Seismic Response Reduction Using a Variable Damping Device and Semi-Active Control

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Abstract
There are basically two ways to mitigate the effects of earthquakes on civil engineering structures. One way is to increase the capacity of the structures, which can be done by stiffening or strengthening them or by augmenting its ductility. Another alternative approach is to reduce the demand, i.e. the seismic loads absorbed by the structure. In turn, there are three approaches to mitigate the loads imposed by the ground motion: by means of passive, active or semi-active control.

This study examines the use of a set of two fixed-orifice viscous fluid dampers installed in the form of ***V*** with a central rod between the two arms. The bottom end of the two arms of the ***V*** can move vertically up or down, with its movement governed by the damping required by the structure at each sampled time. This motion changes the forces applied by the dampers to the structure and thus they effectively modify its damping in a continuous fashion. The mechanism is termed the Variable Damping Semi-Active (VDSA) device. Two *Closed-loop Control* and *Closed-open-loop Control* algorithms (based on the Instantaneous Optimal Control theory) are applied for selecting the appropriate position of the dampers. The algorithms are applied to control the seismic response of a single and a multi-degree of freedom structural model. The benefit of the variable damping device on reducing the response of buildings subjected to three earthquakes records is examined. The results indicate that the proposed control system is able to significantly reduce the seismic responses.

Keywords:
Seismic analysis, earthquake mitigation, semi-active control, optimal control algorithms.

1. Introduction

Usually civil engineering structures rely on its strength to withstand the large forces imposed on them by strong earthquakes. With current design procedures, the structures are expected to suffer significant damage but no collapse if the earthquake scenario that was considered for its design occurs. Although this philosophy has been the standard for many decades, new design procedures and novel devices will change the traditional approach. An example of the new procedures is the one known as performance-based design. This methodology will provide the structural engineer with the tools to pre-determine the amount of damage that the user is willing to tolerate and design the structure accordingly. A number of modern mechanical tools have been proposed in the last two decades to reduce the structural response (and thus the amount of damage). They are known collectively as protective devices and they include added viscoelastic dampers, viscous fluid dampers, frictional dampers, tune-mass dampers, and base isolation systems. The devices themselves and their design methodology are referred to as passive control systems. At the highest level of sophistication for seismic protection are the so called active control systems. Although there are a variety of active systems proposed, they have in common that in one way or another, carefully defined external forces are applied to the structure so that they counteract the effects of earthquakes or winds. So far, and albeit there are already some demonstration installations (in Japan in particular), they did not get the acceptance of the structural engineering community. There are several valid reasons for this opposition, in addition to a reluctance to abandon design methods that serve reasonably well for centuries. Among the most serious are reliability issues dealing with entrusting the safety of a structure to sophisticated devices that may fail to works as planned. Secondly, the energy required to apply these control forces is significant. Moreover, under certain conditions some control schemes, i.e. the methodology used to calculate the forces, can introduce instabilities in an otherwise perfectly stable structure.

There is an intermediate alternative between passive and fully active control systems: they are referred to as semi-active systems, and they are the topic of this paper. As its name indicates, a semi-active control combines the features of active and passive systems to reduce the dynamic response of structures. The technique uses the same passive devices mentioned before, but changes their properties in a beneficial way for the structure. It does require some energy to modify the operational characteristics of the passive devices, but the amount is negligible compared to fully active systems. Moreover, in case of failure of the sensors or the computer hardware that control the passive devices, the system becomes a passive one. In other words, the protective action of the semi-active system still remains in place, albeit with a decreased efficiency.

To calculate the control forces that operate the passive devices, it is necessary to know the response of the structure by measuring it with sensors. A proper numerical algorithm processes this information and calculates how the properties of the (formerly) passive device should be modified. The semi-active control systems can be divided into two types: active variable stiffness and active variable damping devices. In the first category, the stiffness of elements of the structure (special joints, bracings, added springs, etc) is adjusted to draw away the structure from a resonant condition with the base motion. In the second category, the geometry or mechanical properties of supplemental energy dissipating devices are modified in order to achieve enhance the response reduction due to the added damping.

Semi-active control systems have only recently been considered for applications to large civil structures. We believe that the applications of these systems to civil engineering structures were first reported by Hrovat et al. (1983). Several changeable damping devices, such as variable orifice dampers (Symans and Constantinou 1997, Kurata et al. 1999, 2000) and hydraulics dampers (Kawashima et al. 1992, Patten et al. 1993, 1996), have been developed. Variable stiffness devices have been proposed by Kobori et al. 1993, Nagarajaiah et al. 1998, Gluck et al. 2000. Furthermore, numerous algorithms have been developed for selecting the appropriate damping coefficient during the earthquake (Yang et al. 1987, Soong 1990, Sadeck and Mohraz 1996, etc). The list of references is just to provide relevant examples; they are not inclusive because it is not the object of this paper to present a review of these systems.

The present paper describes the implementation of a new variable damping semi-active control (VDSA) device. Contrary to semi-active dampers described in the technical literature, the damper coefficient c is not controlled by modifying the size of an orifice in the piston, but by changing the position of the damper. The required damping coefficient is calculated by means of two instantaneous optimal control algorithms (*Closed-loop control* and *Closed-open-loop control*). It is shown that both algorithms are effective in reducing the response. The damping coefficient $c(t)$ during the response can be adjusted between an upper limit c_{max} and a lower value c_{min}.

2. Optimal Control Algorithms

It is well known that the equation of motion of a structure modeled as a multi-degree of freedom system and subjected to a base acceleration $\ddot{x}_g(t)$ at all its supports is:

$$[M]_{n*n}\{\ddot{x}(t)\}+[C]_{n*n}\{\dot{x}(t)\}+[K]_{n*n}\{x(t)\}=-[M]_{n*n}\{E\}\ddot{x}_g(t) \quad (1)$$

where [M], [C] y [K] are the mass, damping and stiffness matrix respectively, the vectors $\{\ddot{x}(t)\}$, $\{\dot{x}(t)\}$ and $\{x(t)\}$ are the acceleration, velocity and displacement of each dynamic degree of freedom of the structure, {E} is the vector of influence coefficients, and $\boldsymbol{n}$ is the number of degrees of freedom. Although it is not required by the formulation, for simplicity a shear building model will be used to model the structure.

If the structure is outfitted with m semi-active dampers, the previous equation of motion must be changed as follows:

$$[M]_{n*n}\{\ddot{x}(t)\}+[C]_{n*n}\{\dot{x}(t)\}+[K]_{n*n}\{x(t)\}=-[M]_{n*n}\{E\}\ddot{x}_g(t)+[D]_{n*m}\{u(t)\} \quad (2)$$

The matrix [D] defines the locations of controllers, $\boldsymbol{m}$ is the number of controllers and {$u(t)$} is the m-dimensional control force vector.

To solve the system of equations of motion (2) by decomposing into a set of uncoupled equations, it is convenient to change it into a system of 2n first order differential equations. In Linear System Theory this method is referred to as the state-space representation. Introducing the following response vector and matrices,

$$\{z(t)\}=\left\{\begin{array}{c}\{\dot{x}(t)\}\\ \hline \{x(t)\}\end{array}\right\} \quad [A]=\left[\begin{array}{c|c}0 & I\\ \hline -M^{-1}K & -M^{-1}C\end{array}\right] \quad [B]=\left[\begin{array}{c}0\\ \hline M^{-1}D\end{array}\right] \quad [H]=\left[\begin{array}{c}0\\ \hline -E\end{array}\right] \quad (3)$$

equation (2) takes the form:

$$\{\dot{z}(t)\}_{2n*1}=[A]_{2n*2n}\{z(t)\}+[B]_{2n*m}\{u(t)\}+[H]_{2n*1}\ddot{x}_g(t) \quad (4)$$

To define the variation of the control forces in {$u(t)$} one needs to select a control algorithm. In this study the Optimal Control Theory will be used to define these forces. In particular, in the algorithm known as the linear quadratic regulator (or **LQR**) the optimal control forces {$u(t)$} are selected by minimizing a performance index J. The performance index is defined as the integral over time of quadratic forms defined in terms of the response and control vectors:

$$J=\int_0^{t_f}\left[z^T(t)[Q]z(t)+u^T(t)[R]u(t)\right]dt \quad (5)$$

where t_f is the duration of excitation, [Q] is a $2n$ x $2n$ symmetric positive semi-definite weighting matrix and [R] is an m x m positive definite weighting matrix. By altering the coefficients in the weighting matrices [Q] and [R] one can increase the reduction in the response at the expense of having to apply higher control forces, or vice versa.

From this point on, the estimation of the control forces $\{u(t)\}$ provided by the device and the response state vector $\{z(t)\}$ depends of the specific control configuration of sensors and actuators used, and its corresponding algorithm. There are two possible configurations: *Closed-loop control* and *Closed-open-loop control*. The *Open-loop* case is the structure without any active or semi-active control system. The three schemes were used in this study. The theory to define the control and response vectors is very specialized and it can be found in many textbooks and monographs (Soong 1990, Meirovitch 1990, Brogan 1991, Connor 2003). Here only the final results are reported.

For the *Closed-loop control*, the variables $\{u(t)\}$ and $\{z(t)\}$ can be obtained as follows:.

$$\{u(t)\} = -\frac{\Delta t}{2}[R]^{-1}[B]^T[Q]\{z(t)\} \tag{6}$$

$$\{z(t)\} = \left[I + \frac{\Delta t^2}{4}[B][R]^{-1}[B]^T[Q]\right]^{-1}\left[[T]\{d(t-\Delta t)\} + \frac{\Delta t}{2}[H]\ddot{x}_g(t)\right] \tag{7}$$

where $[T]$ is the $(2n \times 2n)$ modal matrix whose columns are the eigenvectors of $[A]$, and $\{d(t\text{-}\Delta t)\}$ is a vector containing the response and control vectors and the excitation at time $t\text{-}\Delta t$, defined as:

$$\{d(t-\Delta t)\} = e^{[\Lambda]\Delta t}[T]^{-1}\left\{\{z(t-\Delta t)\} + \frac{\Delta t}{2}\left[[B]\{u(t-\Delta t)\} + [H]\ddot{x}_g(t-\Delta t)\right]\right\} \tag{8}$$

The matrix $[\Lambda]$ is a diagonal matrix with the eigenvalues of [A].

For *Closed-open-loop Control,* $\{u(t)\}$ and $\{z(t)\}$ is calculated with the following equations:

$$\{u(t)\} = \frac{\Delta t}{4}[R]^{-1}[B]^T\left[[P]\{z(t)\} + \{p(t)\}\right] \tag{9}$$

$$\{z(t)\} = \left[I + \frac{\Delta t^2}{8}[B][R]^{-1}[B]^T[P]\right]^{-1}\left[[T]\{d(t-\Delta t)\} + \frac{\Delta t^2}{8}[B][R]^{-1}[B]^T\{p(t)\} + \frac{\Delta t}{2}[H]\ddot{x}_g(t)\right] \tag{10}$$

In the equations (9) and (10), $[P]$ is the Riccati matrix and $\{p(t)\}$ represents the open-loop control.

$$[P] = -\left[I + \frac{\Delta t^2}{8}[Q][B][R]^{-1}[B]^T\right]^{-1}[Q] \tag{11}$$

$$\{p(t)\} = [P]\left[[T]\{d(t-\Delta t)\} + \frac{\Delta t}{2}[H]\ddot{x}_g(t)\right] \tag{12}$$

3. The VDSA device in SDOF and MDOF Structures

Figure 1 shows a single degree of freedom structure (SDOF) with a **VDSA** system. The device consists of two viscous fluid dampers installed at a variable inclined position. The dampers have fixed-constant damping coefficient c_o. The structure has stiffness k, mass m and the natural (inherent) damping of the structure is represented by a damper with constant c_s. It can be shown that the equation of motion for the structure subjected to the horizontal component of an earthquake-induced ground acceleration is:

$$m\ddot{x}(t)+\left(c_s+2c_o\cos^2\theta\right)\dot{x}(t)+kx(t)=-m\ddot{x}_g(t)+4dc_o\sin 2\theta\dot{u}(t) \tag{13}$$

where:

$$\cos^2\theta(t)=\frac{a^2}{a^2+\left[H-u(t)\right]^2} \quad ; \quad \sin 2\theta(t)=\frac{a\left[H-u(t)\right]}{a^2+\left[H-u(t)\right]^2} \quad ; \quad a=\frac{L}{2}$$

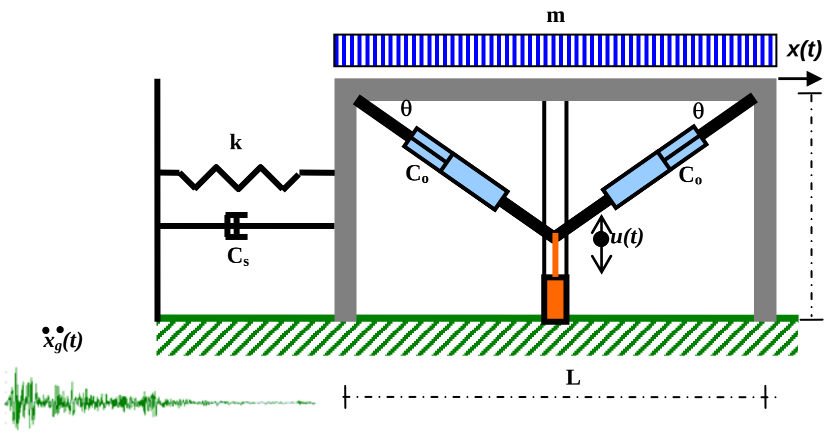

Figure 1: Single degree of freedom structure with the VDSA device

For a structure with two dampers in fixed position, the second term in the right hand side of the equation of motion (13) vanishes. This term arises due to the component of the velocity of the lower end of the dampers in the direction of the axis of the device.

Rewriting Equation (13) using the space-state representation leads to

$$\begin{Bmatrix}\dot{z}_2\\ \dot{z}_1\end{Bmatrix}=\left[\begin{array}{c|c}0 & I\\ \hline -m^{-1}k & -m^{-1}\left[c_s+2c_o\cos^2\theta(t)\right]\end{array}\right]\begin{Bmatrix}z_2(t)\\ z_1(t)\end{Bmatrix}+\left[\begin{array}{c}0\\ \hline 4dm^{-1}c_o\sin 2\theta(t)\end{array}\right]u(t)+\left[\begin{array}{c}0\\ \hline -1\end{array}\right]\ddot{x}_g(t) \tag{14}$$

This equation can be easily solved by decoupling it with the eigenvectors of the matrix in the right hand side, provided that the displacement $u(t)$ of the bottom support of the dampers is known. The term $u(t)$ must be determined by using one of the three control algorithms specified in the previous section.

The application of the VDSA dampers in multi-degree of freedom systems is similar to the SDOF case. The equations of motion for this case will not be presented here for lack of space, but a numerical example of its implementation is presented in the next section.

4. Numerical Examples

To illustrate the effectiveness of the VDSA device in reducing the seismic response, the case of a single and a multi-degree of freedom structures are considered. The responses obtained by applying the closed-loop and the closed-open-loop control algorithm are compared against the response of the uncontrolled structures. The structures were subjected to the horizontal component of three different earthquakes. First, the well known El Centro record of the Imperial Valley, California earthquake of May 18, 1940 is considered. This record has a peak ground acceleration (PGA) of 0.348g. Next, the San Fernando, California record (PGA = 1.007g). of the February 9, 1971 earthquake is used. Finally the record of the Friuli, Italy earthquake (PGA = 0.4788g) of May 6, 1976 is applied to the structures.

4.1 The case of a SDOF structure with the VDSA system

A SDOF model of a structure with a weight of $250x10^3$ lb, stiffness coefficient of $20x10^5$ lb/in, damping ratio of 2%, natural period of 0.11 sec., damping coefficient 16x103 lb.sec/in, with a single controller is used to demonstrate the advantages of the **VDSA** device. The weighting matrices [Q] and [R] in this case become scalars. For the *Closed-loop control* algorithm their values were taken equal to 10^{-2} and $8x10^{-3}$ respectively. The values of [Q] and [R] used in the *Closed-open-loop Control* algorithm are 10^{-2} and $8x10^{-4}$ respectively. The results are presented in a series of figures and later summarized in a table.

4.1.1 Results for El Centro record

The following set of figures (Figures 2 and 3) presents the relative displacement time history of the mass for the El Centro record. The figures show the uncontrolled and controlled response obtained with one of the two control algorithms.

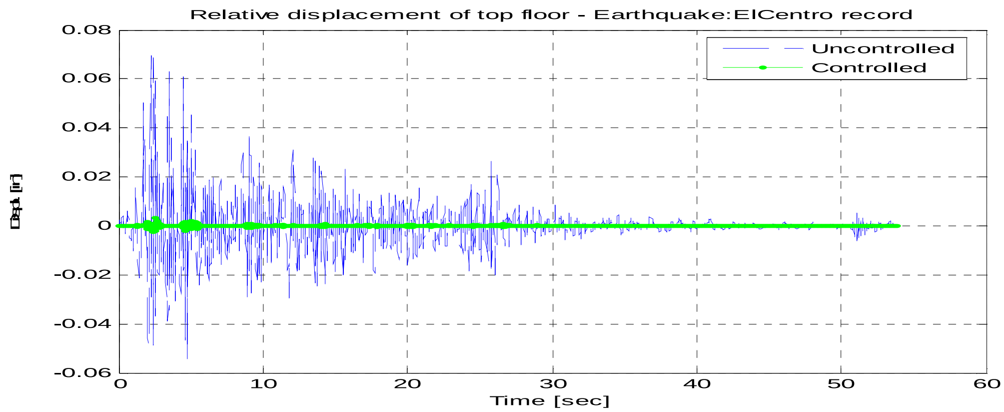

Figure 2: Relative displacements of the SDOF system due to the El Centro record calculated with and without the VDSA device and closed-loop control.

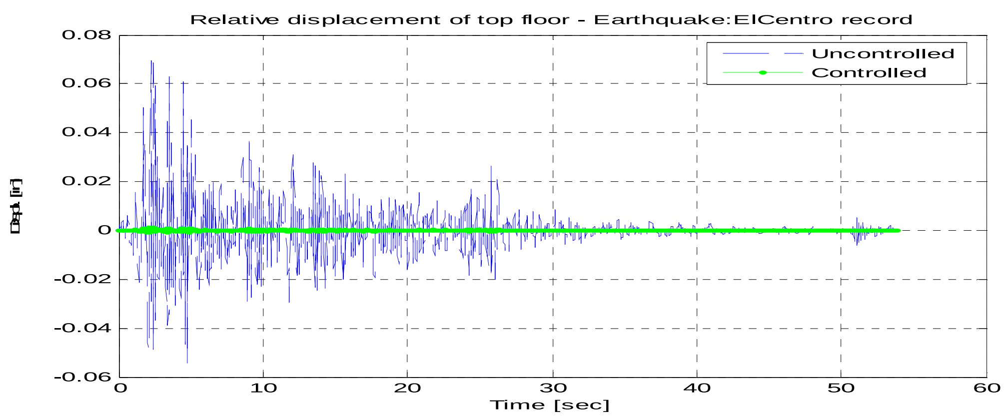

Figure 3: Relative displacements of the SDOF system due to the El Centro record calculated with and without the VDSA device and closed-open-loop control.

4.1.2 Results for the San Fernando record

The previous numerical simulations were repeated this time with the San Fernando record. The comparisons of the controlled and uncontrolled displacements are presented in Figures 4 and 5.

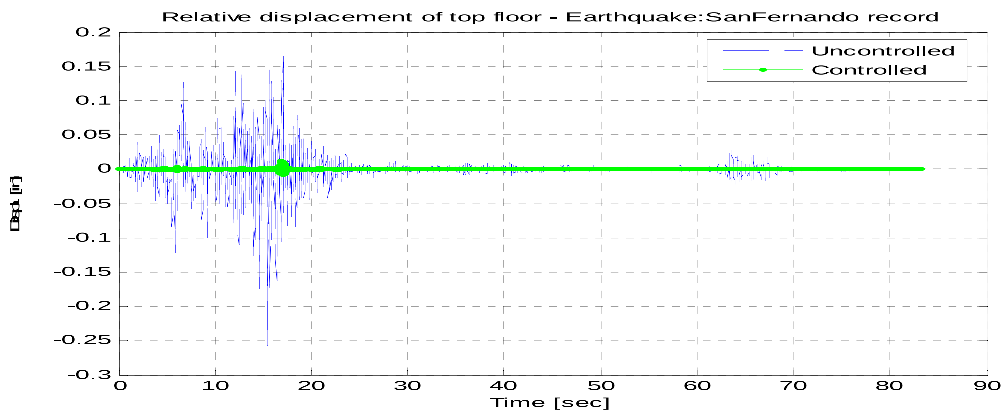

Figure 4: Relative displacements of the SDOF system due to the San Fernando record calculated with and without the VDSA device and closed-loop control.

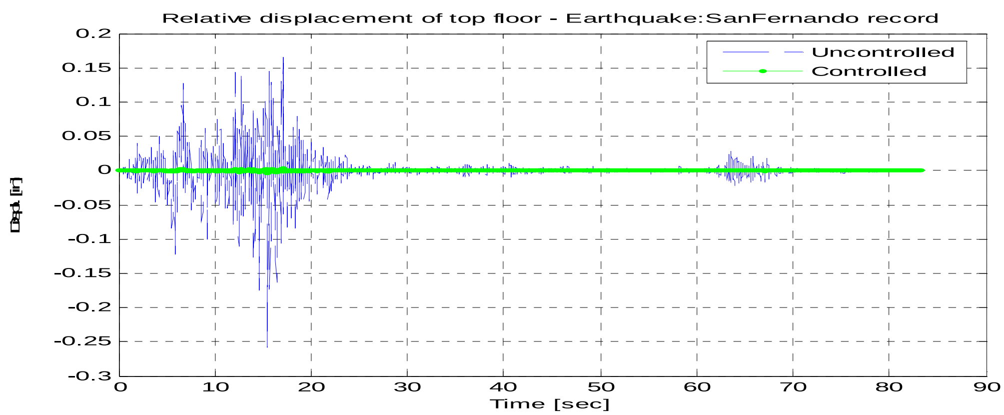

Figure 5: Relative displacements of the SDOF system due to the San Fernando record calculated with and without the VDSA device and closed-open-loop control.

4.1.3 Results for the Friuli record

The last set of results for the single degree of freedom structure is the response to the record of the Friuli earthquake displayed in Figures 6 and 7.

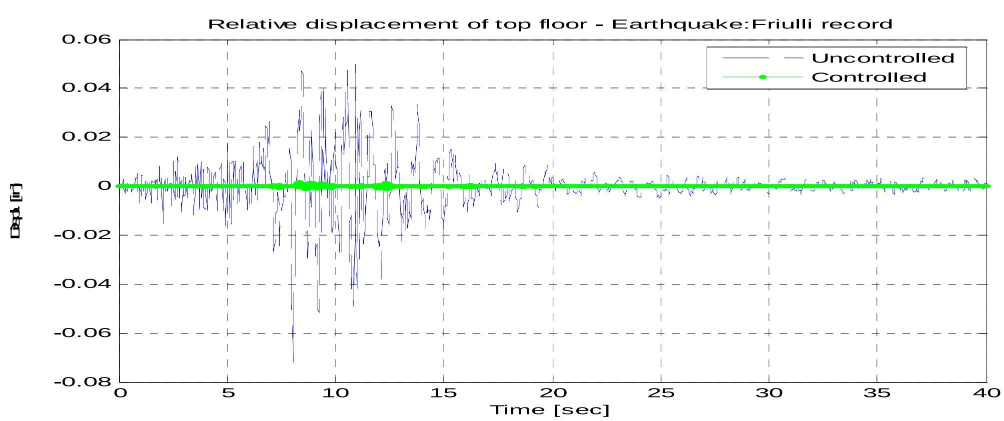

Figure 6: Relative displacements of the SDOF system due to the Friuli record calculated with and without the VDSA device and closed-loop control.

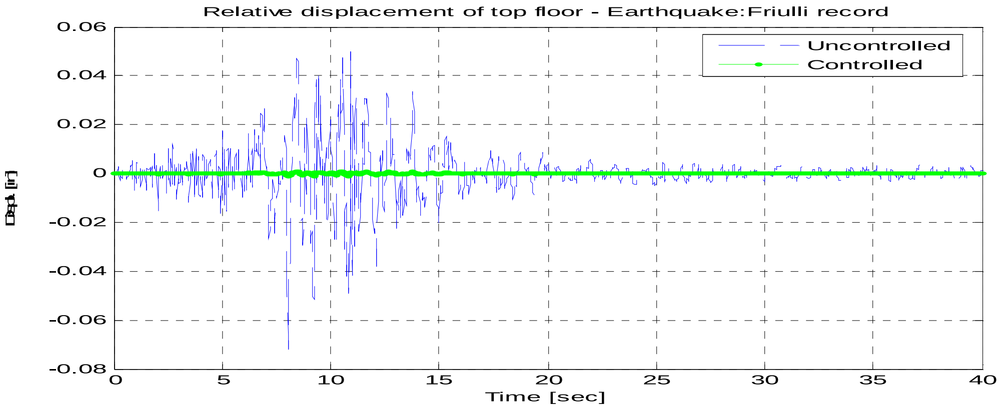

Figure 7: Relative displacements of the SDOF system due to the San Fernando record calculated with and without the VDSA device and closed-open-loop control.

Table 1 shows a summary of the results displayed in the six previous figures. It is evident form the table that the VDSA damper is capable of significantly attenuate the response of the single degree of freedom system. When the control force is determined with the closed-open-loop scheme, the response is reduced more than with the closed-loop control. The maximum displacement of the mass due to the El Centro, San Fernando and Friuli accelerograms is reduced by approximately the same amount: 98.3, 98.6, and 98.2%, respectively.

Table 1: Maximum relative displacement of the SDOF structure

Earthquake	**Uncontrolled**	**Controlled** (*Closed-loop control*)	**Controlled** (*Closed-open-loop* algorithm)
	[in]	**[in]**	**[in]**
El Centro	0.0698	0.0016	0.0012
San Fernando	0.2590	0.0061	0.0036
Friuli	0.0718	0.0017	0.0013

4.2 The case of a MDOF structure with the VDSA system

The eight-story frame shown in Figure 8 was used to illustrate the effectiveness of the **VDSA** device in reducing the response in of a multi-degree of freedom structure. This model was taken from the monograph by Soong (1990). The total stiffness coefficients of the columns are $\boldsymbol{k_i}$ = 3.404x10^5 KN/m and the floor masses are $\boldsymbol{m_i}$ = 3.14x10^5 Kg. The damping ratio of the uncontrolled structure is assumed to be 2% for all modes. The VDSA device was assumed to be installed in the first floor only.

The natural periods of the structure are 1.0849, 0.3658, 0.2246, 0.1661, 0.1355, 0.1177, 0.1074 and 0.1018 sec. The weighting matrices [Q] and [R] are diagonal matrices and were taken equal to [$\boldsymbol{I}$] x 10^{-2} and [$\boldsymbol{I}$]x985x10^{-3}, respectively. The control displacement $u(t)$ of the bottom of the device was calculated with the *closed-loop control* algorithm.

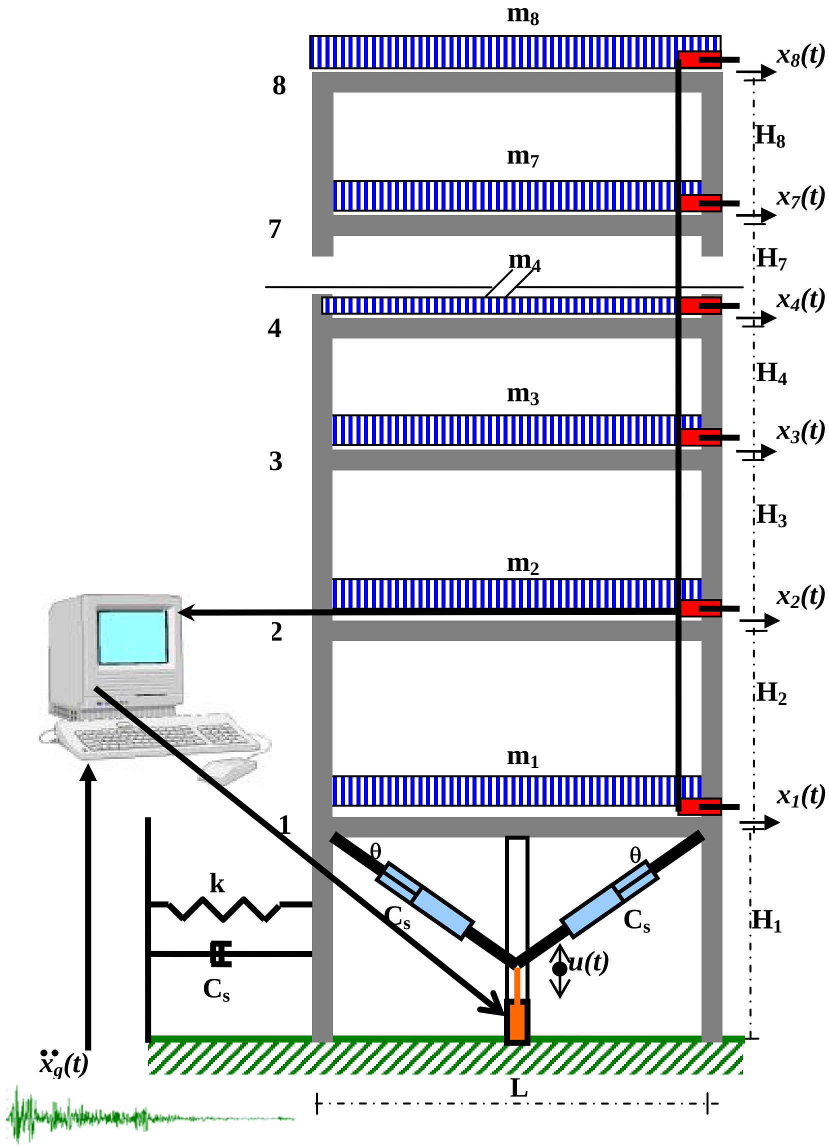

Figure 8: MDOF structure with a VDSA device

The case when the record of the 1940 El Centro record is used as seismic excitation is considered first. Figures 9 and 10 display, respectively, the displacement time histories of the first floor and the top (eight) floor of the building shown in Figure 8 when the VDSA device is turned off (uncontrolled response) and when it is active and controlled with the closed-loop control scheme. Figure 11 shows a comparison of the total base shear as a function of time for the uncontrolled and controlled case.

The next set of results in Figures 12 to 14 corresponds to the 1971 San Fernando accelerogram. Again, the responses compared are the relative displacement of the first and eight building and the total base shear.

Figures 15 to 17 display the uncontrolled and controlled response of the 8-story building to the acceleration time history of the Friuli earthquake.

The response in terms of the maximum values of the relative displacements and total base shear are next summarized in Table 2.

4.2.1 Results for the El Centro record

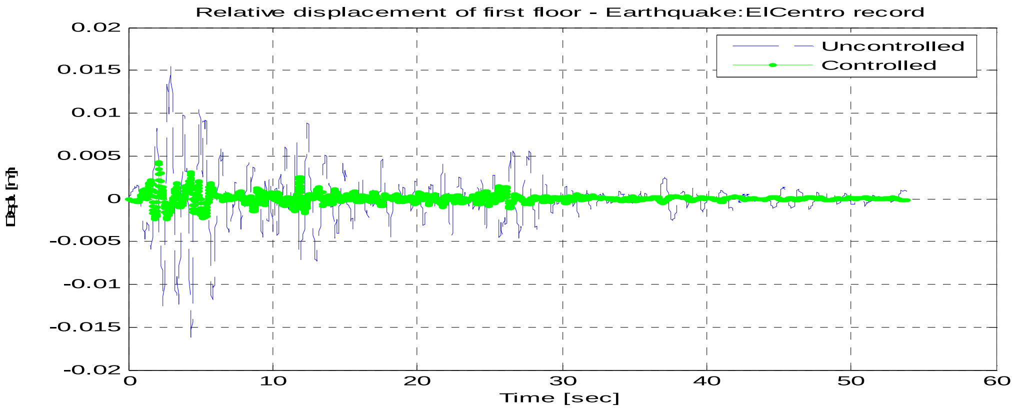

Figure 9: Displacement of the first floor of the building subjected to the El Centro record without and with the VDSA device controlled with the closed-loop control algorithm.

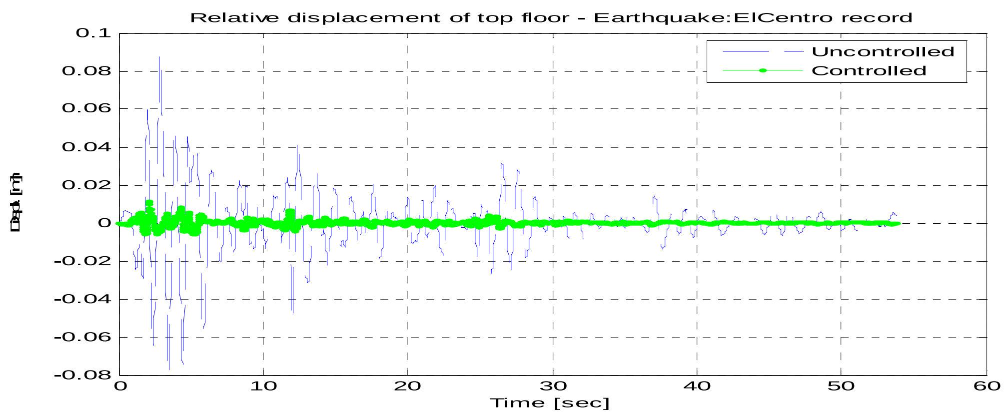

Figure 10: Displacement of the top floor of the building subjected to the El Centro record without and with the VDSA device controlled with the closed-loop control algorithm.

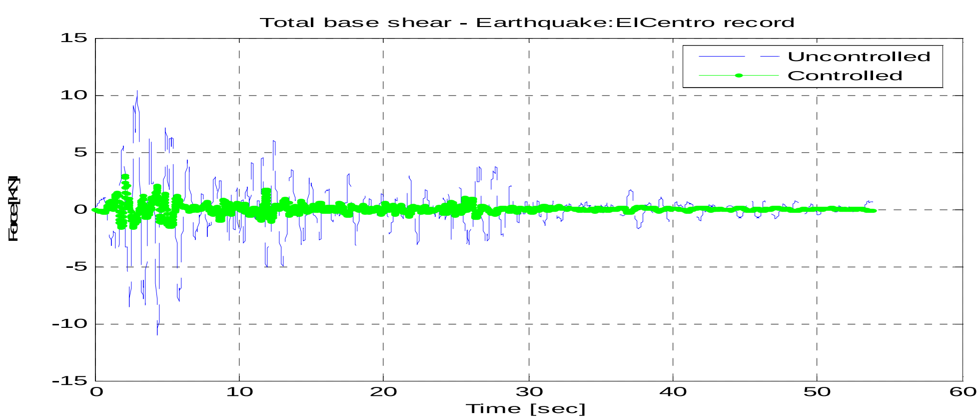

Figure 11: Total base shear of the building subjected to the El Centro record without and with the VDSA device controlled with the closed-loop control algorithm.

4.2.2 Results for the San Fernando record

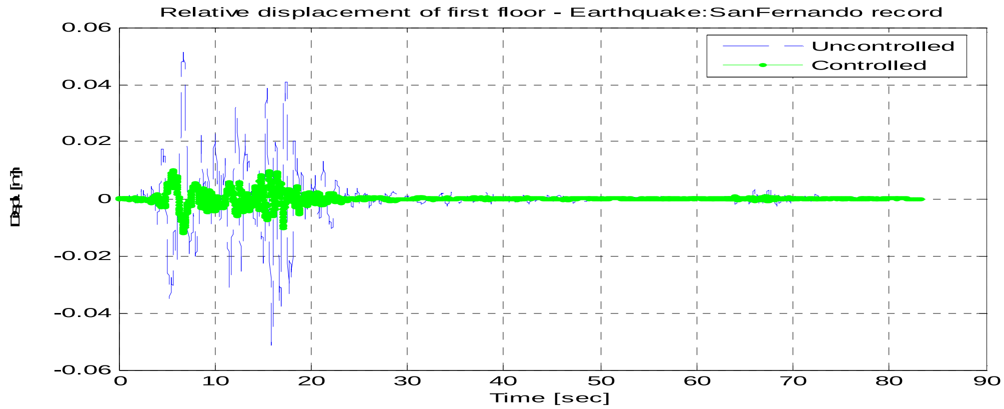

Figure 12: Displacement of the first floor of the building subjected to the San Fernando record without and with the VDSA device controlled with the closed-loop control algorithm.

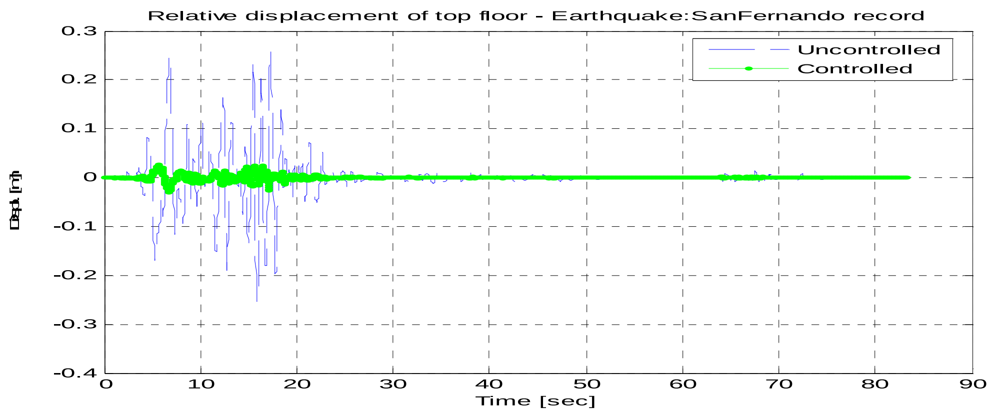

Figure 13: Displacement of the top floor of the building subjected to the San Fernando record without and with the VDSA device controlled with the closed-loop control algorithm.

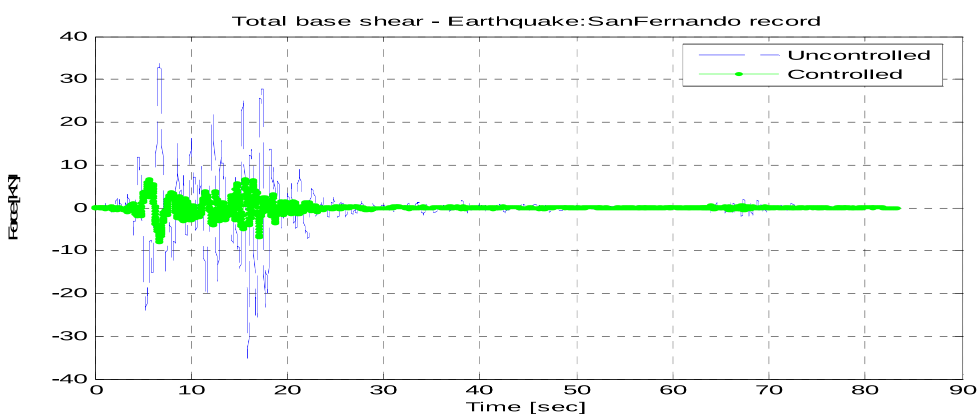

Figure 14: Total base shear of the building subjected to the San Fernando record without and with the VDSA device controlled with the closed-loop control algorithm.

4.2.3 Results for Friuli record

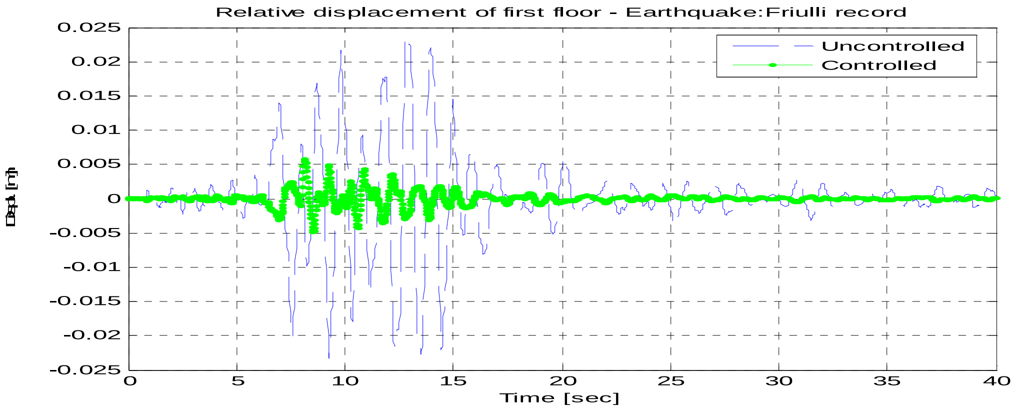

Figure 15: Displacement of the first floor of the building subjected to the Friuli record without and with the VDSA device controlled with the closed-loop control algorithm.

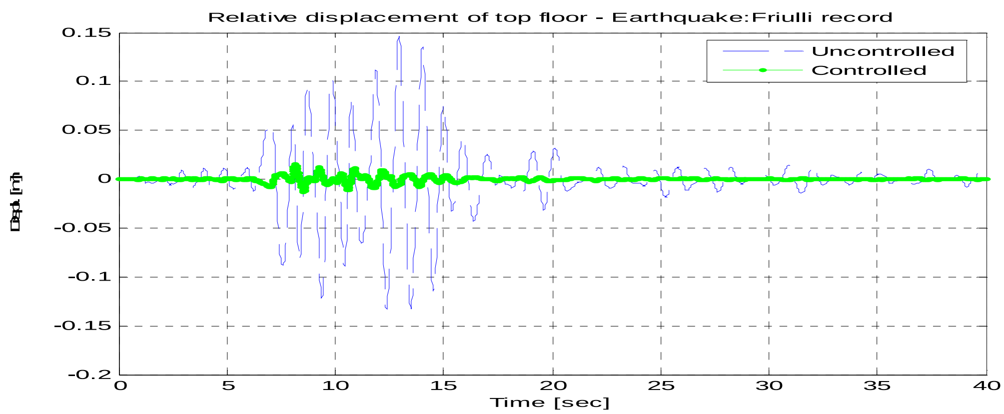

Figure 16: Displacement of the top floor of the building subjected to the Friuli record without and with the VDSA device controlled with the closed-loop control algorithm.

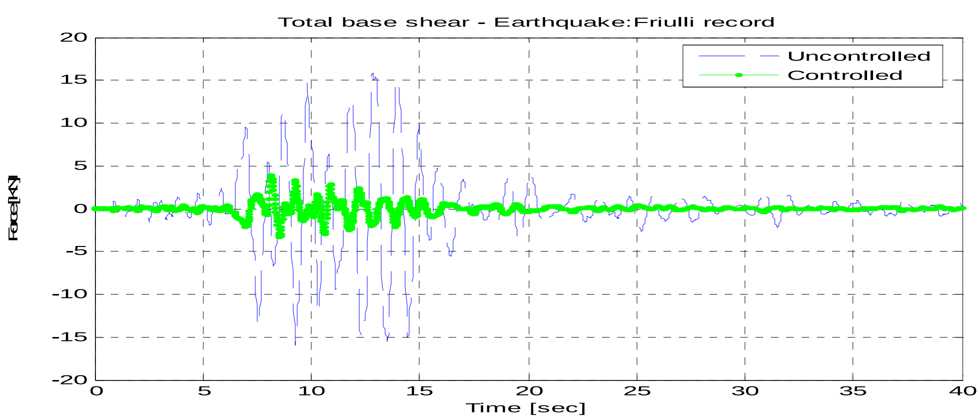

Figure 17: Total base shear of the building subjected to the Friuli record without and with the VDSA device controlled with the closed-loop control algorithm.

The most important information in the nine preceding figures can be summarized in a table showing the peak relative displacements and maximum base shear for the uncontrolled and controlled building. This is done in Table 2. According to the data in the table, the relative displacement of the first floor was reduced by an average of 75.2 % whereas the average reduction in the top floor displacement is 88.2%. The VDSA damper is able to decrease the maximum base shear by the same amount than for the first floor displacement (since they are directly proportional). The level of reduction is comparable (slightly less) to that obtained for the SDOF system. It is recalled, however, that the VDSA device was only installed in one floor of the multi-story building.

Table 2: Maximum relative displacements and base shear for the MDOF system.

Earthquake	Relative displacement First floor		Relative displacement Top floor		Total shear Base	
	Uncont.	Controlled	Uncont.	Controlled	Uncont.	Controlled
	[m]	[m]	[m]	[m]	[KN]	[KN]
El Centro	0.0162	0.0043	0.0878	0.0114	11.0140	2.9431
San Fernando	0.0516	0.0119	0.2604	0.0312	35.0696	8.1221
Friuli	0.0234	0.0058	0.1468	0.0152	15.9154	3.9657

5. Conclusions

The preliminary results presented here indicate that the proposed Variable Damping Semi-Active device is able to significantly reduce the relative displacements and base shear of buildings subjected to earthquake ground motions. Two optimal control methodologies, the *closed-loop* and *closed-open-loop* algorithms, were applied to select the displacement of the movable end of the VDSA device. This displacement also controls the effective damping coefficient of the dampers. Additional research is currently being done to further study the effectiveness of the new device under different seismic scenarios, structural configurations, etc.

References

Soong, T. T. (1990). *Active Structural Control: Theory and Practice*. John Wiley & Sons, Inc., New York, N. Y.

Hrovat, D., Barak, P., and Rabins, M. (1983). "Semi-active versus passive or active tuned mass dampers for structural control". *Journal of Engineering Mechanics, ASCE*, Vol. 109, No. 3, pp. 691-705.

Symans, M., and Constantinou, M. C. (1997). "Seismic testing of a building structure with a semi-active fluid damper control system". *Earthquake Engineering and Structural Dynamics*. Vol. 26, No. 7, pp. 759-777.

Kurata, N., Kobori, T., Takahashi, M., Niwa, and Midorikawa, H. (1999). "Actual seismic response controlled building with semi-active damper system". *Earthquake Engineering and Structural Dynamics*. Vol. 28, pp. 1427-1447.

Kurata, N., Kobori, T., Takahashi, M., Ishibashi, T., Niwa, N., Tagami, J. and Midorikawa, H. (2000). "Forced vibration test of a building with semi-active damper system". *Earthquake Engineering and Structural Dynamics*. Vol. 29, pp. 629-645.

Kawashima, K., Unjoh, S., Iida, H., and Niwa, N. (1992). "Effectiveness of the variable damper for reducing seismic response of highway bridges". *Proceedings of the 2nd U.S-Japan Workshop on Earthquake Protective Systems for Bridges.* PWRI, Tsukuba Science City, Japan. pp. 479-493.

Patten, W. N., Sack, R. L., Yen, W., Mo, C. and Wu, H. C. (1993). "Seismic motion control using semi-active hydraulic force actuators". *Proceedings of the ATC-17-1 Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*. Applied Technology Council. Redwood City, California. Vol. 2, pp. 727-736.

Patten, W. N., Sack, R. L., and He, Q. (1996). "Controlled semi-active hydraulic vibration absorber for bridges". *Journal of Structural Engineering, ASCE*. Vol. 122, No. 2, pp. 187-192.

Kobori, T., Takahashi, M., Nasu, T., Niwa, N., and Ogasawara, K. (1993). "Seismic response controlled structure with active variable stiffness system". *Earthquake Engineering and Structural Dynamics*. Vol. 22, pp. 925-941.

Nagarajaiah , S. and Mate, D. (1998). "Semi-active control of continuously variable stiffness system". *Proceedings of the 2nd World Conference on Structural Control.* Kyoto, Japan. pp. 397-405.

Gluck, J., Ribakov, Y., and Dancygier, A. N. (2000). "Selective control of based-isolated structures with CS dampers". *Earthquake Spectra*. Vol. 16, No. 3, pp. 593-606.

Yang, J. N., Akbarpour, A. and Ghaemmaghami, P. (1987). "New optimal control algorithms for structural control". *Journal of Engineering Mechanics, ASCE*. Vol. 113, No. 9, pp. 1369-1386.

Sadeck, F. and Mohraz, B. (1998). "Semiactive control algorithms for structures with variable dampers". *Journal of Engineering Mechanics*. Vol. 124, No. 9, pp. 981-990.

Sack, R. L., Kuo, C. C., Wu, H. C., Liu, L. and Patten, W. N. (1994). "Seismic motion control via semi-active hydraulic actuators". *Proceedings of the 5th National Conference of Earthquake Engineering*. El Cerrito, California. pp. 311-320.

Meirovitch, L. (1990). *Dynamics and Control of Structures*. John Wiley & Sons, Inc., New York, N. Y.

Brogan, W. L. (1991). *Modern Control Theory 3ed*. Prentice Hall, Englewood Cliffs, New Jersey.

Connor, J. J. (2003). *Introduction to Structural Motion Control*. Prentice Hall, Upper Saddle River, New Jersey.

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