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Probabilistic seismic hazard analysis of semi-active control of tall structures

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ABSTRACT: The structural engineering community has been making great strides in recent years to develop performance-based earthquake engineering methodologies for both new and existing construction. Structural control can provide an additional method to meet desired performance objectives. Design of a structure/controller system should involve a thorough understanding of how various types of controllers enhance structural performance, such that the most effective type of controller is selected for the given structure and seismic hazard. This research develops probabilistic seismic demand curves to evaluate the performance of semi-active control systems under seismic excitations, so that the probability that any damage measure exceeds a pre-determined allowable limit can be determined. This study focuses on steel moment resisting frames, specifically the nine-story SAC Phase II structure designed for the Los Angeles region. Several controllers are developed for each structure, and their performance is judged based on the drift, dissipated hysteretic energy, and floor acceleration demands. The use of a probabilistic format allows for a consideration of structural response over a range of seismic hazards. The resulting annual demand hazard curves can be used to evaluate the effect of different control parameters as well as provide a basis for comparison between different control strategies.

1 INTRODUCTION

The structural engineering community has been making great strides in recent years to develop performance-based earthquake engineering methodologies for both new and existing construction. Both SEAOC's *Vision 2000* projects and BSSC's *NEHRP Guidelines for Seismic Rehabilitation of Buildings* present the first guidelines for multi-level performance objectives (e.g. BSSC, 1997). Structural control can provide an additional method to meet desired performance objectives. Design of a structure/controller system should involve a thorough understanding of how various types of controllers enhance structural performance, such that the most effective type of controller is selected for the given structure and seismic hazard. Current performance-based codes provide guidelines for calculation of seismic demand for a scenario event; the resulting seismic demand is then compared with the allowable limit given. This procedure generally accounts for the uncertainty present in ground motion excitation; however, the uncertainty and variability in the demand estimation generally is not considered. One approach to addressing this issue is the development of probabilistic seismic demand curves, so that the probability that any damage measure exceeds a pre-determined allowable limit can be determined.

Procedures for developing probabilistic seismic demand curves can be found in literature. These methodologies generally relate a record-specific quantity, typically the linear elastic spectral acceleration at a characteristic building frequency, to a structural response parameter, such as

interstory drift, so as to estimate the probability of exceeding the selected structural response parameter

This research utilizes the Probabilistic Seismic Control Analysis to evaluate the performance of various control systems under seismic excitations. This study focuses on steel moment resisting frames, specifically the SAC Phase II structures for the Los Angeles region, and two actuator configurations for magnetorheological (MR) smart dampers. Simulations of these systems, both controlled and uncontrolled, are prepared using the three suites of earthquake records, also from the SAC Phase II project, representing three different return periods for the area. Several controllers are developed for each structure, and their performance is judged based on the drift, dissipated hysteretic energy, and floor acceleration demands.

The use of a probabilistic format allows for a consideration of structural response over a range of seismic hazards. Stable relationships can be developed between the spectral acceleration and controlled structural demands. Similar relationships are also possible for the demands on the control system, such as the peak bearing displacement for the isolation system. As a result, fewer control analyses may be required to estimate the expected structural behaviour. The resulting annual hazard curves can be used to evaluate the effect of different control parameters as well as provide a basis for comparison between different control strategies.

2 BACKGROUND

The final objective of the methodology devised by Cornell (1996) is the estimation of the annual probability of exceedance of a given level of inelastic response in a specific MDOF structure. The structure is located at a specific site with an associated potential seismic hazard. The response of interest can be any structural response parameter, representing either local or global structural demands. The procedure couples conventional Probabilistic Seismic Hazard Analysis (PSHA) with nonlinear dynamic structural analyses in order to determine the annual probability of exceedance of a given response parameter. This procedure is referred to as the Probabilistic Seismic Demand Analysis (PSDA). These concepts are then extended to the analysis of controlled response, referred to as Probabilistic Seismic Control Analysis (PSCA).

2.1 Probabilistic Seismic Hazard Analysis (PSHA)

In view of the rareness of seismic events and the variability of their characteristics, it has become common to describe the seismic threat in probabilistic terms. The earliest work in this area has defined this threat in terms of some measure of the ground motion intensity, such as peak ground acceleration (PGA). More recent work has commonly sought to characterize the ground motion by its response spectral acceleration S_a -- i.e., the peak acceleration the earthquake will induce in a 1DOF system with a specified period T and damping ratio x . Commonly, T and x are chosen to describe the properties of the first mode of the building of interest. In any case, the aim is to describe the probability that the ground motion characteristic of interest (PGA, S_a, spectral velocity S_v, or spectral displacement S_d) is exceeded over a reference period of interest.

For example, the probability that S_a exceeds a critical value s_{cr} in an arbitrary earthquake is formally written as

$$P\left[S_a > s_{cr}\right] = \iint_{m.r} P\left[S_a > s_{cr} \mid M = m, R = r\right] f_{M,R}\left(m,r\right) dmdr \qquad (1)$$

in which $f_{M,R}(m,r)$ is the joint probability density function that describes the magnitude M and site-to-source distance R of an arbitrary event. Equation (1) is commonly evaluated numerically, and has become known as “PSHA'” or *Probabilistic Seismic Hazard Analysis*. Final results are generally rescaled by the rate n of all events being modelled, leading to a mean hazard rate $H\left(s\right)$ associated with those seismic events that produce $S_a > s_{cr}$.

As a simple parametric representation, it has been suggested (Cornell 1996) that the net result is often well-approximated by a relationship of the form:

$$H\left(S_a\right) = k_0 S_a^{-k_1} \tag{2}$$

The coefficients k_0 and k_1 thus serve to characterize the seismic threat at a given site of interest. The approximation in Equation (2) has the advantage of being linear in log-log space and has been shown to be satisfactory over a range of spectral accelerations (Shome et al, 1998)

2.2 **Probabilistic Seismic Demand Analysis (PSDA)**

Unfortunately, S_a does not provide sufficient information to determine the precise response of actual buildings, which generally show both nonlinear and multi-degree-of-freedom (MDOF) behavior. Indeed, there is a growing trend toward "performance-based" seismic design, in which a range of increasingly rare hazards are specified, together with a correspondingly increasing level of permissible "damage" (i.e., nonlinear behavior). This implies the need for explicit recognition, and statistical quantification, of the degree of nonlinear behavior-- e.g., the drift demand d -- as a function of the desired exceedance rate (return period).

Extension of conventional PSHA, to directly describe the seismic demand d of a complex, nonlinear structure, has become known as probabilistic seismic *demand* analysis (PSDA) (Cornell 1996). The power of this method hinges on the following observations. First, given knowledge of the ground motion's intensity, as measured by S_a at the building's first mode, the nonlinear behavior is often found to be *not* substantially influenced by additional ground motion parameters (e.g., *M*, *R*, duration). While somewhat counterintuitive this result has been demonstrated by detailed comparisons (Shome et al, 1998). This result permits S_a to be used as a powerful, scalar descriptor used to summarize the ground motion threat.

Secondly, in practice, considerable variability exists in S_a, which reflects the elastic demand (of a simplified, 1DOF building). A positive consequence of this fact is that one need not have a very precise description of the true nonlinear (MDOF) demand d as a function of the elastic 1DOF demand S_a. In particular, it generally suffices to have a reasonably accurate estimate of the *median* demand $\hat{d}$ for a given level of S_a. Formally, we may convey this idea by the expression:

$$d = \hat{d}\left(S_a\right) \cdot e \tag{3}$$

in which the random variable ε has unit median, and variability that is small compared with that of $\hat{d}$. In view of Equation (3), one may construct a result analogous to Equation (1).

The median relationship between spectral acceleration and drift is established by performing nonlinear dynamic analyses of the model structure for numerous ground motions at different levels of intensity, as measured by spectral accelerations. The spectral acceleration at the fundamental period of the structure for each ground motion is simply obtained from its elastic response spectrum. The response of the model structure subjected to each earthquake record provides the corresponding drift. The functional relationship between drift and spectral acceleration is taken to be:

$$\hat{d} = a S_a^{\,b} \tag{4}$$

where the exponent b is included to capture potential "softening" of the nonlinear relationship between spectral acceleration and median drift. Utilizing this relationship, an expression for the drift hazard rate, H_δ, can be expressed as (Cornell 1996, Barroso 1999):

$$H_d\left(d_{cr}\right) = k_0 \left(\frac{d_{cr}}{a}\right)^{\left(-k_1/b\right)} \cdot C_f \tag{5}$$

The last term in Equation (5), C_f, represents the first moment of a lognormally distributed random variable, given by:

$$C_f = \exp\left(\frac{1}{2}k_1^2\left(\frac{s_{\ln(d)|S_a}}{b}\right)^2\right) \tag{6}$$

Note that a value of 1 for C_f would ignore the effects of dispersion of the structural response in the drift demand hazard. As a result, this coefficient is referred to as a dispersion coefficient.

2.3 **Probabilistic Seismic Control Analysis (PSCA)**

This research proposes an extension of PSDA, which statistically describes the drift demand of a complex nonlinear structure, to analyze the modified behavior of the same structure in the presence of supplemental control (PSCA). The presence of supplemental control devices, whether active or passive, often changes the effective first-mode period and/or damping of the system. As a result, the definition of S_a, on which the response regression in Equation (4) is based should change as a function of the effective period, T_c, and damping, x_c, of the controlled system.

Computationally, the only practical difference that arises is the need to perform sufficient analyses of the controlled structure to predict the median demand $\hat{d}$ as a modified power-law function as follows:

$$\hat{d} = a_c S_a^{b_c} \tag{7}$$

The subscripts "c" here reflect that the parameters are to be fit to the controlled, rather than the original, structural system. One expects that the benefit of control is reflected, statistically, through reduced values of a_c that describe the original system.

3 **PROBLEM DEFINITION**

This investigation has the specific objective of evaluating the effect of the various controller architectures on seismic demands as described through performance-based design criteria. We believe it is critical, in the practical evaluation of the performance of buildings---whether controlled or not---under seismic threats to reflect (1) the realistic potential for nonlinear behavior and (2) the realistic characteristics of ground motion excitations (i.e., by imposing a suite of recorded ground motion records, as opposed to an idealized probabilistic, random vibration description).

3.1 **Structure**

The structure analyzed is a and nine-story steel moment-resisting frame buildings (SMRF) designed as part of the SAC steel project for the Los Angeles area. This building conforms to local code requirements. The structure is an office building designed for gravity, wind, and seismic loads, with a basic live load of 2.4 kPa (50 psf). The structural system for all buildings consists of steel perimeter moment frames and interior gravity frames with shear connections. All columns in the perimeter frame that are part of the lateral force-resisting system bend about the strong axis. The North-South frame of the 9-story structure has four fully moment resisting bays and one partially moment-resisting bay. The simple shear connection in the partial moment resisting bay occurs as a result to avoid bi-axial bending in the corner columns. The columns are pinned at the base of the structure and spliced at every other story, with the splice location six feet above the floor. A detailed description of this structure, including dimensions and member sizes, is provided in several works involving these structures (Barroso, 1999)

The structure is modelled as two-dimensional frames that represent half of the structure in the north-south direction. The frame is given half of the seismic mass of the structure at each floor level. A basic centreline model of the bare moment resisting frame is developed for both structures. The strength, stiffness, and shear distortions of panel zones are neglected. A finite element model of the structure was developed where an assembly of interconnected elements describes the hysteretic behaviour of

structural members. The inelastic behaviour of the members is taken to be concentrated at the end of girders and beams. Thus each structural member is constructed using a lumped plasticity model with nonlinear rotational springs at each end joined by a linear beam-column element. The resulting modal properties are given in Table 1.

Table 1. Modal Properties of the SAC9 Elastic Model

	SAC9 Frame	
	Period	**Damping**
1st Mode	2.27 sec	2.0%
2nd Mode	0.85 sec	1.1%
3rd Mode	0.49 sec	1.1%

3.2 Earthquakes

Suites of ten time histories were generated by Paul Somerville (Sommerville et al, 1997) to represent ground motions having probabilities of exceedance of 50% in 50 years, 10% in 50 years, and 10% in 250 years in the Los Angeles region. These sets of ground motions are referred to as the 50 in 50 Set, 10 in 50 Set, and 2 in 50 Set respectively. A single scaling factor was found for each time history that minimized the squared error between the target spectrum and the average response spectrum of the two horizontal components of the time history assuming lognormal distribution of amplitudes. The weights used were 0.1, 0.3, 0.3, and 0.3 for periods of 0.3, 1, 2, and 4 seconds respectively. The scale factor was then applied to all components of the time history. An important note regarding the earthquake sets is that they should be used only as a set, and not individually or as small sub-sets as representative of the probability levels specified. At any particular period the median spectral acceleration of the set may match the target value reasonably well; however, any individual record may have a value quite different than the expected target spectral acceleration.

3.3 Control Systems

The linear quadratic regulator (LQR) control design system is a well-known method of optimal control, and is used in this investigation to develop clipped optimal controllers for use with a magneto-rheological (MR) damper as semi-active actuators. A primary benefit of the LQR controller is the guaranteed stability that results from the solution of the Algebraic Riccati Equation (ARE) to determine the optimal state feedback gains. The LQR control design essentially seeks to find the optimal trade-off between performance and control cost, with weighting values used to define their relative importance. For multi-input multi-output (MIMO) systems, the LQR controller determines the control output that minimises the cost function:

$$J = \int_0^\infty (\mathbf{x}^T(t)\mathbf{Q}\mathbf{x}(t) + \mathbf{u}^T(t)\mathbf{R}\mathbf{u}(t))dt \tag{8}$$

where $\mathbf{x}(t)$ is the linear state vector, $\mathbf{Q}$ is a real symmetric positive semi-definite matrix containing the weighting factors for structural performance measures, $\mathbf{u}(t)$ is a vector of control inputs, and $\mathbf{R}$ is a real symmetric positive definite matrix containing weighting factors relating to the cost of control effort. For most applications, $\mathbf{Q}$ and $\mathbf{R}$ are diagonal, weighting only autocorrelations of the states and control inputs, and not the cross correlation terms. The state feedback control output is defined as:

$$\mathbf{u}(t) = -\mathbf{K}\mathbf{x}(t) \tag{9}$$

where $\mathbf{K}$ is the state feedback gain matrix obtained from the LQR control solution, with $\mathbf{u}(t)$ and $\mathbf{x}(t)$ as previously defined.

The LQR control design was undertaken using the MATLAB control toolbox, with a lumped mass reduced state model of each structure. Following the formation of the reduced model mass, stiffness, and damping matrices, the reduced actuator force mapping vector was obtained, with the appropriate

degrees-of-freedom multiplied by their relative proportion of the total maximum force in order to obtain gains in the correct proportions for each actuator saturation level. The effect of these relative force proportion factors on the gain magnitudes was removed through re-multiplication of the final state feedback gain matrix by the relevant proportions. This approach to design is necessary when considering variable actuator saturation levels, and represents a new approach to LQR designs for civil structural control.

The primary difficulty with using optimal control techniques is in the selection of the appropriate weighting factors to give the best performance attainable for the available control effort. An iterative process was used, with weighting factors adjusted until the control force magnitudes gave sufficient actuator saturation during strong structural motions but not during smaller transient cases. As building damage is commonly assessed using floor drifts, the weighting of displacements in the state vector were set higher than the velocity performance weightings. Hence, the LQR designs are heavily weighted toward displacement reduction.

The LQR control law for the force in the ith actuator is defined:

$$u_i = \begin{cases} -\mathbf{K}_{LQR}(i,:)\mathbf{v}_{temp} & (\text{if } u_i < F_{imax}) \\ -F_{imax}\,\mathrm{sgn}(v_i) & (\text{if } u_i > F_{imax}) \end{cases} \tag{10}$$

where $\mathbf{K}_{LQR}(i,:)$ is the ith row of the matrix optimal gains, $\mathbf{v}_{temp}$ is the reduced state vector containing displacements and velocities of the lateral degrees-of-freedom for that moment in time, F_{imax} is the ith actuator saturation force, and vi is the displacement of the floor to which the ith actuator is attached, relative to the equilibrium position of the structure at that point in time, also referred to as the moving-zero. Due to the nonlinear nature of the structure, the equilibrium position is not always the same as the original undeformed position (Hunt, 2002).

The distribution of actuators in tall structures is crucial, as poor placement of actuators can be detrimental to the dynamic response by changing the balance of structural modes in the response. To assess the effect of actuator architecture on the response of the SAC9 building, two different arrangements were used. Each architecture has all of the actuators placed on the ground floor of the structure, with tendons used to apply actuator authority to the required floors. Due to the large tendon angle that would result when attached to the top floor, an active tuned mass damped (ATMD) is placed on the roof of the building in an attempt to reduce top-whip, as is already implemented in some existing structures (Chase et al, 1996).

As shown in Figure 1(a), the first architecture, SAC9-A3, has tendons attached to the second and third stories only, with the soft first story retained. The SAC9-A8 architecture, shown in Figure 1(b), has tendons attached to floors 2 through 8, with an ATMD positioned on the roof. In both actuator architectures, the tendons span two horizontal bays, effectively reducing the application angle of the actuator forces. Practically, if installation of these long tendons is difficult due to structural members, it may be possible to install them into elevator shafts, which commonly extend through the centre of the building. These two architectures were selected primarily to determine whether it is necessary to have actuators attached to each floor when controlling tall buildings, of whether it is sufficient to use a combination of large actuator forces low in structure, and smaller forces applied to the top floor through an ATMD.

The maximum actuator force is set at 6675kN, which represents 13.6% of the structural weight of 48937kN. The distribution of the maximum actuator authority is shown in Table 4.2. It is important to note that the maximum actuator authority on each floor is in fact the force within the tendon, hence, actual lateral forces applied to the structure are reduced by the cosine of the tendon angle. For example, the force applied to the eighth floor is reduced by a factor of 0.48.

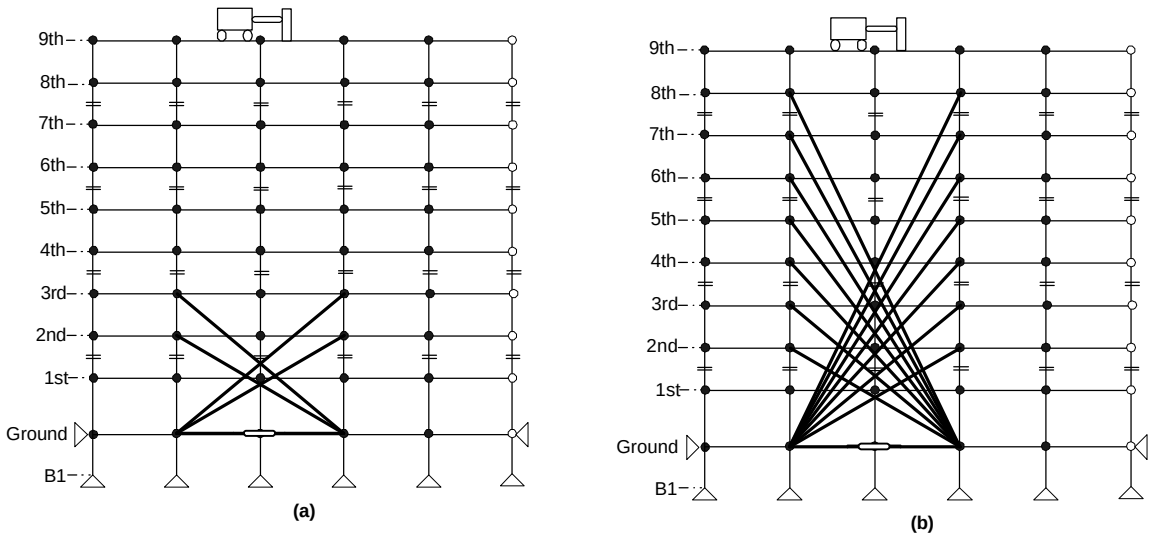


Figure 1: Actuator Architecture Configurations.

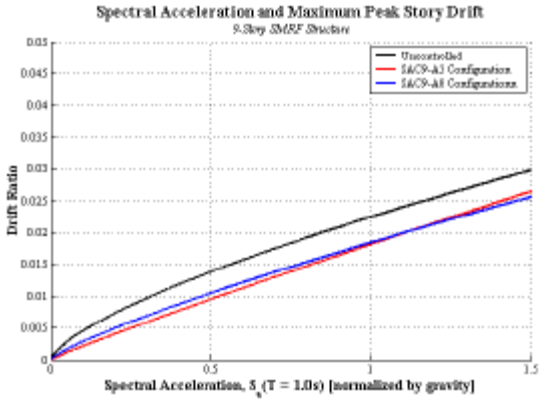
Table 2: Distributions of Maximum Actuator Forces.

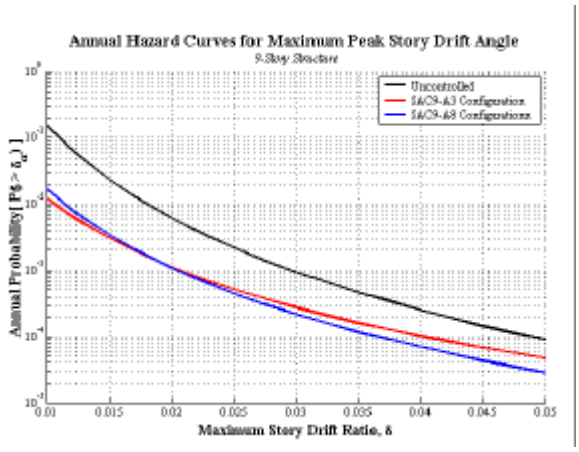
	Maximum Actuator Force Distribution (kN)									
Architecture	**Fl 1**	**Fl 2**	**Fl 3**	**Fl 4**	**Fl 5**	**Fl 6**	**Fl 7**	**Fl 8**	**Fl 9**	**Total Force (kN)**
SAC9-A3	0	3335	2450	0	0	0	0	0	890	***6675***
SAC9-A8	0	2000	1560	445	445	445	445	445	890	***6675***

4 RESULTS

The Probabilistic Seismic Control Analysis described in Section 2 is applied to the response of both actuator architectures and the original uncontrolled structure. A functional relationship of the form given in Equation (7) can be fit to the data by: 1) using the median values for S_a and $d_{\max}$ for each hazard set, or 2) utilizing the full data set of S_a and $d_{\max}$ points. A linear regression of the log of the data selected was performed to establish the parameters a_c and b_c. Regardless of whether the full data set or only the median values were used in the regression, the dispersion of $d_{\max}$ given S_a must be calculated as the square root of the mean squared deviation of the full data set from the fitted curve. Plots of S_a vs. $d_{\max}$ over the height of the structure from nonlinear structural analyses of all 3 systems are presented in Figure 2(a). Note that all 3 systems show a softening relationship between spectral acceleration and drift, indicating that yielding of the system occurs for all systems when the ground intensity is very high. Very little difference is observed between the two actuator configurations; however, the introduction of the control system significantly reduces the drift demands on the structure for the same intensity seismic event.

Once the median relationship between spectral acceleration and drift (i.e., the median drift given spectral acceleration), and the dispersion of drift given spectral acceleration are known, the spectral acceleration hazard curve can be used to create a drift demand hazard curve using Equation (5). Median values are used as the ground motion sets were developed to be used as a unit, so that the seismic demand for a structure for a given hazard is best approximated by the median response. The resulting drift demand curves are then used to provide a basis for comparison between systems. As expected, both controlled systems perform significantly better than the uncontrolled system. While the response of the controlled systems is virtually identical, the SAC-A3 marginally outperforms the SAC-A8 at the low demand level. However, as expected, the SAC-A8 configuration is significantly more effective reducing the probability that high deformation demands will be placed on the structural system.





5 CONCLUSIONS

Current performance guidelines provide for the determination of seismic demands for a given scenario event, which is then compared with an allowable limit. These procedures do not account for the variability in seismic demand estimation. One approach to address this issue is the development of probabilistic seismic demand curves. This study uses the Probabilistic Seismic Demand Analysis methodology developed by Cornell (1996) that combines information regarding the seismic hazard and nonlinear structural response. The procedure relates a record specific quantity, such as the elastic spectral acceleration, to a structural response parameter, such as drift, to estimate the probability of exceeding the selected structural parameter. The case study utilized to demonstrate this approach utilizes a 9-story steel moment-resisting frame and 2 actuator configurations for semi-active dampers. Both actuator configurations are shown to significantly reduce the annual probability of excedance for the range of drift ratios of interest, though neither approach is consistently the better choice.

6 ACKNOWLEDGEMENTS

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