IMPLEMENTATION OF DISTRIBUTED ACTIVE CONTROL IN STRUCTURES LOCATED IN SEISMIC AREAS

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SUMMARY

A solution for avoiding the harming effects of strong earthquakes or strong winds is the Structural Control [Yao 1972]. The author started a research envisioning to control the response of large structures with many active devices. The analytical study is succeeded to show that optimal active control can be used. An energy-based method of generating the parameters of control was developed. Using optimal active control method, it is possible to control large structures using many devices and, this way, the efficiency and reliability of the control are highly increased. The paper presents results obtained when large structures (as long bridges and tall buildings) equipped with several active tuned mass dampers acting on different levels and directions are subjected to strong time-history seismic acceleration records. It is shown that the response is considerably reduced. At the same time, the characteristics and the performances of the actuators are presented. The results are also showing important reductions for the frequency responses, too.

INTRODUCTION

Due to the progress of various technologies and new materials, the structures became longer and higher and, therefore, more flexible. Meanwhile, the computational methods for this kind of structures usually lead to large, three-dimensional models. When subjected to strong earthquakes or winds, flexible structures might present responses that can cause malfunction or discomfort.

The characteristics of the seismic action (as in case of winds) are far to be predicted and eventual damages could be very serious. It is more and more obvious that supplementary level of protection must be introduced to diminish the energy of the input into the structures if it is desired to build safer, higher buildings and longer bridges.

Because the effectiveness of the passive control is not proven, many analytical and experimental studies were conducted for the introduction of active control. However, the models used are usually small, simplified and thus leading to many restrictions and uncertain results. The number of active devices is also small and only in few cases they are dedicated to control more than one moving direction. The explanation of this situation is not only a cost or technological matter but it is also a matter of too many parameters involved in this sort of work.

In 1994, the fast growing of the field determined a *First World Conference on Structural Control* to be organised at Los Angeles, California, USA. With that opportunity, three prestigious scientists had shown that "at the present, several active control systems have been developed for real structural applications. These are, by large, discrete and localised sensing and control systems. In fact, only a single control mechanism, such as an active bracing system or an active mass damper, is usually incorporated into a structure. A logical extension of this research is distributed sensing and control" [Housner 1994].

Following the idea shown above, a research envisioning to control the response of large cable suspended bridge structures with many active devices was conducted [Paulet-Crainiceanu 1997]. The analytical study is succeeded to show that optimal active control can be used. An energy-based method of generating the parameters of control was developed [Paulet-Crainiceanu 1998].

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Using optimal active control method, it is possible to control large structures using many devices and, this way, the efficiency and reliability of the control are highly increased. Each actuator has a small individual (but large at the whole structure scale) supplementary available power. Therefore, one could imagine scenarios in which, in case of failure for some devices, the rest of controllers would be able to assume the tasks of the failed active means.

The problem of optimal active control is to minimise a performance index involving weighting matrices. For large structures, the choice of weighting matrices is difficult without a well-defined procedure. Therefore, the procedure that reduces the degree of arbitrariness for the coefficients in these matrices to the choice of only one parameter is presented.

Starting from a FEM computer program, *SAP IV* [Bathe 1974], the method was implemented in an original computer program and analyses were performed for some structural models.

Results obtained when large structures equipped with several active tuned mass dampers acting on different levels and directions are subjected to strong time-history acceleration seismic records are presented. It is shown that the response is considerably reduced: displacements, velocities, accelerations, forces, bending moments, and stresses are kept into allowable limits. Characteristics and the performances of the actuators are also presented.

The proposed method is proven to be efficient. It brings a simple way to apply active control for large, and therefore more accurate, three-dimensional models with large number of active devices. The increased number of active devices assures a more exact and reliable active control implementation.

ENERGY BASED ACTIVE CONTROL METHOD

Full state optimal linear active control is applied. Even if this control strategy was one of the firsts to be used, application of it to large structures is still a problem. That is mainly because of the number of parameters that intervene.

A common way to solve the situation is to adopt a trial and error iterative procedure, based on observations and experience. This is almost impossible to be applied when the model is using hundreds of lumped masses and tens of actuators. In addition, it should be noted that many newer strategies, see for example [Yang 1987] and [Yang 1994], are based on classical optimal control and therefore improvements on classical optimal control would imply improvements on other control procedures.

Under seismic action, the equation of motion for a *n* degree of freedom controlled system is

$$\mathbf{M}_1\ddot{\mathbf{z}}(t) + \mathbf{C}_1\dot{\mathbf{z}}(t) + \mathbf{K}_1\mathbf{z}(t) = \mathbf{f}(t) + \mathbf{u}(t) \tag{1}$$

where $\mathbf{M}_1 = n \times n$ mass matrix of the structure; $\mathbf{C}_1 = n \times n$ damping matrix; $\mathbf{K}_1 = n \times n$ stiffness matrix; $\mathbf{z}(t)$ = n-dimensional vector of generalised displacements; $\mathbf{u}(t)$ = n-dimensional vector of control actions; $\mathbf{f}(t)$ = n-dimensional vector of external actions.

The external force, $\mathbf{f}(t)$, is proportional to the unidirectional seismic ground acceleration

$$\mathbf{f}(t) = \mathbf{h}_1\ddot{x}_g(t) \tag{2}$$

where $\mathbf{h}_1$ = n-dimensional vector showing the points of application and the values for inertia.

Writing Equation (1) as a state equation leads to

$$\dot{\mathbf{x}}(t) = \mathbf{A}\mathbf{x}(t) + \mathbf{B}\mathbf{u}(t) + \mathbf{h}\ddot{x}_g(t) \tag{3}$$

where $\mathbf{x}(t)$ = $2n$-dimensional vector of the states.

The above transformation was obtained using:

$$\mathbf{x} = \begin{Bmatrix} \mathbf{z} \\ \dot{\mathbf{z}} \end{Bmatrix}, \quad \mathbf{A} = \begin{bmatrix} \mathbf{0} & \mathbf{I} \\ -\mathbf{M}_1^{-1}\mathbf{K}_1 & -\mathbf{M}_1^{-1}\mathbf{C}_1 \end{bmatrix}, \quad \mathbf{B} = \begin{bmatrix} \mathbf{0} \\ \mathbf{M}_1^{-1} \end{bmatrix}, \quad \mathbf{h} = \mathbf{B}\mathbf{h}_1 \tag{4}$$

In Equation (3), the dimensions for **A**, **B**, and **h** are $2n \times 2n$, $2n \times 2n$, and $2n$ respectively.

Supposing that the control actions are a function of the states, i.e. $\mathbf{u}(t) = -\mathbf{K}(t)\mathbf{x}(t)$, then the goal, [Soong 1990], is to obtain the feedback gain matrix $\mathbf{K}(t)$ such that to minimise a performance index J, defined by

$$J = \int_0^t \left[\mathbf{x}'(t)\mathbf{Q}\mathbf{x}(t) + \mathbf{u}'(t)\mathbf{R}\mathbf{u}(t)\right]dt \tag{5}$$

where **Q** and **R** = weighting matrices. In Equation (5), the prime sign means the transpose.

The matrices **Q** and **R** are square and they show the relative importance of minimising the states (structural response) or the actuating forces. The first is n-dimensional and the second is m-dimensional, where m is the number of the actuators.

Minimising J and considering (with acceptable approximation) the unknown a constant matrix **P**, the next Riccati equation takes place

$$\mathbf{PA} - \frac{1}{2}\mathbf{PBR}^{-1}\mathbf{B}'\mathbf{P} + \mathbf{A}'\mathbf{P} + 2\mathbf{Q} = \mathbf{0} \tag{6}$$

and the control gain matrix is a constant matrix

$$\mathbf{K} = \frac{1}{2}\mathbf{R}^{-1}\mathbf{B}'\mathbf{P} \tag{7}$$

Then, the necessary forces for actuators are calculated and the corresponding commands are generated for the actuators.

For applying the classical method shown above, a difficult task is to choose the coefficients involved by weighting matrices **Q** and **R**. For large systems it is not possible to accept individual trial for each coefficient. Therefore, general rules are better to envision.

In Equation (5), the first term in the brackets, $\mathbf{x}'(t)\mathrm{Q}\mathrm{x}(t)$, can be written as energy expression and thus leading to minimisation of the energy of the structural response.

In order to see some ways to calculate the structural energy and other energetic considerations see for example [Mita 1992] and [Sone 1993].

A minimising solution (in terms of energy) is got if the weighting matrix **Q** is composed from the structural matrices $\mathbf{M}_1$ and $\mathbf{K}_1$, i.e.

$$\mathbf{x}'(t)\mathbf{Q}\mathbf{x}(t) = \{\mathbf{z}' \quad \dot{\mathbf{z}}'\}\begin{bmatrix} \mathbf{K}_1 & \mathbf{0} \\ \mathbf{0} & \mathbf{M}_1 \end{bmatrix}\begin{Bmatrix} \mathbf{z} \\ \dot{\mathbf{z}} \end{Bmatrix} = \mathbf{z}'\mathbf{K}_1\mathbf{z} + \dot{\mathbf{z}}'\mathbf{M}_1\dot{\mathbf{z}} \tag{8}$$

However, the matrix **Q** should be defined as

$$\mathbf{Q} = \begin{bmatrix} \mathbf{K}^* & \mathbf{0} \\ \mathbf{0} & \mathbf{M}^* \end{bmatrix} \tag{9}$$

where $\mathbf{K}^*$ and $\mathbf{M}^*$ are modified stiffness and mass matrices.

In connection to Equation (9), a problem occurs when it is the case of the embedded active devices dynamics: the minimisation from Equation (5) should refer only to the energy of structural response. Therefore, it is not advisable to use the true mass and stiffness matrices but modified matrices in which the terms provided by the active devices are considerably diminished [Paulet-Crainiceanu 1999a].

Regarding the weighting matrix **R**, it can be taken as a diagonal matrix with terms showing the relative importance between the active devices and/or the technological differences between them.

$$\mathbf{R} = \begin{bmatrix} r_1 & \cdots & 0 \\ \vdots & \ddots & \vdots \\ 0 & \cdots & r_m \end{bmatrix} \tag{10}$$

where $r_1 \ldots r_m$ = the corresponding relative importance factors for actuators. If all of these factors are equal, then the matrix **R** becomes more easily to generate

$$\mathbf{R} = r\mathbf{I} \tag{11}$$

where r = unique relative importance factor, **I** = diagonal one matrix (identity matrix).

APPLICATIONS

A first example of application of the method presented above is the case of a cable suspended bridge, Akashi-Kaikyo [Miyata 1993]. This bridge is the world longest cable suspended bridge with a 1991m central span and two 990 m lateral spans. Towers are almost 300 m tall. Figure 1 shows a FEM model of the bridge and the dynamic characteristics are presented by Table 1 [Miyata 1996]. The total number of equations of the model is 1444 and there are 286 lumped masses.

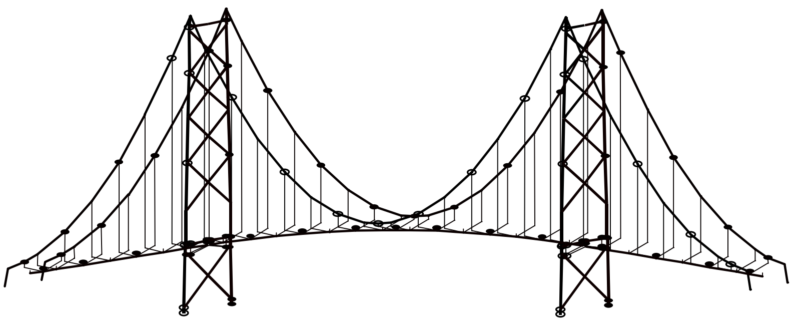

Figure 1. FEM model of a cable suspended bridge (Akashi-Kaikyo Bridge)

The analyses were conducted under El-Centro NS 1940 and Kobe NS 1995 earthquakes horizontally acting on main axis and at 45 degrees. ATMDs used were placed on deck and on towers, on the longitudinal and on the horizontal lateral directions.

The number of 100 t each active masses was 10 for the results presented in Figure 2 when El-Centro NS 1940 earthquake was acting on longitudinal direction. Comparisons with the passive and non-controlled cases are showing very good behavior in time-history, Figure 2 (left), and in frequency domain responses, Figure 2 (right). However, it should be noticed that the need of energy, power, and displacements for each actuator were large, therefore the number of actuators should be increased and studied in the future as shown by [Paulet-Crainiceanu 1997].

As for all the examples, the simulations were performed using a genuine control program written in FORTRAN 77 with elements of FORTRAN 90, [Paulet-Crainiceanu 1997]. However, the input data and the generation of mass and stiffness matrices are due to SAP IV, finite element program, [Bathe 1974].

Table 1. Dynamic characteristics of the structure in Figure 1

Mode	Freq. (Hz)
1	.03640800
2	.06073258
3	.07104239
4	.07575343
5	.07986140

To the goal to verify the strategy presented above, another example, a 3D finite element model of a long cable-stayed bridge, the recent open Tatara Bridge in Japan, is employed [Paulet-Crainiceanu 1999a]. The bridge has two towers, 215 m tall. The deck covers two lateral spans, 320m and 270 m, and one central span 890m wide. The cables' configuration is a fan type configuration [Miyata 1991]. A lateral view, horizontal perpendicular on the deck axis, of the model is shown in Figure 3.

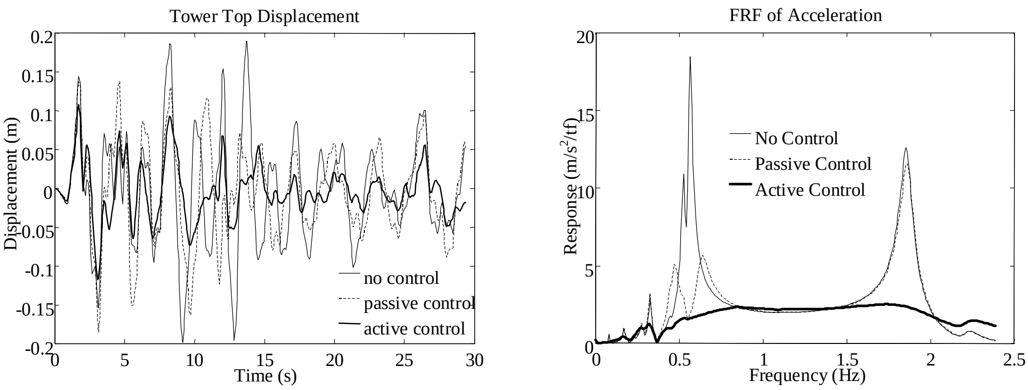

Figure 2. Results for the case of the structure in Figure 1

The towers of the bridge and the deck are modeled as beams and the cables as truss elements. All the elements take into account the geometric stiffness. 236 nodes describ the structure and generate 1370 equations, while the lumped mass number is 118. Table 2 is showing the first five natural frequencies of this bridge.

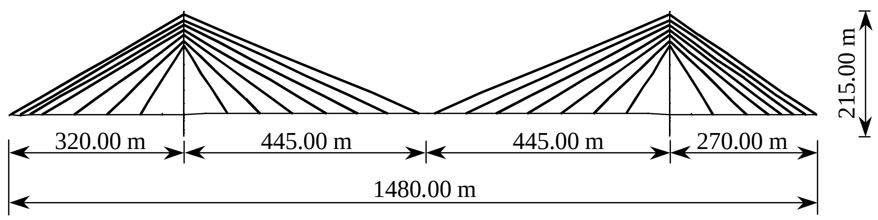

Figure 3. The model of a cable-stayed bridge (Tatara Bridge)

The seismic action was El-Centro NS, 1940 that had been applied for three main situations: no-control, passive control, and active control. As active control devices, ATMDs were employed. For the passive case the ATMDs were used as TMDs.

The placement and the number of the active/passive devices was changed many times, but in what follows it is described a typical case.

For an action in horizontal plan, perpendicular to the bridge deck axis, a number of seven ATMDs where placed. Three 8·104 kg mass ATMDs were located in the central part of the bridge deck and at 71.25 m from it. The other four ATMDs with half the mass, 4·104 kg, were placed in pairs on tops of the bridge towers.

Table 2. Dynamic characteristics of the structure in Figure 3

Mode number	Circular frequency (rad/s)	Frequency (Hz)	Period (sec.)
1	0.475	.0756	13.224
2	1.257	.2000	4.998
3	1.536	.2445	4.090
4	1.611	.2564	3.899
5	1.846	.2937	3.404

In the case with an horizontal seismic action acting longitudinal on the bridge deck, the ATMDs placement is the same plus four other 4·104 kg mass ATMDs were located in pairs in towers at a height of 128.7 m. Therefore, for this case the total number of devices was increased at 11.

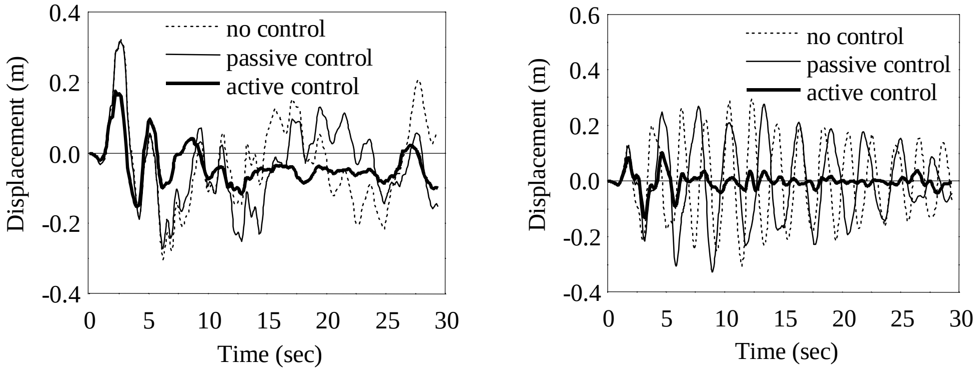

Figure 4. Some results for the case of the structure in Figure 3

For the last case, a 30-story building's three-dimensional FEM model was employed. The structure is 122 m high and its model is shown in Figure 5. It is consisting from seven 6.20 m spans on X direction and six 7.20 m spans on Y direction, assuring a square shape of the floor. The floor heights are varying from 5 m to 4 m. The columns are made from steel with rectangular sections varying from 1.60 m on 1.60 m to 1.20 m on 1.20 m. They are located in axis intersections only for the perimetral area. For the inside, free space is allowed for functions (offices), columns on Y directions being planned at 14.40 m distance. The steel girders, I shaped, have sectional heights dimensions varying from 0.90 m to 1.20 m. A 10% viscous proportional damping is assumed for the structure. The main dynamic characteristics of the building are summarised in Table 3.

Table 3. Dynamic characteristics of the structure in Figure 5

Mode number	Direction	Circular frequency (rad/s)	Period (sec.)
1	x	2.89	2.18
2	y	3.03	2.08
3	θ	3.62	1.74
4	x	4.27	1.47
5	y	5.66	1.11
6	θ	5.92	1.06
7	x	6.03	1.04
8	y	7.26	0.87
9	θ	7.67	0.82
10	x	7.93	0.79

For the active control, 16 ATMDs, with 32 actuators, 16 acting on X direction and 16 acting on Y direction, were placed on levels 15, 20, 25, and 30. Each ATMD is supposed to be from 10000 kg at level 15 to 25000 kg at level 30 and placed at the corners of the respective floors.

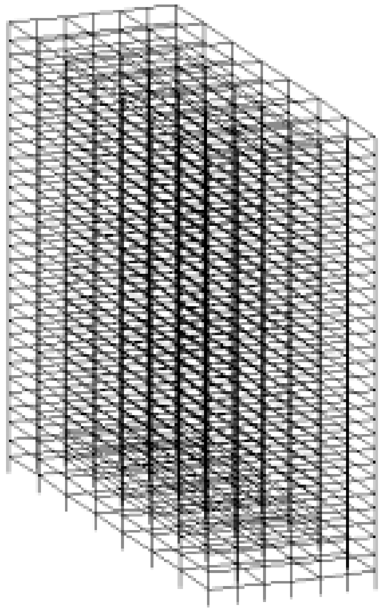

Figure 5. FEM model of a 30 story building

Simulations were conducted using two seismic actions, El-Centro NS 1940 and Vrancea (Romania) NS 1977, acting in three different manners: separately 100% on X, 100% on Y, and both 70% on X plus 70% on Y direction.

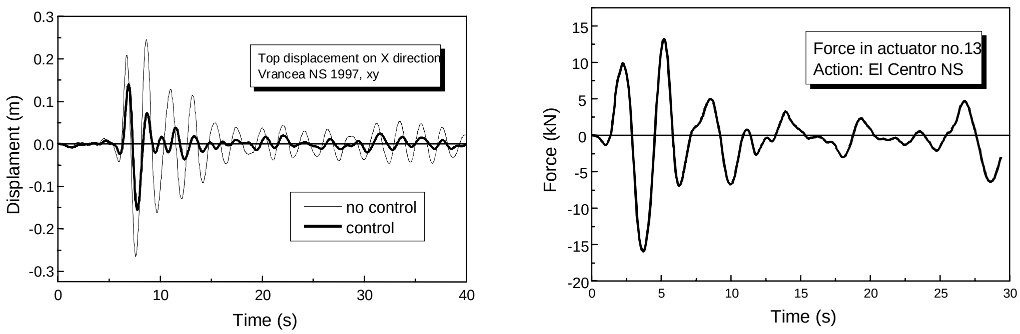

Figure 6. Some results for the case of the structure in Figure 5

From the studied cases, Figure 6 (left) shows a comparison between the displacement of the top of the building with active devices and without them when the structure is subjected to El-Centro NS 1940 earthquake. The maximum displacement 0.19 m is reduced at almost half, at 0.10 m, through the use of active control. For the same situation as above a time-history of the forces generated by the actuator number 13 located on top of the building and acting on X direction is shown in Figure 6 (right).

CONCLUSIONS

The paper presents active control as a solution to the need for flexible structures to become safer and more reliable when subjected to strong earthquakes. Use of many active devices is difficult especially because involves a large number of coefficients. For the classical optimal active control, an energy approach in generating the weighting matrices is shown. This makes the method easier to be handled and allows a more accurate control application.

The case studies are showing 3D models of a cable suspended bridge, a cable-stayed bridge, and a 30-story building equipped with ATMDs acting on different directions. Effectiveness is demonstrated through the time-history and frequency responses.

Placing more active devices means that structures become more intelligent and therefore new concepts as *Structural Robotics* would be needed [Paulet-Crainiceanu 1999b].

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