

Changing the Paradigm for Performance-Based Design¹

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Summary

The principal investments in building construction are made in non-structural components and contents (NCCs). An efficient performance-based design paradigm should focus on these key investments and a new design paradigm is needed in order to do so. Structural framing systems should be selected on the basis of the required performance of NCCs. Protective systems appear to offer significant advantages over traditional framing systems in terms of both smaller median demands and smaller dispersion in demand for acceleration- and displacement-sensitive NCCs. The impact of structural framing system type on the NCCs demands is illustrated through response-history analysis of a 1960s-era hospital building located in Southern California.

Introduction

To date, tools for performance-based earthquake engineering have focused on performance *assessment* of structural framing. Only modest attention has been paid to *assessment* of nonstructural components and contents (NCCs) and to the development of tools for *design* of structural framing and NCCs.

HAZUS (NIBS 1997) provides important information on the financial importance of NCCs in a wide variety of building structures. Figure 1 displays the average percent investment in structural framing, nonstructural components and building contents for three types of building structures:

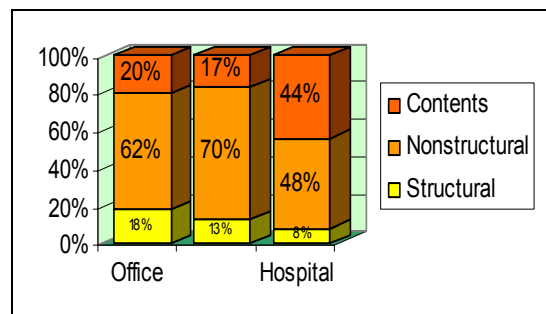


Figure 1. Investments in building construction (after E. Miranda)

office, hotel and hospital. In all cases, the investment in the structural framing is less than 20% of the total investment, and the percent investment in hospital construction is a mere 8% of the total. If a goal of performance-based earthquake engineering is to protect financial investments by minimizing total cost (including construction cost, annual maintenance cost and annualized earthquake-damage-related cost), close attention must be paid to those parts of a building in which the greatest investment is made.

Updating the design paradigm

Traditionally, structural engineers have paid scant attention to NCCs because their design and detailing had not formed part of the structural-engineering scope of work. In those cases where structural engineers have designed and detailed NCCs, the components have been analyzed and designed (albeit indirectly) for the output of the structural framing. We contend that such an approach is inappropriate and that the performance-based design process should focus first and foremost on the most significant investments in the building, namely, the nonstructural components and contents. A preliminary design of the framing system (framing layout, system type, material, etc), should be selected considering both structural and nonstructural components and then analyzed for *performance capability*. If the computed performance is unacceptable, the design of the structural and nonstructural components is revised and then re-analyzed for performance capability. Assuming that fragility functions are developed in sufficient number and detail to characterize the vulnerability of NCCs for common building occupancies, guidance will be required to assist the structural engineer to select the structural system type (incl. material, seismic framing system, strength, ductility) that will deliver the intended building performance.

Studies are under way at the University at Buffalo to aid in the identification of optimal structural framing systems, noting that the optimal solution will vary as a function of the performance objectives. Weak and flexible, strong and stiff, and *protected* framing systems are being studied. A hospital structure was chosen for the baseline building because of the high value (measured as a percentage of the total investment) of the nonstructural components and building contents in such buildings (see Figure 1). Sample *preliminary* results from these studies are presented in the following sections with emphasis on demands on acceleration- and drift-sensitive NCCs.

Assessment of an acute care facility

To illustrate the impact of structural-system choice on the response of acceleration- and drift-sensitive NCCs, a four-story mathematical model was developed based on the MCEER demonstration hospital. The lateral-load resisting system is composed of perimeter steel moment-resisting frames. The column bases and beam-column connections were modeled as rigid in all MRFs and as pin connections in all non-MRFs.

Protection of structural framing systems against gross damage during severe earthquake shaking motivated the initial development (in the 1970s and 80s) and implementation (1980s) of seismic protective systems: seismic isolation bearings and supplemental passive damping devices. Hospital buildings were early candidates for the use of protective systems because of the need to maintain hospital function after a major earthquake: essentially eliminating damage to the structural framing. Nowadays, seismically isolated buildings are designed to restrict substantial (or all) inelastic action to the isolators in maximum capable earthquake shaking. Buildings incorporating supplemental dampers are designed typically to restrict substantial inelastic action (damage) to the damping devices in design and maximum earthquake shaking and thus to eliminate damage to components of the gravity-load-resisting system.

Fifteen mathematical models representing different traditional and *protected* lateral-force-resisting systems were developed in the OpenSees software environment (<http://opensees.berkeley.edu/>) for analysis and evaluation. The 15 models are summarized in Table 1; the baseline model was M3. The *traditional* framing systems are M3 and M6: moment-resisting frames. Buckling restrained braces (BRBs), displacement-dependent dampers, were implemented in M7. Fluid viscous dampers (FVDs),

velocity-dependent dampers, were implemented in M8 and M9. Models M10 through M13 include linear viscoelastic seismic isolation bearings: one mathematical model used for low- and high-damping rubber bearings. Models M14 and M15 include bilinear seismic isolation bearings: the mathematical model used typically for lead-rubber and Friction Pendulum™ bearings. Much additional information will be available in Astrella (2004).

Table 1. Description of mathematical models

Model	Description	T_1^1 (secs)
M1	Baseline model of 1970s in-situ building; designed for the strength and drift limits of the 1970 Uniform Building Code; best-estimate model for <i>non-moment</i> -resisting connections.	0.70
M2	Similar to M1 except rigid connections used for <i>non-moment</i> -resisting connections.	0.68
M3	Similar to M1 except pinned connections used for <i>non-moment</i> -resisting connections.	0.71
M4	1960s variant of M1: design drift limits of M1 not imposed.	1.74
M5	Similar to M4 except rigid connections used for <i>non-moment</i> -resisting connections.	1.58
M6	Similar to M4 except pinned connections used for <i>non-moment</i> -resisting connections.	1.81
M7	M6 augmented with buckling restrained braces (BRBs) to provide approximately a 300% increase in lateral stiffness. BRBs installed in paired diagonal braces in the exterior bays on grid lines B and H.	0.97
M8	M6 equipped with fluid viscous dampers (FVDs) to provide approximately 25% of critical damping in the first mode. FVDs installed in paired diagonal braces in the exterior bays on grid lines B and H.	1.81
M9	M6 equipped with fluid viscous dampers (FVDs) to provide approximately 40% of critical damping in the first mode. FVDs installed in paired diagonal braces in the exterior bays on grid lines B and H.	1.81
M10	M3 equipped with viscoelastic seismic isolation bearings; isolated period is 2.5 seconds; approximately 10% of critical damping in the first mode.	2.60
M11	M3 equipped with viscoelastic seismic isolation bearings; isolated period is 2.5 seconds; approximately 20% of critical damping in the first mode.	2.60
M12	M3 equipped with viscoelastic seismic isolation bearings; isolated period is 3.5 seconds; approximately 10% of critical damping in the first mode.	3.57
M13	M3 equipped with viscoelastic seismic isolation bearings; isolated period is 3.5 seconds; approximately 20% of critical damping in the first mode.	3.57
M14	M3 equipped with coupled bilinear seismic isolation bearings: $Q_d = 0.06W$; second-slope isolation period is 2.5 seconds; isolator yield displacement is 25 mm.	2.60 ¹
M15	M3 equipped with coupled bilinear seismic isolation bearings: $Q_d = 0.06W$; second-slope isolation period is 3.5 seconds; isolator yield displacement is 25 mm.	3.57 ¹

1. First mode period in transverse (short) direction.

2. Period calculation based on second slope (post-yield) isolator stiffness.

Preliminary results are presented in this paper for 11 of the 15 models: M3, M6, M7, M8, M9, M10, M11, M12, M13, M14 and M15. Seismic demands on NCCs in the 11 buildings was assessed by nonlinear response-history analysis in the transverse (north-south) direction only. The earthquake histories used for the response-history analysis were those generated for a NEHRP Soil Type S_D (firm soil) site in Los Angeles as part of the SAC Steel Project (Somerville et al. 1997). Three bins of 20 histories were developed, each representing a different probability of exceedance (2% in 50 years, 10% in 50 years and 50% in 50 years).

Sample results from the response-history analysis are presented in Figures 2 through 5 for the 10/50 motions. Figure 2 presents a summary of the maximum drift responses for the 11 models noted above. Median, maximum, minimum, 16th percentile and 84th percentile results are presented assuming that the maximum responses are lognormally distributed. Drift data (relative displacement as a percentage of height) are presented for the 2nd story and the roof.² The horizontal axis in each subplot denotes the model number (e.g., M3 per Table 1). The yield drift in each story for M3 and M6, based on nonlinear static analysis, are shown in the subplots of Figure 2 to identify the degree of inelastic action (damage) in the non-isolated building frames. The trends of Figure 2 are well established, namely, that adding lateral stiffness, viscous damping and seismic isolation reduce interstory drift. As expected, the drifts in the isolated frames (M10 through M15) are substantially smaller than those in the *traditional* frames (M3 and M6) and the frames equipped with supplemental damping devices (M7, M8 and M9). The addition of the displacement and velocity-dependent dampers led to significant reductions in the median maximum displacement response of the weak and flexible frame (M6). Based on median response data, a) the traditional moment frames (M3 and M6) each sustained structural damage; b) damage in the building equipped with BRBs (M7) was limited primarily to the BRBs; and c) the viscous damped frames (M8 and M9) sustained negligible damage. For the non-isolated buildings (M3, M6, M7, M8 and M9), the coefficient of variation in the peak roof drift is greatest (0.33) for M6 (mean peak roof drift = 2.3 %) and smallest (0.28) for M9 (mean peak roof drift = 0.95 %). The addition of viscous dampers (M8 and M9) to the weak and flexible building (M6) reduced substantially the median maximum roof drift (by 44% for M8 and 56% for M9) and the coefficient of variation in the maximum roof drift (from 0.33 for M6 to 0.29 for M8 and 0.28 for M9).

Figure 3 summarizes the maximum *peak* floor acceleration responses at the 3rd and roof levels. Median, maximum, minimum, 16th percentile and 84th percentile results are presented assuming that the maximum responses are lognormally distributed. The trends seen in the four subplots of Figure 3 are also well established, namely, that adding lateral stiffness increases peak floor accelerations, and adding viscous damping or seismic isolation bearings reduce peak floor accelerations. For the non-isolated models, the coefficient of variation in the peak 2nd floor acceleration is greatest (0.37) for M7 (mean peak acceleration = 0.98 g) and smallest (0.30) for M8 and M9 (mean peak acceleration = 0.49 g). The addition of viscous dampers to the weak and flexible building (M6) reduced the median peak 2nd floor acceleration (by 29% for M8 and M9) and the coefficient of variation in the peak 2nd floor acceleration (from 0.32 for M6 to 0.30 for M9).

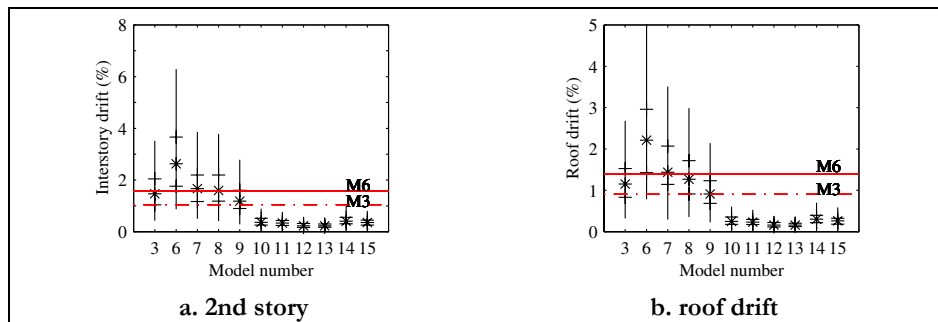


Figure 2. Maximum drift responses for 10/50 earthquake histories

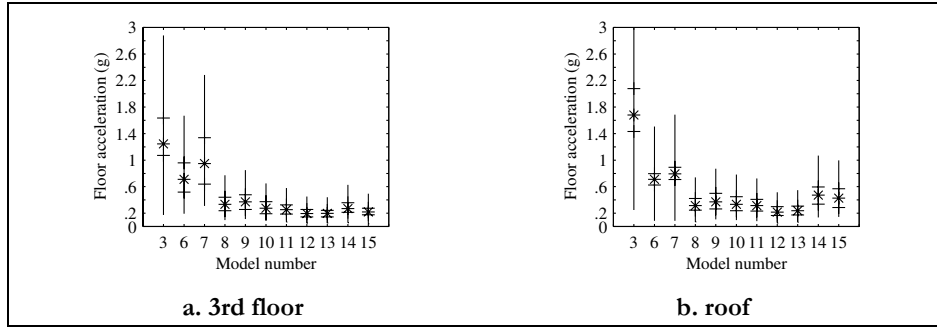


Figure 3. Maximum acceleration responses for 10/50 earthquake histories

In FEMA 273/356, the intersection of the median capacity (pushover) and median demand (hazard) curves is termed a *performance point*. Such a point, although instructive, provides no information on the impact of uncertainty and randomness on the capacity and demand calculations and by extension on the building performance. Reinhorn extended the concept of the performance point to a *performance space*, to account for both uncertainty and randomness in a rigorous manner. Figure 4 presents performance points using median maximum drift (ID*) and median peak floor acceleration (A*) as the performance metrics (ID* and A* are defined in the figure) and performance spaces as boxes defined by the 16th and 84th percentile maximum drift and zero-period floor acceleration responses. Alternate groupings of ID* and A* (e.g., A2/ID1) might be more appropriate for nonstructural components such as suspended ceiling systems. In terms of demands on NCCs, performance points adjacent to the origin and performance spaces that are small in size (indicating small variability in displacement and acceleration responses) are preferable to points remote from the origin and spaces that are large in size. On the basis of the chosen metrics, the performance of the buildings equipped with supplemental fluid viscous dampers or seismic isolation bearings is superior to that of the *traditional* moment-frame buildings or the building equipped with BRBs.

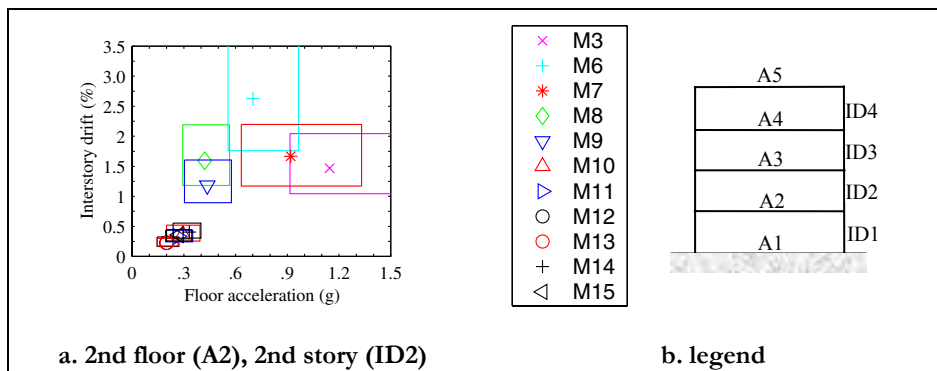


Figure 4. Performance points and spaces for 10/50 earthquake histories

On the basis of the data presented in Figure 4, the performance of the isolated buildings is superior to that of the other buildings in terms of smaller displacement and acceleration demands on NCCs. Of the remaining *traditional* and *protected* lateral-force-resisting systems, the buildings equipped with fluid viscous dampers (M8 and M9) outperform the remaining 3 buildings (M3, M6 and M7).

For many acceleration-sensitive NCCs, peak floor acceleration alone is an inefficient predictor of damage. Better estimates of the vulnerability of acceleration-sensitive NCCs can be developed through the use of floor (in-structure) acceleration spectra. Median 5% damped median floor acceleration spectra for the 2nd floor (A2) and 4th floor (A4) of the 11 models for the 10/50 earthquake histories are presented in Figures 5a and 5b. The stiff and strong moment frame building (M3) and the building equipped with BRBs (M7) produce the highest spectral acceleration demands across a frequency range from 1 Hz to 100 Hz. The smallest acceleration demands are associated with the viscous damped frames (M8 and M9) and the isolated frames (M10 through M15). Importantly, the spectral peaks of the moment-frame structures (M3 and M6) are suppressed through the addition of viscous damping: an observation reported first by Pavlou and Constantinou (2004).

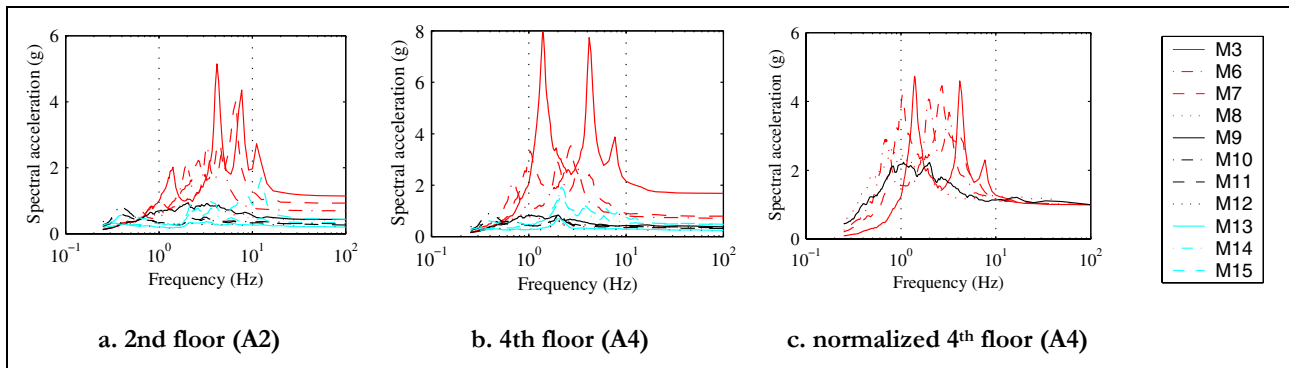


Figure 5. Floor acceleration spectra for 10/50 earthquake histories

Normalized 5-percent damped 4th floor acceleration spectra are presented in Figure 5c for the 5 non-isolated buildings (M3, M6, M7, M8 and M9). The figure shows the amplification of the peak floor acceleration as a function of structural framing system. For the same peak floor acceleration, the performance of the viscous damped buildings, M8 and M9, is clearly superior to the *traditional* moment-frame buildings and the building equipped with BRBs

Closing remarks

The next generation tools for PBEE will recognize the substantial financial investment in nonstructural components and contents (Figure 1). Significant research work is underway at the three NSF-funded earthquake research centers to develop performance assessment tools for NCCs (Whittaker and Soong, 2003).

To reduce losses in buildings in future earthquakes, the current performance-based design paradigm must be updated to shift the focus to NCCs. The geometry, type and materials that comprise a structural framing system should be selected by the structural engineer so as to protect the primary investment: the NCCs. Guidance on the appropriate choice of structural framing system to meet NCCs-driven performance objectives is needed. The *preliminary* studies reported in this paper illustrate the benefits of seismic protective systems in reducing the median seismic demand and/or the variability in seismic demand on acceleration- and displacement-sensitive NCCs. On the basis of the limited studies reported herein, framing systems incorporating seismic isolation bearings and/or supplemental fluid viscous dampers appear to offer superior performance to traditional framing systems.

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Notes

¹ This paper has been adapted from Astrella and Whittaker (2004).

² The earthquake shaking is imposed at the 1st (ground) floor level (A1) in the non-isolated models M3, M6, M7, M8 and M9, and at the basement level (A0) for the isolated models M10, M11, M12, M13, M14 and M15.