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REHABILITATION OF TALL BUILDINGS BY ACTIVE CONTROL SYSTEM SUBJECTED TO CRITICAL SIESMIC EXCITATION

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ABSTRACT

It is of substantial importance to minimize the roof displacement between two adjacent tall buildings under severe earthquakes. To eliminate the pounding effects, critical accelerations can be used to control of the structures. One of the methods to decrease the roof displacement is using active control systems. In this paper, a steel building strengthened by a belt truss system is considered. Two different constraint conditions are used, and critical accelerations which maximize the roof displacement of the structure are obtained for each. These critical accelerations are computed by solving an inverse dynamic problem, using nonlinear programming technique, based on the available information from the recorded ground motion obtained from the site of the structure or at region with similar properties to the site of building. Finally, different number of actuators are attached to the roof and other stories of the building. It shows that the active control system can reduce the roof displacement when the building subjects to the critical excitations effectively. In addition, it can be found that these critical excitations can be used to retrofit or construct the buildings which is equipped by active control system.

Keywords: Linear dynamic response analysis; critical excitation; near field acceleration; active control.

1. INTRODUCTION

In recent decades, belt truss systems have been widely utilized in tall buildings to decrease structure's deformation and increase its resistance to lateral loads [1-7]. Raj Kiran Nanduri et al. [6] considered a 30-storey three dimensional model of outrigger and belt truss system subjected to wind and earthquake loads. These results were compared to determine the lateral displacement reduction due to the outrigger and belt truss system location.

The critical excitation is produced based on information from past recorded ground

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motion. Problem of ground motion variability is very important and tough to deal with. Code-specified design ground motions are usually constructed by taking into account the knowledge from past observations and probabilistic insights. However, uncertainties in characteristics of earthquakes, the fault rupture mechanism, wave propagation mechanism, ground properties, etc. present serious difficulties in defining reasonable design ground motion; especially for important buildings in which severe damage or collapse must be avoided [8-10].

There is some literatures that studied the problem of modeling earthquake ground motions as design input for multi degree of freedom inelastic structures [11-13].

There are four different methods to control the structure: passive, active, semi-active and hybrid control. Over the past two decades, modern control approaches have become an important part of the structural seismic design and retrofitting system, especially in steel structures. Active control is one of the modern approaches in seismic design of steel structures. Judicious supervision of an active control system, as a main class of control outlines, and mitigates a structures' response by applying suitable control forces. However, a major problem with this kind of practice is its high power requirements and maintenance costs. Therefore, the design doctrine in these systems is to soothe the structural response to an acceptable level by utilizing limited applied forces. This approach is constrained by a number of actuators and their required driving energy [14].

Over the past two decades, active and passive control vibration has been used extensively by the researchers. For example, modeling and vibration control of a smart beam investigated using piezoelectric damping-modal actuators/sensors by Lin and Nien [15]. In this paper, theoretical formulations based on damping-modal actuator/sensors and numerical solutions presented for the analysis of laminated composite beam with integrated sensors and actuators.

Linear quadratic regulator (LQR)-based control force is composed of an elastic restoring force component and a damping force component. These two factors used by Ou and Li to quantify the capability of a semi-active damping system and a passive damping system to achieve the performance of a fully active control system [16].

A design strategy for control of the buildings experiencing inelastic deformations during seismic response is formulated by Cimellaro et al. [17]. The strategy was using weakened, and/or softened, elements in a structural system while adding passive energy dissipation devices (e.g. viscous fluid devices, etc.) in order to control simultaneously accelerations and deformations response during seismic events. A design methodology developed to determine the locations and the magnitude of weakening and/or softening of structural elements and the added damping while insuring structural stability.

In this paper, by using SAP 2000 program and allowable stress design (ASD 1989) method, a steel building strengthened by a belt truss system (a kind of passive control system) is analyzed and designed. It is assumed that interior frames resist gravity loads while the exterior, which is strengthened by belt truss system, resists the lateral load and some part of gravity loads. Therefore, one of the exterior frames, strengthened by belt truss system, is considered as a two dimensional shear building subjected to lateral loading. Two constraint states are considered and for each, critical accelerations are obtained which maximize the absolute value of roof displacement. For this purpose, limited information on strong ground motion is assumed to be available at any given site. The design earthquake acceleration is expressed as a Fourier series, with unknown amplitude and phase angle, modulated by an envelope function. The belt truss system is placed at 18th storey and by using time history analysis, for different time steps, the critical excitation based on constraints on ground motion is computed so that the roof displacement is maximized. The Newmark β - method is selected for time history analysis. For each constraint state, from the set of these critical accelerations one which produces the maximum roof displacement is selected. Finally, to reduce roof displacement, active control of the structure is used. Single and multi actuators are attached to the roof and to some stories. It shows that the active control system can reduce the roof displacement when the building is subjected to the critical excitations effectively. In addition, it can be found that these critical excitations can be used in the number of actuators and their maximum control force.

2. CRITICAL EXCITATION FOR ELASTIC STRUCTUTES

Derivation of critical earthquake loads for multi degree of freedom elastic structures are expressed in this section. Ground acceleration is represented as product of the Fourier series and an envelope function as follows [12]:

$$\ddot{u}_g(t) = A_0(\mathrm{e}^{-\alpha_1 t} - \mathrm{e}^{-\alpha_2 t})\sum_{i=1}^{N_f} R_i \cos(\omega_i t - \phi_i) \tag{1}$$

In Eq. (1), the R_i and ϕ_i are unknown amplitudes and phase angles, respectively. In addition, the ω_i $i = 1,2,...,N_f$ are frequencies which are selected to span the frequency range in the ground acceleration (for example $0.1\ to\ 25\ \mathrm{Hz}$). In Ref. [12], it was proposed that the frequencies must be selected so that to coincide with natural frequencies of the elastic structure. α_2 and α_1 are the two parameters that impart the observed transient tendency in the past recorded ground motion. In addition, the A_0 parameter is a scaling constant [18].

In constructing the critical acceleration, it is assumed that $A_0(\mathrm{e}^{-\alpha_1 t} - \mathrm{e}^{-\alpha_2 t})$ is specific. Therefore, the goal is determining the R_i and ϕ_i parameter so that the maximum response obtained. The information on energy E, peak ground acceleration M_1 and upper bound Fourier amplitude spectra $M_2(\omega)$ (UBFAS) for available accelerations are also determined by the following constraints ([13] and [19]):

$$\begin{aligned} &\sqrt{\int_0^{T^*} \ddot{u}_g^2(t)dt} \le E, \qquad \left|\ddot{U}_g(w)\right| < M_2(\omega), \\ &\max\left|\ddot{u}_g(t)\right| \le M_1; \qquad 0 < t < T^*. \end{aligned} \tag{2}$$

In Eq. (2), T^* and $\ddot{U}_g(w)$ are earthquake duration time and Fourier transform of the ground acceleration $\ddot{u}_g(t)$, respectivelly. The constraint on earthquake energy is related to Arias intensity [20]. Therefore, the problem is reduced to find the $2N_f$ unknown amplitudes and phase angles which give the maximum response for objective function.

To proceed further, it is needed to express the constraints in terms of the Fourier coefficients R_i and ϕ_i. Constraints listed in Eq. (2) can be expressed in terms of the unknown variables using Eq. (1).

To determine the quantities E, M_1 and $M_2(\omega)$ it is assumed that a set of N_r earthquake records denoted by $\ddot{v}_{gi}(t);\ i = 1,2,...,N_r$ are available for the site under consideration or from other sites with similar geological soil conditions. The largest value of energy and peak ground acceleration are selected as E and M_1, respectivelly. The selected records are normalized so that for each record, the Arias intensity is set to unity. These normalized records are denoted by $\ddot{\bar{v}}_{gi}(t);\ i = 1,2,...,N_r$. The bound $M_2(\omega)$ is determined by the following equation [12].

$$M_2(\omega) = E \max_{1 \le i \le N_r} \left|\bar{v}_{gi}(\omega)\right| \tag{3}$$

where $\bar{v}_{gi}(\omega);\ i = 1,2,...,N_r$ shows the Fourier transform of $\ddot{\bar{v}}_{gi}(t);\ i = 1,2,...,N_r$.

The objective function states that the absolute value of roof displacement should be maximized. For this purpose, a building strengthened by belt truss system is considered. This building is modeled as a two dimensional shear building. A time history analysis is performed and the critical excitation is obtained based on ground motion constraints at different time steps so that the absolute value of roof displacement is maximized. At the end of the analyses, one of the computed critical accelerations which resulted in the maximum roof displacement is selected. A sequential quadratic programming method and optimization algorithm was used to solve the nonlinear constrained optimization problem [21-22]. The following convergence criteria were adopted as follows:

$$\sqrt{\left(f_j - f_{j-1}\right)^2} \le \varepsilon_1; \qquad \sqrt{\left(y_{i,j} - y_{i,j-1}\right)^2} \le \varepsilon_2 \tag{4}$$

where f_j, $y_{i,j}$, ε_1 and ε_2 are the objective function at j^{th} iteration, i^{th} optimization variable at j^{th} iteration, and small quantities to be specified. Details of the procedure involved in computation of the critical earthquake are shown in Fig. 1.

3. LINEAR QUADRATIC REGULATOR ALGORITHM

Generally, for a Multi Degree Of Freedom structural system subjected to an earthquake excitation ($\ddot{x}_g(t)$) and control forces, the equation of motion can be written as:

$$\begin{aligned}&[M]_{n\times n}\{\ddot{x}(t)\}_{n\times 1}+[C]_{n\times n}\{\dot{x}(t)\}_{n\times 1}+[K]_{n\times n}\{x(t)\}_{n\times 1}\\&=-[M]_{n\times n}\{\iota\}_{n\times 1}\ddot{x}_g(t)+[D]_{n\times r}\{U(t)\}_{r\times 1}\end{aligned}\tag{5}$$

where $[M]_{n\times n}$, $[C]_{n\times n}$ and $[K]_{n\times n}$ are the mass, damping and stiffness matrices, respectively; $\{\ddot{x}(t)\}_{n\times 1}$, $\{\dot{x}(t)\}_{n\times 1}$ and $\{x(t)\}_{n\times 1}$ are the acceleration, velocity and displacement vectors of the structure at time *t*, respectively; $\{U(t)\}_{r\times 1}$ is the control force vector at time *t*, $\{\iota\}_{n\times 1}$ is a vector of order *n* with each element equal to unity, $[D]_{n\times r}$ is a matrix which shows the position of sensors and actuators and *r* is the number of sensors and actuators that has been installed on the stories of building. In this paper Linear Quadratic Regulator (LQR) method is adopted. Therefore, Eq. (5) can be written in the state space description as follows:

$$\begin{gathered}\{\dot{Z}(t)\}_{2n\times 1}=[A]_{2n\times 2n}\{Z(t)\}_{2n\times 1}+[B]_{2n\times r}\{U(t)\}_{r\times 1}+[H]_{2n\times 1}\ddot{x}_g(t)\\ \{Z(t)\}=\begin{Bmatrix}\{x\}_{n\times 1}\\ \{\dot{x}\}_{n\times 1}\end{Bmatrix};[A]=\begin{bmatrix}[0]_{n\times n} & [I]_{n\times n}\\ -[M]_{n\times n}^{-1}[K]_{n\times n} & -[M]_{n\times n}^{-1}[C]_{n\times n}\end{bmatrix}\\ [B]=\begin{bmatrix}[0]_{n\times r}\\ [M]_{n\times n}^{-1}[D]_{n\times r}\end{bmatrix};[H]=\begin{Bmatrix}\{0\}_{n\times 1}\\ -\{1\}_{n\times 1}\end{Bmatrix}\end{gathered}\tag{6}$$

where $[0]_{n\times n}$ and $[I]_{n\times n}$ are *n*-by-*n* zero and identity matrices for a *n*-degree of freedom system. Also $\{0\}_{n\times 1}$ and $\{1\}_{n\times 1}$ is the zero and identity vectors of order *n*.

In a closed-loop system with the state feedback, the control force vector $\{U(t)\}$ may be defined as the gain matrix $[G]$ multiplied by the state vector $\{Z(t)\}$. In this method, by using the linear quadratic optimal control method, the control force vector $\{U(t)\}$ is determined by minimizing the quadratic objective function

$$J(t)=\int_0^{\infty}\left(\{Z(t)\}^T[Q]\{Z(t)\}+\{U(t)\}^T[R]\{U(t)\}\right)dt\tag{7}$$

where *J(t)* is called performance index. The matrices $[Q]_{2n\times 2n}$ and $[R]_{r\times r}$ are weighting positive semi-definite and positive-definite matrices. The optimal control force without consideration of the disturbance is

$$\{U(t)\} = -[R]^{-1}[B]^T[P]\{Z(t)\} = -[G]\{Z(t)\} \tag{8}$$

where $[P]$ is a matrix of $2n\times 2n$ order that can be computed from the Ricatti equation:

$$[P][A]+[A]^T[P]+[Q]-[P][B][R]^{-1}[B]^T[P]=0 \tag{9}$$

Also $[G]$ is a matrix of $r\times 2n$ order that is called as the control gain. In this paper, the weighting parameters in Eq. (7) are considered as follows:

$$[Q]=[I]_{2n\times 2n} \qquad [R]=10^{\alpha}\times[I]_{r\times r} \tag{10}$$

where α is computed so that the maximum absolute value of control force reach to a certain value. Substituting Eq. (8) into Eq. (6), the state equation can be rewritten as:

$$\{\dot{Z}(t)\} = ([A]-[B][G])\{Z(t)\}+[H]\ddot{x}_g(t) \tag{11}$$

Therefore, in this paper, by substituting the matrices $[Q]$, $[R]$, $[A]$ and $[B]$ into Eq. (9), the $[P]$ matrix can be computed and then by using Eq. (8), the gain matrix $[G]$ is obtained. These values are substituted into Eq. (6) and the responses of the structure (displacement and velocity) due to critical excitation at roof of the structure are computed. It is noted that α parameter is selected and after computing the responses, maximum control force is determined. This value must be reach to the specified maximum value of control force. One can find more information about LQR method in Ref. [23].

4. NEUMERICAL EXAMPLE

A set of 18 earthquakes near field ground motions are used to quantify the constraint bounds E, M_1 and $M_2(\omega)$ [24]. Table 1 provides information on these records. Any new record that changes the value of constraints will automatically alter the critical acceleration. This is an inherent feature of the method [12].

The SeismoSignal program is used to compute and modify the Fourier amplitude spectra for normalized acceleration [25]. Therefore, the upper Fourier amplitude spectra is plotted in Fig. 2.

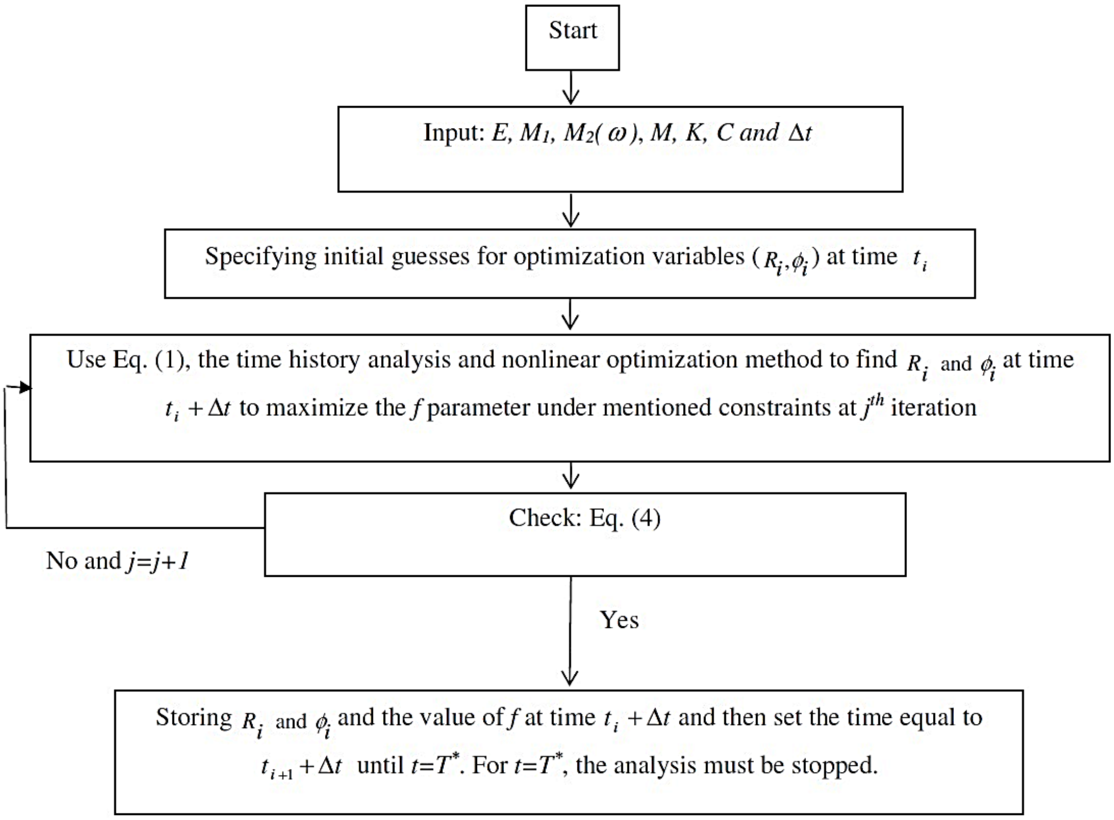

Figure 1. Flowchart for deriving the critical earthquake loads

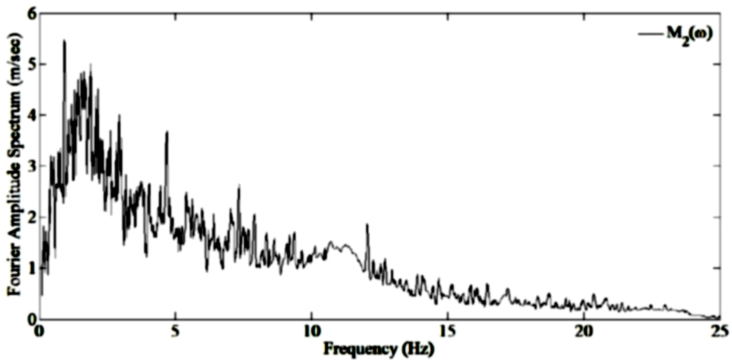

Figure 2. Upper bound of Fourier amplitude spectra for normalized selected accelerations

A twenty storey shear building (see Fig. 3) strengthened by one belt truss system is considered. The dimension of columns of the building are shown in Fig. 3. All members of the belt truss system are the same with the cross sectional area of 0.0040 (m^2). Also, all section of beams are IPE 330. The Young's modulus and specific weight of the material are $19.91\times10^{10}(N\ /\ m^2)$ and $76977.1\ (N\ /\ m^3)$, respectively. The stiffness and mass matrices are determined and the damping matrix is computed such that the damping ratio for all modes to be equal to 0.05.

The belt truss system is located at 18th storey. The first natural period of the structure is 2.96 (sec). In addition to mass of the members, dead and live loads are also considered in computing mass matrix that are equal to 15444.45 (N/m) and 588.36 (N/m) respectively for all stories except for the roof. However, the values of dead and live loads for the roof are 12502.65 (N/m) and 441.27 (N/m), respectivelly. Also, the unconditionally stable avereage acceleration method is selected for time history analysis. Here, values of α_1 and α_2 are selected to be equal to 0.13 and 0.50, respectively, which forces the earthquake duration time (T^*) to be about 30 second. The number of frequency term (N_f) is determined by using a parametric analysis and $N_f = 90$ was found to be proper in order to obtain a better convergence for the critical acceleration and objective function. In this example, the stiffness value of belt truss system is computed using ($k_{belt} = \sum_{n=1}^{Nb} \frac{AE}{L} \cos^2\theta$). In this equation, A, E, L, N_b and θ represent the cross sectional area, the modulus of elasticity, length, number of members in the belt truss and angle of members for the belt truss as measured with respect to the horizontal line, respectively. It must be noted that only the stiffens of tensile members are computed. In this example, two constraint scenarios are considered in deriving the critical earthquake from data listed in Table 2.

Table 1: Information on past ground-motion records for firm soil site

Earthquake date	Magnitude	Epic. Dist. (Km)	Comp.	PGA (m/s^2)	Energy* ($m / s^{1.5}$)	Site
Coalinga	6.5	30.1	360	2.82	2.69	Cantua Greek
(05.02.1983)			270	2.19	2.16	
Imperial Valley	6.6	15.4	S45W	2.68	2.31	Calexico fire
(10.15.1979)			N45W	1.98	2.16	
Loma Prieta	7.0	9.7	90	3.91	2.85	Capitola
(10.18.1989)			0	4.63	5.23	
Mammoth Lakes	6.2	1.5	90	4.02	3.73	Convict Greek
(05.25.1980)			180	3.92	4.02	
Morgan Hill	6.1	4.5	240	3.06	2.33	Halls Valley
(04.24.1984)			150	1.53	1.65	
Northridge	6.7	5.9	S16W	3.81	4.19	Canoga Park
(01.17.1994)			S74E	3.43	3.52	
Parkfield	5.0	9.1	90	2.89	1.33	Parkfield fault
(12.20.1994)			360	3.80	1.74	
San Fernando	6.6	27.6	N21E	3.09	2.08	Castaic Old Ridge
(02.09.1971)			N69W	2.65	2.48	
Westmorland	5.0	6.6	180	4.66	3.44	Westmorland fire
(04.26.1981)			90	3.77	3.30	

*$E = \sqrt{\int_0^\infty \ddot{v}_g^2(t)\,\mathrm{d}t}$ (similar to Arias 1970).

Table 2: Nomenclature of constraint scenarios considered

Case	Constraint imposed
1	Energy and PGA
2	Energy, PGA and UBFAS

Table 3, shows the properties of computed critical excitation such as PGA, Arias intensity and the value of roof displacement for different cases. Fig. 4, shows the acceleration time history as well as the Fourier spectra for various cases. Since, there is no constraint on upper bound of the Fourier spectra in case one, therefore the value of Fourier spectra increases leading to critical excitation with high frequency content in a frequency that coincides with the natural elastic frequency of the structure, thereby producing resonance.

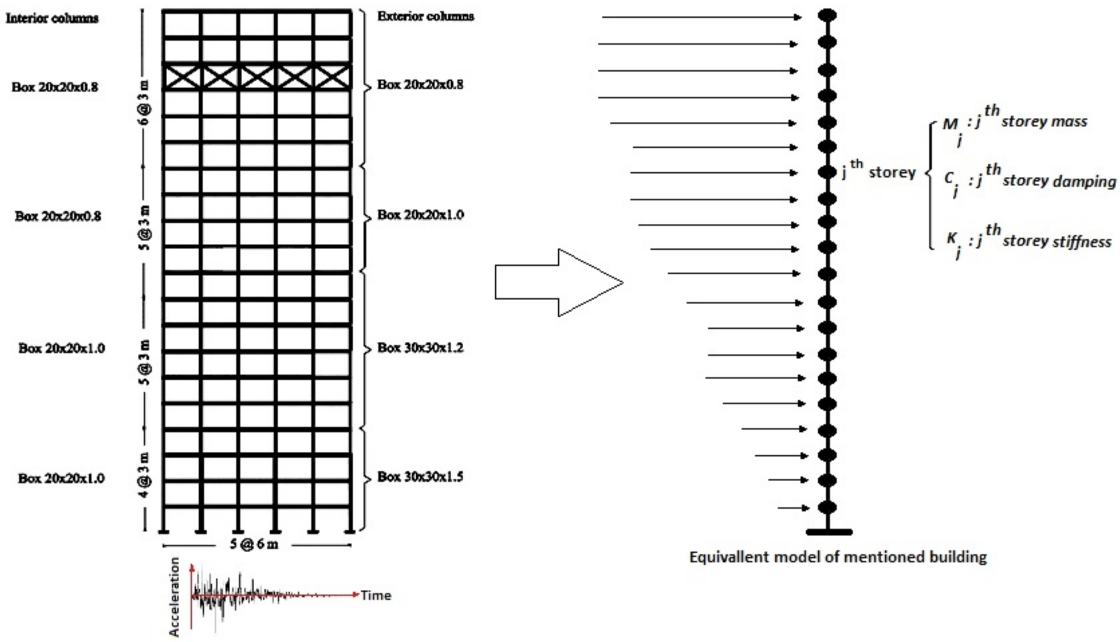

Figure 3. Model of shear building subjected to critical excitation

Table 3: Information on roof displacement under critical excitation for different cases

Case	Arias intensity (m/sec$^{1.5}$)	PGA (m/sec^{2})	The value of roof displacement for the uncontrolled structure (m)
1	5.17	4.66	0.55
2	3.48	4.62	0.19

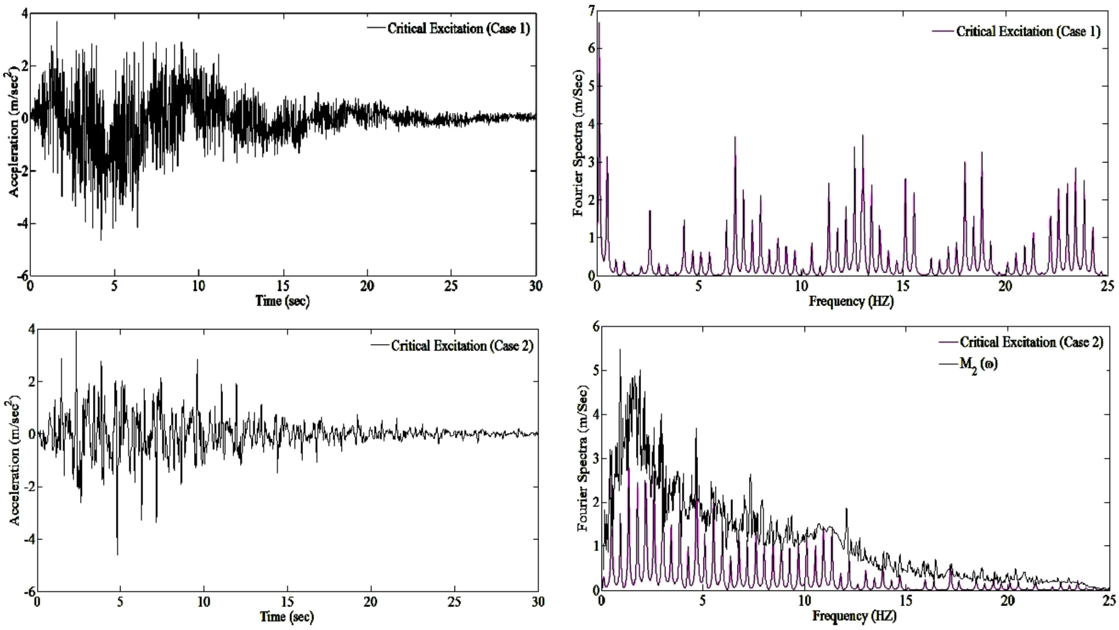

Figure 4. Acceleration time history and Fourier spectra of critical excitation for different cases

These critical accelerations are used to excite actively controlled building. The mentioned building is controlled in two states. In state one that entitled as OA, only one actuator installed on the roof of the structure and in the state two that entitled as MA, there is ten actuators that are installed in even stories. For both of states, the mentioned building is controlled so that the maximum absolute value of the control force reaches to 5, 10 and 30 percent of the total mass of the structure.

The maximum value for roof displacement and displacement time history of uncontrolled and controlled building is shown in Tables (4-5) and Figs. (5-6) for both state, respectively. Fig. 4 and Table 5 show these values for controlled building with maximum absolute value of the control force equal to 5, 10 and 30 percent for state OA and Fig. 5 and Table 6 show these values for controlled building with maximum absolute value of the control force equal to 5, 10 and 30 percent for state MA.

In these tables, the value of roof displacement for maximum absolute value of the control force equal to 5, 10 and 30 percent has been shown as MR5, MR10 and MR30, respectively.

Table 4: Information on roof displacement for controlled building by one actuator installed on the roof of the building under critical excitation for different cases

	Displacement (m)		
Critical Excitation	MR5	MR10	MR30
Case 1	0.5	0.46	0.45
Case 2	0.17	0.14	0.09

Table 5: Information on roof displacement for controlled building by multi actuator installed on the even building' storey under critical excitation for different cases

	Displacement (m)		
Critical Excitation	MR5	MR10	MR30
Case 1	0.44	0.35	0.03
Case 2	0.11	0.05	0.01

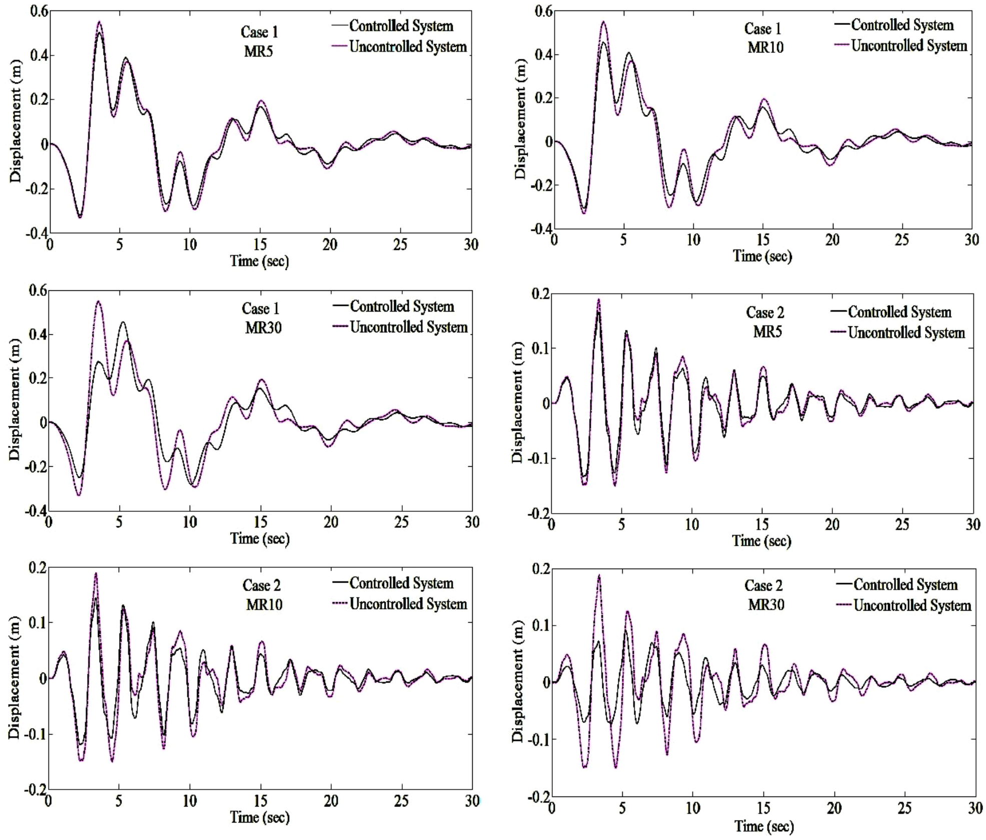

Figure 5. Displacement time history for controlled and uncontrolled of mentioned building subjected to critical excitation for state OA

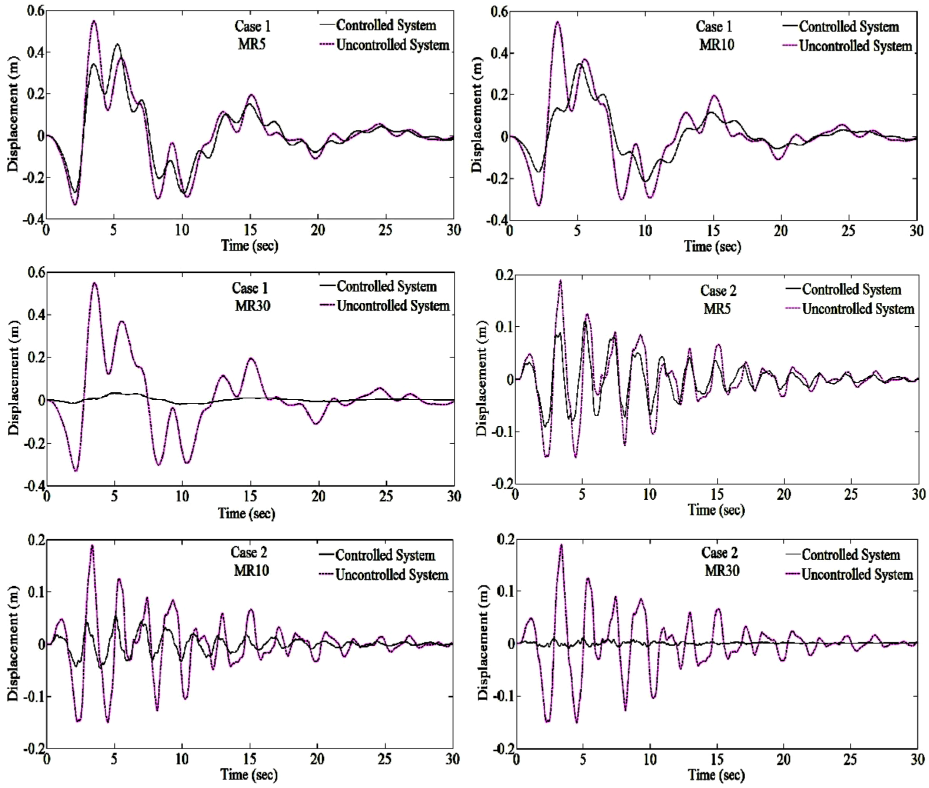

Figure 6. Displacement time history for controlled and uncontrolled of mentioned building subjected to critical excitation for state MA

Based on numerical results obtained, the following observations are made:

1- The frequency content and Fourier amplitude of the critical earthquake are strongly depend on the constraints imposed. If available information on earthquake data is limited to the total energy and PGA, the design input is narrow band (highly resonant) and the uncontrolled structure deformation is conservative (see Figs. 4, 5 and 6 and Table 3). Furthermore, for case 1, as shown in Fig. 4, most of the power of the Fourier amplitude is concentrated at a frequency close to the first natural frequency of the elastic structure ($f = 0.34(Hz)$), while the Fourier amplitudes at other frequencies are low. In addition, by applying additional constraint on the Fourier amplitude spectra in case 2, the Fourier amplitude of the critical acceleration will be distributed across other frequencies. These results match well with earlier work reported by Moustafa, [12].

2- Constraint scenario 1 leads to pulse like ground motion which has been observed during some of the recent earthquakes (e.g., 1971 San Fernando, 1985 Mexico, and 1995 Hyogoken-Nanbu earthquakes). Thus, maximum roof displacement of the structure based on design earthquake is about 5.5 and 1.9 times that of the San Fernando (N69W component) earthquake, for case one and two respectivelly.

3- As shown in Figs. (5-6) and Tables (4-5), increasing the maximum value of control force and the number of actuators decreases the value of roof displacement. In addition, it can be shown from these figures that there are a few reductions in the value of roof displacement for the controlled structure when only one actuator and sensor has been installed on the roof of the building and controlled structure has been excited by critical acceleration. In this case, increasing the maximum value of control force cannot decrease the value of roof displacement for large values. Also, it has been shown that for resonant acceleration (case 1), reduction amount of roof displacement for each level of control force is less than the other critical acceleration (case 2). The maximum value of reduction is restricted to 18.18 percent for case 1 while this value for another acceleration (case 2) is 52.63 percent.

4- As shown in Fig. 6 and Table 5, when the building is controlled by multi actuators installed on the even stories, by increasing the maximum value of control force to 30 percent of total mass of the building, the roof displacement of the structure decreases to small values for critical accelerations (case 1 and case 2).

5- Because of the critical accelerations are computed based on the properties of the structure (mass, damping and stiffness matrices) and imposed constraints, therefore, these accelerations are suitable to control of the structure by decreasing the roof displacement.

5. CONCLUSION

In this paper, using inverse dynamic analysis and nonlinear optimization methods, critical accelerations are estimated on the basis of available information to control an elastic structure at sites having limited earthquake data. A steel building strengthened by a belt truss system is considered and modeled as a two dimensional shear building. By using time history analysis, the critical excitation based on the constraints on ground motion at different time steps is computed so that the absolute value of roof displacement is maximized. Of computed critical accelerations, one of them which produces the maximum roof displacement is selected. Two types of control (one actuator that is installed on the roof and multi actuators that are installed on the even stories) are considered. It shows that when the building is controlled by multi actuators installed on the even stories rather than single actuator on the roof of the building, by increasing the maximum value of control force to 30 percent of total mass of the building, the roof displacement of the structure decreases to small values for critical accelerations. In addition, because of the critical accelerations are computed based on the properties of the structure and imposed constraints, these accelerations are suitable for design of important buildings. Therefore, these critical excitations can be used in controlling of the structure to decrease the roof displacement and therefore eliminating the pounding effect between two adjacent tall buildings. In addition, it shows that the multi actuators that are installed on the even stories have better behavior in decreasing the roof displacement rather than a single actuator installed in the roof. To show the ability of the proposed method, numerical examples are presented. These numerical examples demonstrate validity and efficiency of the proposed method.

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