Seismic Performance Assessment of Active/Hybrid Controlled Building by Response Probability Approach

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SUMMARY:
The primary goal of current performance based design is to minimize loss to human life when an earthquake strikes. The next generation of design code aims to protect the building from damage and to minimize the cost of repair. The Response Probability Approach (RPA) adopted in this paper calculates the responses with respect to the probability of non-exceedance in future earthquakes for building seismic performance and cost evaluation.

A representative hospital building in California is considered in this paper for the potential use of a hybrid damper actuator bracing control (HDABC) system as a retrofit scheme. HDABC as a hybrid control system combines visco-elastic dampers and hydraulic actuators.

The control is primarily designed to realize a roof displacement reduction objective. Since the seismic cost associates with structural and non-structural components, the probability based displacement, drift and acceleration responses are calculated for evaluating structural and non-structural displacement or acceleration sensitive components.

Keywords: Ground motions; Probability; Performance; Structural control; Non-structural component

1. INSTRUCTIONS

The next generation performance based seismic design program, ATC-58 (ATC 2011), that is under development in the United States evaluates seismic designs based on the probability of damage. The fragility curves for non-structural components are developed using certain loading protocols. Two of the main protocols are AC156 (ICC-ES 2007) and FEMA461 (FEMA 2006), trying to cover possible earthquake vibration inputs, are relatively generic. Site specific motions can more accurately represent the seismic hazard for a particular building. For an evaluation of structural components a FEMA developed program, Hazus-MH, uses incremental dynamic analysis to evaluate the probability based response and takes the input with a set of preselected ground motions per building site classes and the site-to-source distance. The ground motions recorded in historical earthquakes will not be repeated in future earthquakes even if it is the same site produced by a same fault. This paper adopts the Response Probability Approach (RPA), detailed in Zhang *et al*. 2012, to evaluate building seismic performance for building structural and non-structural components. RPA is a statistic approach and uses the Monte Carlo method, for which groups of potential ground motions are numerically generated for tectonic earthquakes, at several possible magnitudes of future earthquake. The simulation is based on seismic sources around the site and applies the finite fault method, in which the earthquake wave propagation and local site effect are considered. The peak responses caused by each motion in the group are collected and analyzed following the extreme value distribution. This approach needs more engineering effort to conduct but would satisfy the need of a precise evaluation for special design goals. The responses are given for a probability of occurrence, and the design objective is evaluated at prescribed probabilities. The extreme value distribution used for data analysis is a distribution type suitable to uncertain time dependent output. The RPA is generic at the seismic evaluation, though it is specifically applied for structural performance assessment in this paper.

While the performance based design focuses on the displacement evaluation for structural components to satisfy life safety and building collapse requirement, the next generation performance design aims at the cost evaluation. The cost caused by earthquake damage could be related to structural components or non-structural components, and the investment of non-structural parts accounts for a majority of the building cost (82%, 87%, and 92% for office, hotel and hospital buildings, respectively, Taghavi and Miranda 2003). In the 1994 Northridge earthquake, the economic loss due to non-structural damage accounted for 50% of the total damage. The non-structural components can be classified as displacement or acceleration sensitive, therefore the evaluation in this paper covers building story drifts and also floor accelerations.

In current industry practices, the building is designed to withstand earthquakes by satisfying strength and drift limit prescribed by building codes. This approach is not sufficient at some circumstances in which special requirements exist to protect important facilities. The design will require a work beyond code prescriptions and an accurate prediction of how the structure will perform during extreme earthquakes. Hospitals are notable examples of a building that would have special requirements for an earthquake since some medical equipment may be damaged by large floor acceleration. Controlling acceleration becomes a special design objective.

Structural control is a relatively new technology for building protection under a severe environmental disturbance such as strong winds or earthquakes. It can be classified as active, passive, hybrid and semi-active controls. The passive control technique including base isolation is the earliest and most widely used application for its simplicity, and it doesn't require an external power. The active control system put control forces on the building structure through employment of actuators with external power input, which may exclude the inelastic deformation for considered earthquakes. The hybrid control system combines the passive and active controls to reduce the input power required by the active system. The active/hybrid control system is promising at realizing certain prescribed objective and its application to a hospital building structure is investigated in this study. A hybrid control system (Cheng *et al.* 1996, Zhang *et al.* 2006) is employed in this paper. The system is composed of visco-elastic dampers and hydraulic actuators mounted on a chevron brace between adjacent floors, called hybrid actuator-damper-bracing control (HDABC).

In California, United States, the Alquist Act establishes a building seismic safety program for hospitals built on or after March 7, 1973. Senate Bill 1953 defines seismic performance categories, specifically the Structural Performance Categories (SPC) and the Non-structural Performance Categories (NPC). The goal of regulations is to upgrade existing hospital buildings in order to achieve a certain seismic performance. Each general acute care hospital facility must be at certain seismic performance category levels by specified timeframes. Analysis and retrofit of existing structures for earthquake loading is complex. Many different approaches of linear and non-linear static, pseudo-dynamic and dynamic analytical procedures have been developed for use in the retrofit, repair, modification of existing hospital buildings in particular cases. A sample study is conducted in this paper to retrofit a hospital building in California, United States. The building is classified as SPC1, the lowest performance level. The rehabilitation work needs to be done to improve it to SPC2, a higher performance level, by 2013, to meet the minimum life-safety requirements of Senate Bill 1953. When improved to SPC2 category, the building does not significantly jeopardize life, but may not be repairable or functional after a severe earthquake. And they are required to be brought into higher performance level by 2030, or be removed from acute care service. The HDABC control system is applied for seismic retrofit and the control effectiveness is evaluated by the RPA providing a reference for the performance evaluation, but classification of the building performance category is not covered in this study. The simulation to generate site specific ground motions considers seismic faults near the site. Ground motions can be created for prescribed earthquake magnitude, while the magnitude of 8.0 is specifically considered in this study. The design reduces the earthquake induced lateral force and story drift to satisfy the code requirement. The floor accelerations are also calculated for evaluating the acceleration sensitive equipment placement.

2. HYBRID CONTROL SYSTEM

The motion equations for a hybrid controlled *n*-story building structure under a horizontal earthquake acceleration input can be derived as (Cheng 2001)

$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+[K]\{x\}=[\delta_a]\{f_a\}+[\delta_p]\{f_p\}+\{\delta_r\}\ddot{x}_g \tag{2.1}$$

where $\{x\}$ is the vector of floor displacement; [M], [C], and [K] are mass, damping and stiffness matrices, respectively; $\{f_a\}$, $\{f_p\}$, and $\ddot{x}_g$ are active, passive input forces, and the ground motion acceleration, respectively; $[\delta_a]$, $[\delta_p]$, and $\{\delta_r\}$=-$[M]\{I_n\}$ are the input location matrices for active, passive forces and ground acceleration inputs, respectively. $\{I_n\}$ is a unit vector in order of *n*. The motion equation for an active controlled structure can be obtained by setting zero $\{f_p\}$ in Eqn. 2.1. The state space representation of the motion equation can be obtained by choosing a state vector of $\{z\}=[\{x\}^T \quad \{\dot{x}\}^T]^T$ as

$$\{\dot{z}\}=[A]\{z\}+[B_{fa}]\{f_a\}+[B_{fp}]\{f_p\}+\{B_r\}\ddot{x}_g \tag{2.2}$$

where $[A]=\begin{bmatrix}[0] & [I]\\ -[M]^{-1}[K] & -[M]^{-1}[C]\end{bmatrix}$ is the state vector and system matrix; and $[B_{fa}]$, $[B_{fp}]$ and $\{B_r\}$ are the input location matrices as $[B_{fa}]=[[0]^T\ [M]^{-1}[\delta_a]^T]$, $[B_{fp}]=[[0]^T\ [M]^{-1}[\delta_p]^T]$, and $\{B_r\}=-[\{0\}^T\ \{I_n\}^T]^T$.

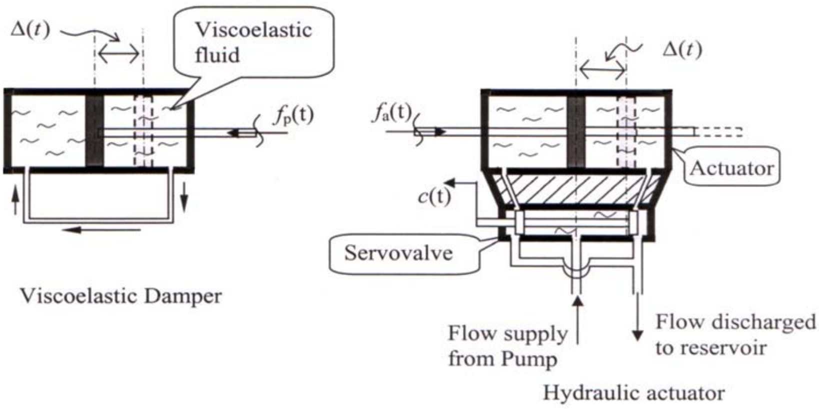

Figure 2.1. Hybrid control system: viscoelastic damper plus hydraulic actuator

As shown in Fig. 2.1, the hybrid control system is composed of visco-elastic dampers as the passive part and hydraulic actuators as the active part. Cylinders of the damper and actuator are connected to a structural floor and the piston bar of both damper and actuator are connected to the Chevron-brace. The displacement difference between the floor and brace $\Delta(t)=x_1(t)-x_b(t)$ is the piston's relative movements. The dynamic behaviour of the damper follows the constitutive relationship of visco-elastic fluids, which could be described by the Maxwell Model (Darby 1976) as

$$\lambda_0 \dot{f}_p(t)+ f_p(t) = C_0\dot{\Delta}_p(t) \tag{2.3}$$

where $f_p\ (t)$ and $\Delta_p(t)$ are the passive force and the piston displacement, respectively. C_0 is the passive damping coefficient and λ_0 is the relaxation time.

The hydraulic actuator system consists of an actuator, a servo-valve and a fluid pumping system (Babcock 1990). The actuator and the servo-valve are modelled as

$$f_a(t)=(\frac{2\beta A^2}{V})\dot{\Delta}_a(t)+(\frac{\beta A K_v}{V}\sqrt{2P_s})c(t) \tag{2.4}$$

and $\tau\dot{c}(t) + c(t) = u(t)$, (2.5)

respectively. Where in Eqn. 2.4, $f_a(t)$ and $\Delta_a(t)$ are the active force supplied and the actuator piston displacement, respectively. P_s is the fluid input pressure, which is generated by the pumping system and supposed to be a constant. A, V, β and K_v are actuator cylinder cross-section area, half cylinder volume, fluid bulk modulus and servo-valve pressure loss coefficient, respectively. Where in Eqn. 2.5, $u(t)$ is the control command and $c(t)$ represents servo-valve piston displacements; $\tau=1/(2\pi f_b)$ and f_b is servo-valve bandwidth. For the active controlled case, the optimal control is regulated the linear quadratic regulator (LQR). The objective of the algorithm is to find a control force, $\{f_a\}$, which minimizes the performance index, J, in the duration of 0 to final time t (Soong 1990) as

$$J = \int_0^t [\{z(\tau)\}^T [Q]\{z(\tau)\} + \{f_a(\tau)\}^T [R]\{f_a(\tau)\}]d\tau \tag{2.6}$$

where [Q], [R] are weighing matrices. Magnitudes of [Q], [R] represent the relative importance to the structural response and to the control forces. The influence is decided by the ratio of two matrix magnitudes. The assignment of larger values for elements in [Q] relative to those in [R] indicates the response reduction is given priority over the control force and larger control forces will be generated to cause more response reduction.

3. NUMERICAL GROUND MOTIONS AND PROBABILITY ANALYSIS

Beresnev and Atkinson 1998, developed the FINSIM program with the finite source model for tectonic earthquake simulation. In this model, the rupture is composed of small ruptures in each sub-fault, which are properly divided from the whole rupture plane with consideration of the earthquake magnitude. The rupture plane is supposed to be rectangular in appearance within the rectangular fault plane in an earthquake. The size of the rupture plane ($L\times W$) can be determined based on their empirical relations to the earthquake magnitude and tectonic motion types, such as intraplate, or interplate earthquakes, with motions of normal, reverse, or strike slip.

The FINSIM program employs a standard summation procedure, with the rupture propagating rapidly from the hypocenter and triggering sub-sources as it passes them. In the program, the motion from each sub-fault is modelled by the Point-source Stochastic Green's Function proposed by Boore 1983. First, a Gaussian white noise is modulated in time domain by use of a shaping window (Saragoni and Hart 1974) as

$$w(t) = at^b \exp(-ct)H(t) \tag{3.1}$$

where H(t) is a unit step function and normalizing factor a, shape parameters b and c are given as

$$a = \left[\frac{(2c)^{2b+1}}{\Gamma(2b+1)}\right]^{1/2},\quad b = -\varepsilon\ln\eta/[1+\varepsilon(\ln\eta-1]\,,\quad \mathrm{c} = b/\varepsilon T_w \tag{3.2}$$

where $T_w=2T_d$, $\varepsilon=0.2$, $\eta=0.05$ and Γ is the gamma function. $T_d=2\pi/\omega_c$ and ω_c is the corner frequency. The time history is then transformed into the frequency domain to be multiplied by the acceleration spectrum, $A(\omega)$, and finally transformed back into the time domain. The acceleration spectrum of the shear wave at the distance, R_d, from the rupture fault is given as

$$A(\omega) = C_n M_0 S(\omega)P(\omega)\exp\left(-\frac{\omega R_d}{2QV_s}\right)/R_d \tag{3.3}$$

where $C_n= R_{\theta\varphi}$ xFSx$RD/4\pi\rho V_s^3$ is a constant. $R_{\theta\varphi}$ is the radiation pattern coefficient; FS=2 of the amplification due to the free surface; and $RD = 1/\sqrt{2}$ of the reduction factor for partitioning the energy into two horizontal components; ρ and Vs are density and shear velocity, respectively; and Q is the propagation factor. M_0 is the seismic moment, which represents the physical strength of an earthquake. Its empirical formula related to the earthquake magnitude m_j as $\log_{10} M_0 = 1.5m_j+16.1$ (Purcaru and Berckhemer 1978). $P(\omega)$ is the high-cut filter, which is used to consider sharp decreases with increasing frequency at some cutoff frequency of ω_m observed in the acceleration spectra. $P(\omega)$ is given as $P(\omega)=[1+(\ \omega/\omega_m)^{2s}]^{-1/2}$, where s controls the decay rate at the high frequencies, suggested to take the value of 4 by Boore 1983. $S(\omega)=\ \omega^2/(1+(\omega/\ \omega)^2)$ is the source spectrum with $\omega_c\ =7.8\text{x}10^5\ V_s(\Delta\sigma\ /\ M_0)^{1/3}$ is the corner frequency, where $\Delta\sigma$ is a parameter controlling the strength of high-frequency radiation.

With a target earthquake magnitude specified, the generation of ground motions requires additional information of 1) Fault geometry including strike, dip, length, width of the fault plane and depth of the upper edge; 2) Fault location and building site location (geographic coordinates); and 3) Seismic source parameters including density and shear-wave velocity of crustal bed rock; the rupture velocity, dynamic stress drop ($\Delta\sigma$) fault slip distribution, and model for shear wave Q. After the ground motion is generated at the bedrock surface beneath the building site, amplification shall be performed to obtain ground surface motions. The cover soil is typically composed of multiple soil layers and cannot be simplified as a homogenous elastic material. The amplification effect as the wave goes though can be simulated by SHACK'91, with a soil layer profile provided.

All of generated ground motions in a group represent a future earthquake happening at the project site, of a certain magnitude. Monte Carlo analysis is performed for the interested response by collecting the amplitudes for repetitive input of all generated motions in a group. A statistical analysis is performed for a sample set for X, composed of the maxima of responses, with sample size in the number of motions. The distribution of the largest maximum (extreme value) is based on Gumbel-type distribution, which is the most frequently applied and can be used in the dynamic analysis in a civil structure for which the probability distribution function (PDF) can be expressed as (Kotz and Nadarajah 2000)

$$P_e(\eta_e) = \Pr[X \le \eta_e] = \exp[-\exp(-\alpha(\eta_e - u))] \tag{3.4}$$

The distribution is described by u and α. u is the most probable value of the extreme value, η_e. α is inversely proportional to the standard deviation, σ, and expresses the degree of dispersing. The distribution parameters of u and α are to be estimated from the data set of the response amplitudes.

4. SAMPLE STUDY OF A HOSPITAL BUILDING IN CALIFORNIA

The sample study is applied to a representative hospital building located in San Diego, California. It is an eight-story steel moment-frame building with a roof penthouse. The area of the building is approximately 39,000 square meters. The roof and floor slabs serve as horizontal diaphragms that distribute lateral loads to the steel moment frames. The steel moment frames transfer lateral loads to the spread footings and soil. The building is rectangular-shaped in plan with reduced floor plans below the third floor (Refer the building section shown in Fig. 4.1).

Table 4.1. Parameter Values for Ground Motion Generation

Fault orientation- strike/dip	138^o /89.5^o	Stress parameters (bars)	50 bars
Fault reference point	34.032176^o -118.372192^o	Crustal shear wave velocity	3.5 km/s
Depth of upper edge of fault	0 km	Crustal density (kg/m^3)	2.7 kg/m^3
Fault dimensions along strike	225 km x 112.5km	Q (f)	150 $f^{0.5}$
Rupture dimensions	$L=10^{0.5mj-1.88}$ km, W=L/2	F_{max} (rad/sec)	10 rad/sec
Subfault dimensions	$dL = dW = 10^{0.4mj-2}$ km	Geometric spreading	1/R

The Rose Canyon fault connecting to Newport-Inglewood fault cuts through the heart of downtown San Diego, and it is considered as the major seismic source for the building site. In the simulation, the geologic properties for the ground motion generation are listed in Table 4.1. A hundred ground motions are generated for earthquakes with a magnitude (m_j) of 8.0. The strike-slipe type interplate earthquake is considered and the rupture plane size and sub-fault size depends on the earthquake magnitude and the relationships are listed in Table 4.1 per Sato 1979.

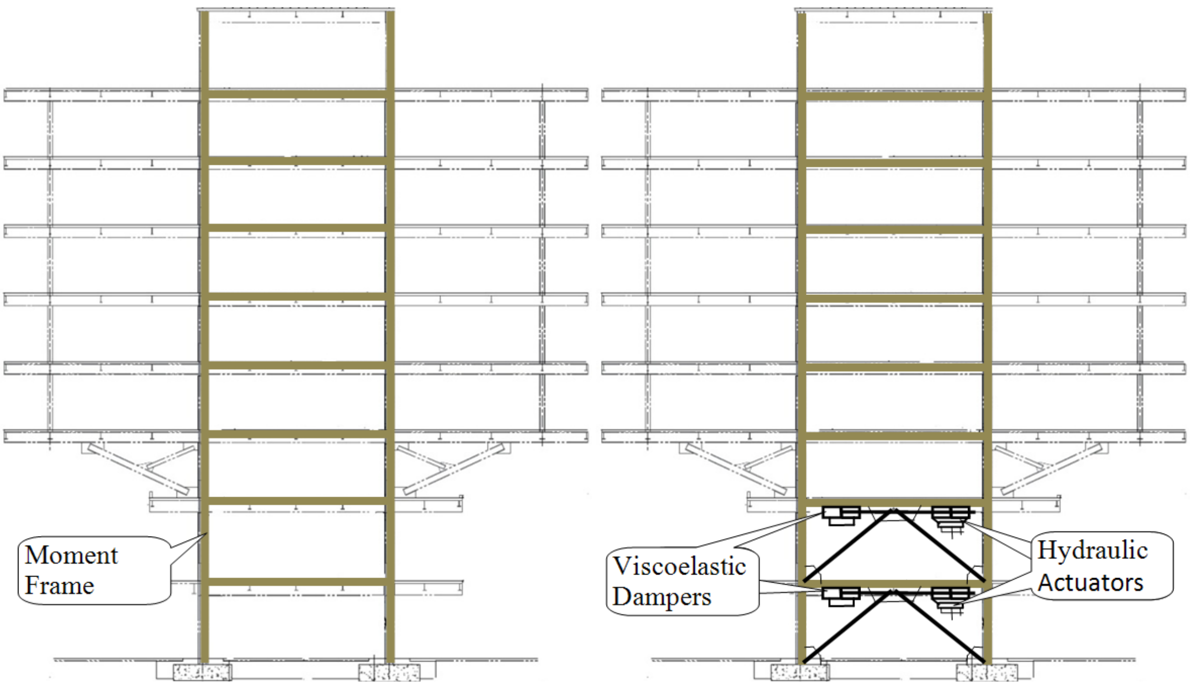

Figure 4.1. a) Existing building E-W section b) Retrofit with HDABC system

By conducting the Mote Carlo analysis described in section 3, the response probabilities are calculated for floor displacement, story drift and floor acceleration. The results are drawn in Fig. 4.2 for displacement and drift, and Fig. 4.3 for acceleration. The 90% probability of non-exceedance at a magnitude 8.0 earthquake is set as the evaluation standard in this study for all responses. As marked in Fig. 4.2, the 90% probability roof displacement is about 0.4m. It can be observed that the response amplitudes do not simply decrease from the high to low stories, for story drift and especially for floor acceleration. The amplitude distribution rather depends on the mass and stiffness distribution, which can be explained by a general dynamic analysis, but it is well illustrated in the figures. Therefore, looking into each floor specifically at the non-structural component evaluation is necessary. The probability based responses sampled in Fig. 4.2 and Fig. 4.3 can provide a good reference for the non-structural component placement.

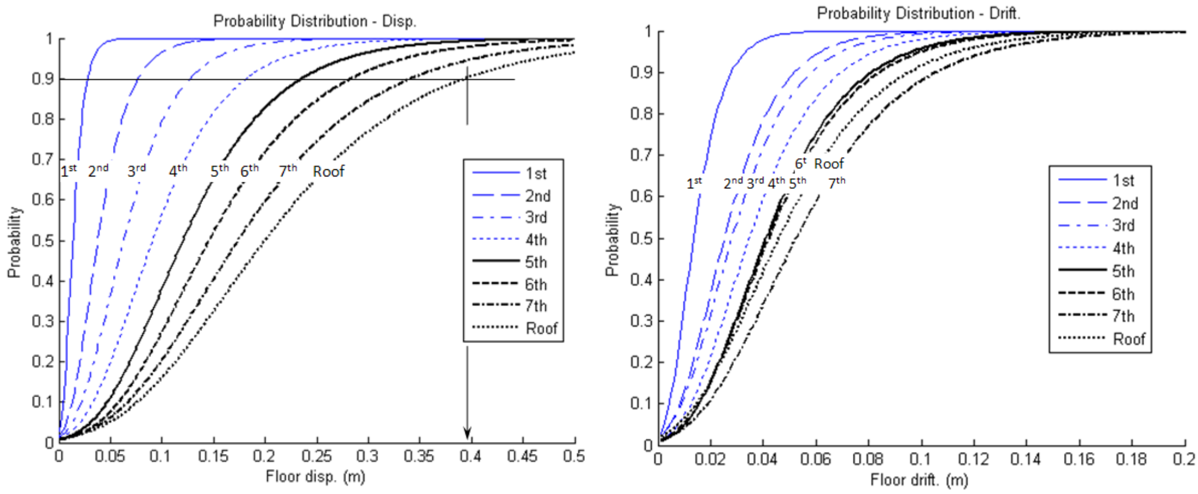

Figure 4.2. a) Response probability - floor displacement b) Response probability - story drift

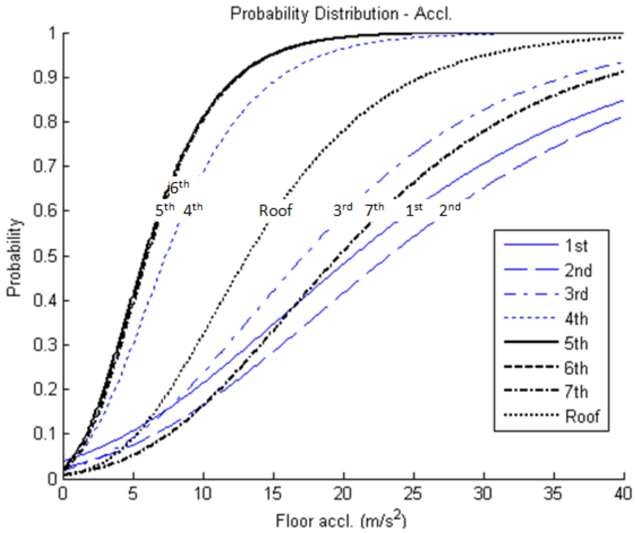

Figure 4.3. Response probability - floor acceleration

The installation of control system needs to coordinate the architectural needs with the control effectiveness. For a new construction, an optimal control placement analysis can be performed to locate the controllers. However, the placement options are quite limited for a retrofit project. In this project, the controls are placed at the first and second stories, as shown in Fig. 4.1b and only one primary direction (E-W) is studied. Upon locating the controllers, the determination of the actuator capacity is based on the design objective. The objective is to bring the roof displacement down to 70% of the pre-retrofit condition at a non–exceedance probability of 90%, which will keep the building lateral system staying within the elastic stage.

The roof drift (D/H) responses for three cases are presented in Figs. 4.4a, 4.4b and 4.4c, for pre-retrofit, passive controlled and hybrid controlled cases. The hundred maximum roof displacements are marked on the structural nonlinear push-over curve, for each case. The roof drift at 90% of probability, denoted as $(D/H)_{90}$, is brought down from 1.35% to 1.26% and 0.94% by the passive damper and the hybrid control system, respectively. And the maximum roof drift caused by the hundred inputs is reduced from 2.4% to 2.3% and 1.8% by the passive and the hybrid control systems, respectively. If the displacement passing degradation point is considered as a structural failure, the hybrid control system protects the structure from failure for a number of 99 out of 100 ground motions caused by a magnitude 8.0 earthquake.

For regulating the active control, weighting can be assigned to the elements in [Q] matrix to realize specific objectives. In this design, the roof displacement is set as a reference parameter; therefore one scheme of control regulation is to put a single weighting on the roof displacement. The other scheme is to set an equal weighting for all floor displacements and velocities. Both control schemes are bringing the roof displacement to 70% of the pre-retrofit condition and the control effectiveness to other responses are compared and evaluated for the two schemes. Similar probability response curves as Figs. 4.2 and 4.3 can be created for the controlled responses. Not to show controlled responses at every floor, responses of roof displacement and acceleration for the single weighting scheme are drawn in Figs. 4.5a and 4.5b as a demonstration. The 90% probability responses are marked in the figures to show the control effectiveness for the two responses. For all other structural responses of displacement, drift and acceleration at each floor, the results are organized in Table 4.2 for the single weighting scheme and in Table 4.3 for the equal weighting scheme. It can be observed from Table 4.2 and Table 4.3 that the control always brings a higher reduction to the peak floor displacement and drift in comparison to the reduction to the peak floor acceleration. While the displacement and drift can always be reduced by the controls, the floor acceleration can go up at some floors for the hybrid controlled structure.

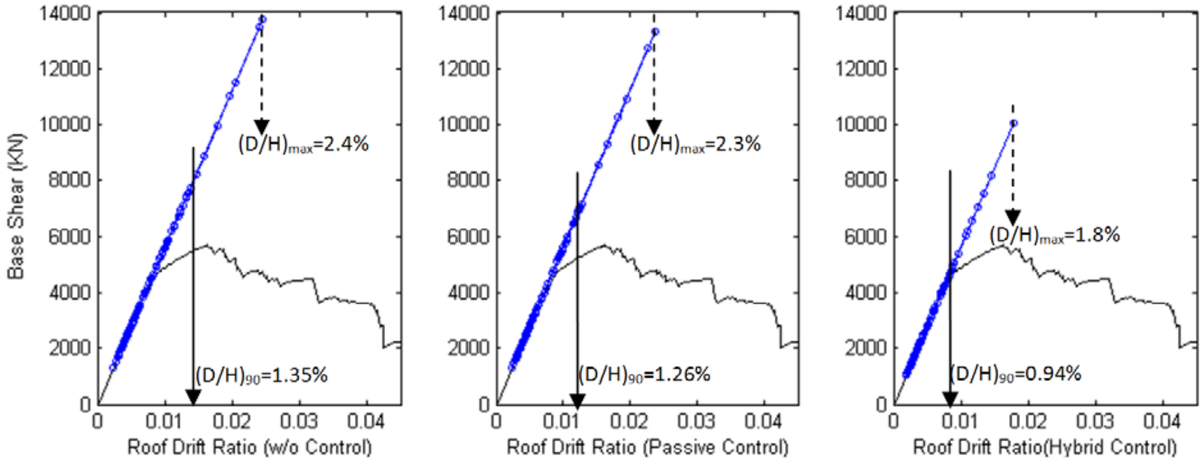

Figure 4.4. a) Without control b) With passive damper c) With damper and actuator

A comparison of the acceleration outputs between Table 4.2 and Table 4.3 reveals that the equal weighting scheme more effectively reduces the floor acceleration than the single weighting scheme. However, increasing of floor acceleration still happens at the second floor. The 2nd floor acceleration is 74% of the pre-retrofit condition under the passive control, but increases to 88% under the hybrid control. The required control forces by the two control schemes are compared by outputting the probability control force responses, given in Figs. 4.6a and 4.6b for the single weighing and the equal weighting cases, respectively. Even though the active control force demand in the equal weighting scheme is a little higher than the single weighting, the former scheme outperforms the other for a more effective reduction of displacement and drift, especially of the floor acceleration.

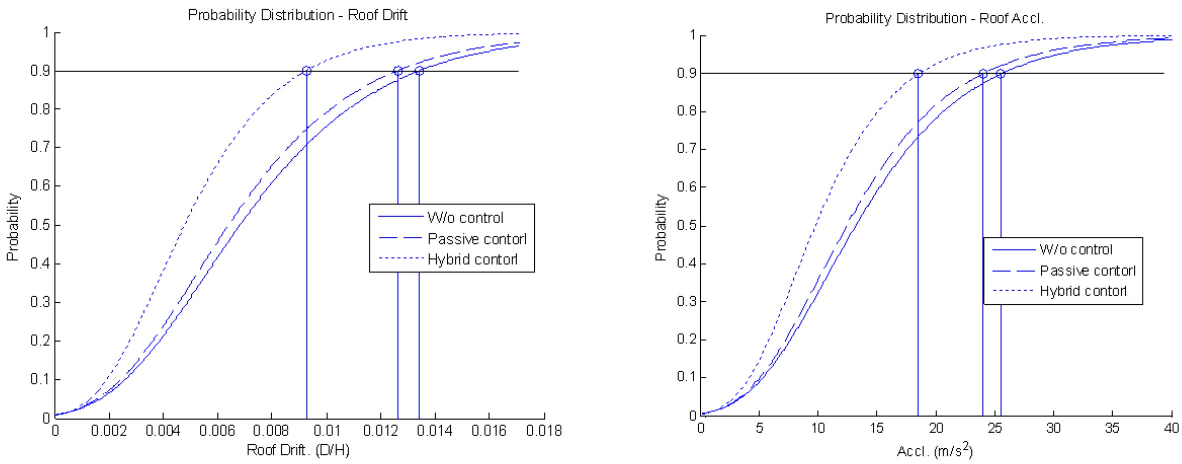

Figure 4.5. a) Response probability- roof drift b) Response probability – roof acceleration

Table 4.2. Responses at 90% Probability with Active Roof Displacement Weighting Control

	Floor	1st	2nd	3rd	4th	5th	6th	7th	Roof
D/H	a)w/o control	0.027	0.076	0.126	0.181	0.235	0.283	0.341	0.393
	Passive/(a)	85%	83%	87%	90%	92%	94%	95%	94%
	Hybrid/(a)	74%	75%	67%	67%	68%	67%	68%	**70%**
Drift (m)	b)w/o control	0.027	0.076	0.126	0.181	0.235	0.283	0.341	0.393
	Passive/(b)	85%	83%	87%	90%	92%	94%	95%	94%
	Hybrid/(b)	74%	75%	67%	67%	68%	67%	68%	70%
Accl. (m/s^2)	c)w/o control	43.9	48.3	35.2	15.3	12.2	12.3	38.7	25.5
	Passive/(c)	79%	74%	86%	88%	94%	93%	94%	94%
	Hybrid/(c)	92%	102%	84%	100%	116%	118%	77%	73%

Table 4.3. Responses at 90% Probability with Active Equal Weighting Control

	Floor	1st	2nd	3rd	4th	5th	6th	7th	Roof
D/H	a)w/o control	0.027	0.076	0.126	0.181	0.235	0.283	0.341	0.393
	Passive/(a)	85%	83%	87%	90%	92%	94%	95%	94%
	Hybrid/(a)	56%	62%	58%	62%	66%	69%	70%	**70%**
Drift (m)	b)w/o control	0.027	0.051	0.057	0.069	0.08	0.083	0.105	0.094
	Passive/(b)	85%	78%	89%	91%	94%	92%	90%	87%
	Hybrid/(b)	56%	67%	65%	71%	73%	71%	70%	68%
Accl. (m/s^2)	c)w/o control	43.9	48.3	35.2	15.3	12.2	12.3	38.7	25.5
	Passive/(c)	79%	74%	86%	88%	94%	93%	94%	94%
	Hybrid/(c)	70%	88%	68%	71%	80%	78%	73%	69%

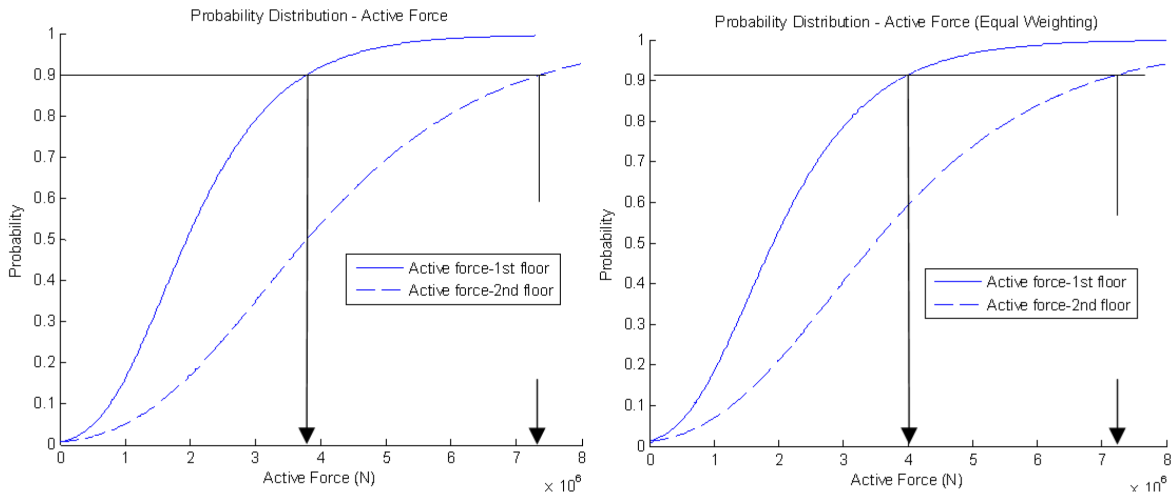

Figure 4.6. a) Control force probability by roof weighting b) Control force probability by equal weighting

5. CONCLUSIONS

The research employs the Response Probability Approach for the building performance evaluation. The approach is based on simulated site specific ground motions and presents probability based results, which is more accurate for predicting structural and non-structural component behavior during an earthquake. Since a hospital's continuous function is important after an earthquake, it is assigned with a higher safety factor in the design. Besides the reliability of building structural support, evaluation must also be given for non-structural components.

A hybrid control system, HDABC, is considered in this study for the retrofit of a representative hospital building in California. The design goal for the selection of the controllers is to bring down the reference displacement to a certain acceptable level. The probability responses are calculated for floor displacement, story drift as well as floor acceleration to evaluate structural and non-structural components. It is noted that the magnitudes of floor acceleration and story drift for different levels depend on the structural mass and the stiffness distribution. The floor acceleration probability curves can provide a useful reference for evaluating acceleration sensitive components and their floor placement. Regarding the control effectiveness, it is revealed that the percentage of reduction for floor displacement is typically greater than that for floor acceleration. Attention should be paid to the weighting selection for regulating an active control. The sample study shows that equal weighting is a favorable scheme for its effective reduction of floor accelerations.

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