled with the median pushover curve.

This example is only included here as an illustration. System-level acceptance criteria can only be selected after evaluating many different frame buildings and seeking trends in criteria similar to that listed in Table 2.

Summary

The ongoing research described above has highlighted the methodology required to select system-level acceptance criteria, which can be used to perform seismic assessment on pre-1970s reinforced concrete structures. However, the procedure is still in its early stages of development and needs to be applied to several other reinforced concrete frame buildings before appropriate system level acceptance criteria can be selected. The methodology for accomplishing the objective consists of three steps; detailed nonlinear analysis, probabilistic analysis, and coupling the results from the first two steps to identify the system-level criteria. An example is presented to explain the methodology and demonstrate its application for non-ductile reinforced concrete frames. The result is different system-level criteria for this structure including global/interstory drift, strength degradation and maximum number of column shear failures in each story.

Acknowledgment

The four-storey reinforced concrete frame model utilized in the example in this study was generated by Dr. Abbie Liel during her PhD studies at Stanford University. The authors are grateful for this generous contribution. The research described herein was made possible through financial support from the Natural Sciences and Engineering Research Council of Canada. This support is gratefully acknowledged.

Table 2. Trial system-level criteria (based on the example).

(a)		(b)	
Global Drift [%]	1.1 – 1.2	Global Drift [%]	1.0
Maximum Interstory Drift [%]	2.6 – 2.9	Maximum Interstory Drift [%]	2.35
Shear Strength Degradation / Peak Base Shear [%]	0.1	Shear Strength Degradation / Peak Base Shear [%]	0.04
Max |Post-Peak Tangent Stiffness / Elastic Stiffness| [%]	2.0 – 8.0	Max |Post-Peak Tangent Stiffness/ Elastic Stiffness| [%]	3.0
Maximum # of Shear Failure in each floor	2	Maximum # of Shear Failure in each floor	1

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