Modified Roof-Top Garden as a Tuned Mass Damper for Vibration Control of Building Structure Under Earthquake Excitation

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Abstract

Tuned Mass Damper(TMD) is very popular to mitigate the vibration of the structure under any type of random loadings like as Wind or Earthquakes. This research work mainly focuses on to modify the roof- top garden as a passive vibration controlling devices (RTGD) using the same principle of TMD.One of the important features of TMD is using huge mass to create the inertial force against earthquake forces. This passive energy dissipation system is not neededany additional mass because the mass of roof garden used as a TMD mass which is come from the mass of soil and plants. The roof and roof-top garden have been isolated by springs which have been given the spring force and also installed a viscous damper to provide the damping force against earthquake forces. Partially saturated soil condition of roof top garden has been taken to avoid the detuning effect. The obtained structural response under different earthquakes proved that this system is one of most capable to mitigate the Earthquake vibration and can be easily used in practically for building thestructures.

Keywords: Tuned Mass Damper, RTGD, Vibration Control, Earthquake.

1.0. Introduction

The high-rise buildings, long-span bridges, towers and others modern structures are growing up frequently nowadays, those structures are very susceptible to random forces like earthquakes, winds and waves loads. Those forces create excessive vibration whose have a devastating effect on a civil structure that's why the engineers and researchers are a very concern to save the structures and keep the deflection withinthe desired limit. The researcher already invented some controlling devices to reduce the effects of random force effect on a structure like Tuned mass damper (TMD), Tuned Liquid Damper (TLD), Base- Isolator and Others. Among that device, TMD is considered as the most popular and commonly passive control device for mitigating the dynamics response of structures due to effectiveness, robustness and relatively easy installation [1,2]. Although TMDs have been installed in many buildings around the world, such as the CN tower at Toronto, 1975 and Shanghai World Finance Center at Shanghai, 2008, the 660-ton TMD installed at the top of the Taipei Tower at Taiwan, 2004 is considered as the largest and most known TMD [2]. The use of TMDs was studied as a control technique, focusing on the directions of research in the US in structural control [1]. Many investigations have been carried out regarding the mathematical formulations, numerical applications, and response of TMD-controlled systems [3,4]. TMDs are used in buildings not only to control the dynamic response under lateral loads but also to mitigate the torsional behavior of extremely torsional coupled buildings [5,6]. The seismic response of severe torsional coupled buildings was investigated by conducting a large-scale parametric study to obtain the optimum values for the parameters of a TMD system, such as the location of the added mass damper, tuning frequency ratio, tuning mass ratio, and tuned damping ratio [5].

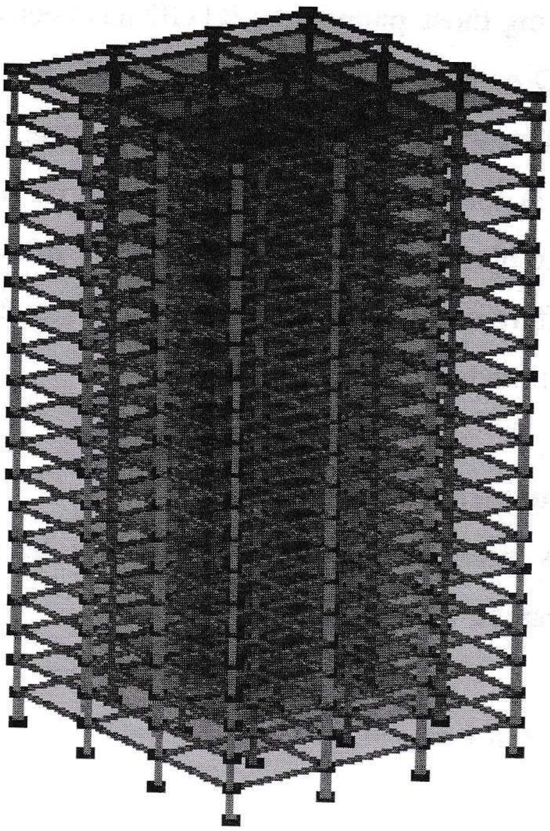


Fig.1: 3D Structural Model of 20-Stories Building by OpenSees

For mitigating wind or earthquake induce vibration Passive TMD [7,8,9] has been applied to control the vibration under random excitation but the uses of this device are rare due to their effectiveness to impulse loads being conditional upon adoption of larger mass ratio [10]. Instead of recurring to cumbersome metal or concrete devices, this paper suggests meeting the conditions by turning to TMDs non-structural masses that are available a top of the buildings.

TMD need huge masses to meet the desired effect of controlling the vibration under earthquake force, this research suggests to use non-structural mass as a TMD mass which is already available on the top of building a structure in form of roof top garden. So, in specific, this is the very promising solution to meet the mass demand of TMD as well as this mass help for gardening to meet some environmental demands.

1.0. Research Methodology

A twenty-story building with roof top garden has been considered for analysis under different Earthquakes.The structural elements and other consideration have been discussed in section 3. To design the roof top garden damper, the natural frequencies and others modelparameters are needed for uncontrolled (without spring and damping device)structure. To effective design of RTGD,first, remove the roof top garden from the top and modal analysis has been done to get modal parameters of theuncontrolled structure. Using those parameters RTGD has been designed and installed (Details in sec 2.2). After design RTGD equation of motions (Details in Sec 2.1) have been developed for controlled(with RTGD damper) structure for analyzing thestructure.Then different earthquakes have been selected (Details in sec2.3) for analyzingthe performance of RTGD. Total analysis has been performed in OpenSees software platform. After gathering result of theanalysis, the performance has been checked by comparing different point of views. Compared results proved that the building with RTGD has been performing well under random vibrations.

2.1.Equation of Motion of Structure with the RTGD

The twenty-story building has been modeledwith RTGD has been shown in figure 1 and the governing equation of motion can be written as:

$$M\ddot{x} + C\dot{x} + Kx = -M\ddot{x_g}(1)$$

where x,$\dot{x}$, $\ddot{x}$ and $\ddot{x_g}$respectively represents the displacement, velocity, acceleration and ground acceleration vectors of the system relative to the base point.

The dimension of the matrix can be presented as $(N + 1) \times 1$ .Where $N$is number of story (Degree of freedom) of uncontrolled structure and (N+1) resepent matrix dimension with RTGD.M,$C$and,Karethe mass, damping and stiffness matrices respectively.

Where the matrix dimension is $(N + 1) \times (N + 1)$.

M,C and K,are as follows:

$$M = \begin{bmatrix} M_{sN\times N} & 0_{N\times 1} \\ 0_{1\times N} & m_{d1\times 1} \end{bmatrix}_{(N+1)\times(N+1)}$$

$$C = \begin{bmatrix} C_{sN\times N} & 0_{N\times 1} \\ 0_{1\times N} & c_{d1\times 1} \end{bmatrix}_{(N+1)\times(N+1)}$$

$$K = \begin{bmatrix} K_{sN\times N} & 0_{N\times 1} \\ 0_{1\times N} & k_{d1\times 1} \end{bmatrix}_{(N+1)\times(N+1)}$$

where, M_s, C_s, and $K_s$are the mass, damping and stiffness matrices of the uncontrolled structure respectively, having a matric dimension of $N \times N$. In addition, m_d,c_d and $k_d$are the garden mass, damping and stiffness, respectively.

2.2. Design of RTGD

The roof top garden dampers are design base on modal parameters. To get the natural frequency and other modal properties of building eigenvector and eigenvalue analysis have been carried out. Moreover, effective modal mass is also obtained from the modal analysis. In this research, only first modal vibration has been considered to control, that's why to design damper stiffness and damping value the first modal properties has been taken. There are three types: Dry soil, partial saturated and fully saturated of the soil of soil condition can happen in different weather condition. In this paper, the partially saturated condition has been considered to calculate the mass of roof top garden. Den Hartog (11) equations is performed to obtain the optimum frequency ratio and damping ratio of the top garden damper with considering the mass of roof top garden under partially saturated soil condition. The Den Hartog equation to get the optimum frequency and damping is provided below:

$$\alpha_{opt} = \frac{1}{1+\mu} \tag{2a}$$

$$\xi_{opt} = \sqrt{\frac{3\mu}{8(1+\mu)}} \tag{2b}$$

Where, μ is the ratio of mass between roof top garden mass and modal mass of 1st mode. α_{opt} is the frequency ratio and ξ_{opt} is the optimum damping ratio.

The required spring stiffness (K_d) and Damping (C_d) is obtained from following equation ,

$$K_d = m_d\left(\alpha_{opt}\omega_s\right)^2 \tag{3}$$

$$C_d = 2\xi_{opt}\left(\alpha_{opt}\omega_s\right)m_d \tag{4}$$

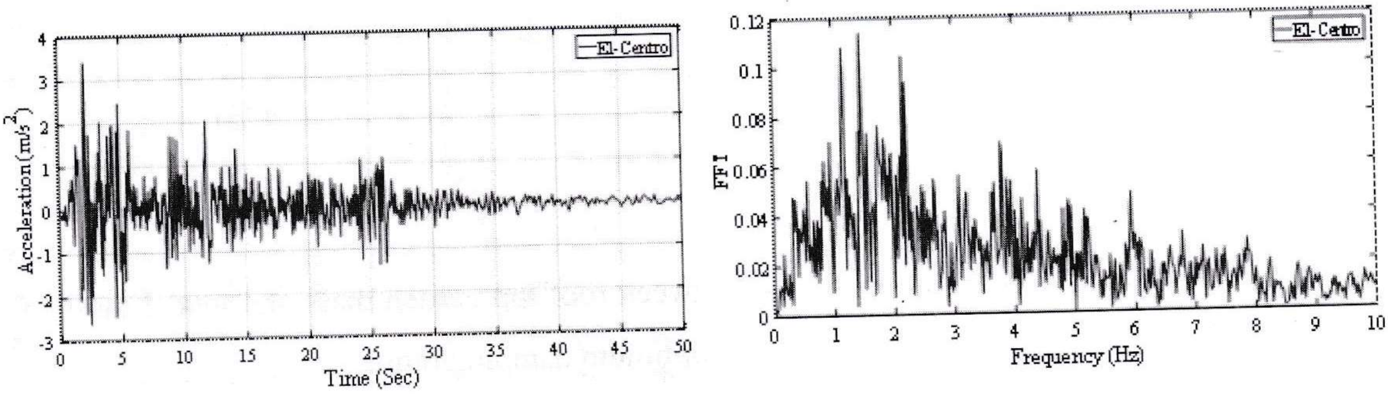
Where, m_d is the mass of roof top garden under partial saturated condition, ω_s is the 1st natural frequency of uncontrolled structure which has been obtained from modal analysis without considering roof top garden.

2.3. Applied of Ground Motion

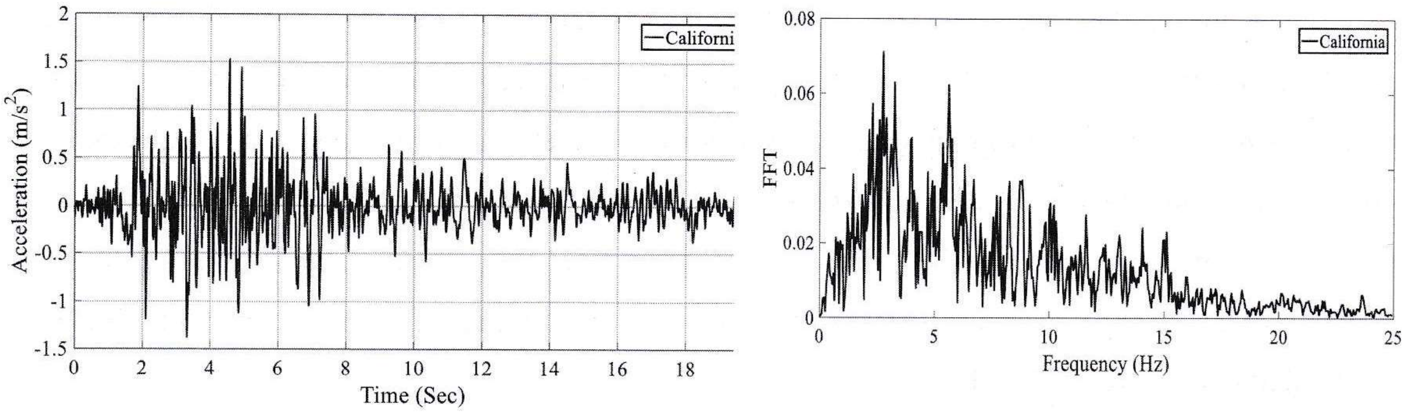
This present study, four types of ground motion as acceleration has been applied to evaluate performance the roof top garden damper systems in the building structure. One of ground motion is Sine wave which has signal frequency 0.332 Hz (1st modal frequency of uncontrolled structure) with amplitude 1. Besides that, three different types of the earthquake, such as El-Centro, California, and North-Ridge are considered. All the acceleration is executed as a time history. The motive behind to apply several earthquakes are the different earthquake contains various frequencies. As a result, every passive controlling system gives the best performance where the time history signal had to optimize the design of the controlling system. Application of ground motion and its PGA and time interval is given below in **Table.1** and **Fig.2** Depicts the time history analysis of applied ground motion.

Table.1 Time history data of ground motion

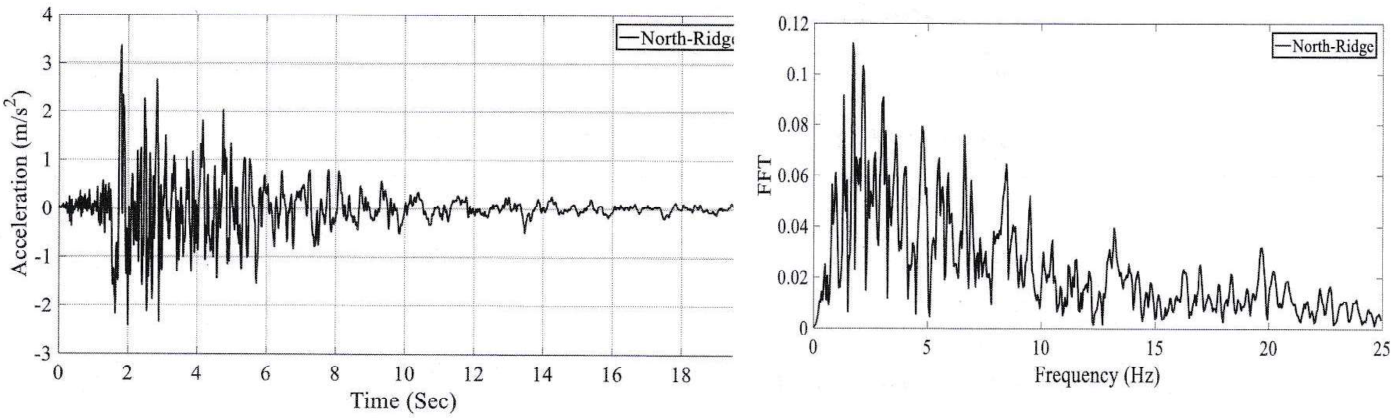
	Sine Acceleration (F=0.332 Hz)	El-Centro Earthquake	North-Ridge Earthquake	California Earthquake
Load steps	4000	2500	2000	2000
Time interval (sec)	0.01	0.02	0.01	0.01
PGA(g)	1	0.348	0.343	0.158



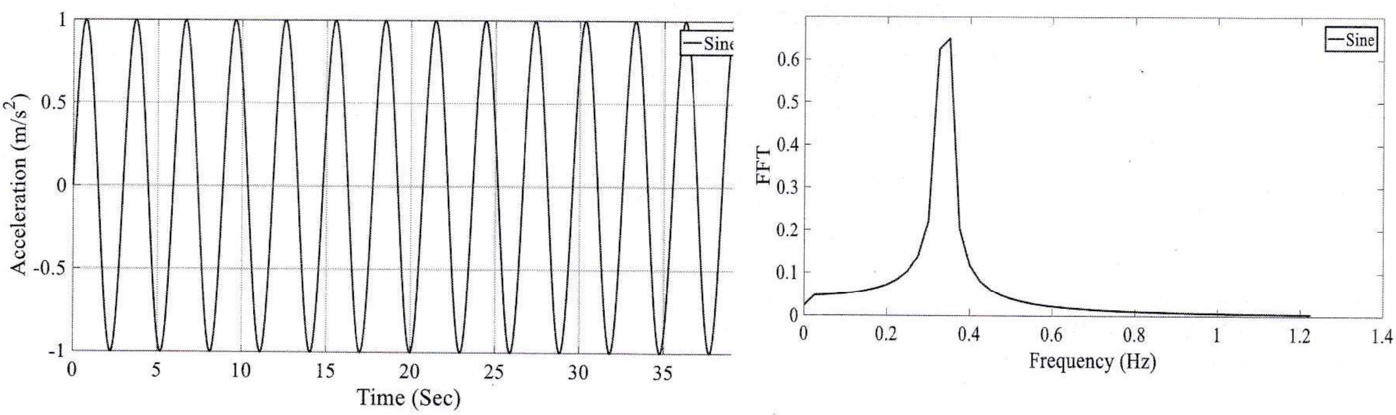
(a) Time history and FFT of El-Centro Earthquake



(b) Time history and FFT of California Earthquake



(c) Time history and FFT of North-Ridge Earthquake



(d) Time history and FFT of Sine Acceleration

Fig. 2: Applied ground motion

3.0. Numerical Example

In this study, a twenty-story building is considered with 5% structural damping ratio to analyze the seismic response. The building has modeled in finite element modeling using OpenSees. Three bays with 6 m and each story height with 3 m has been considered for this structure. Materials property and element details have been provided in Table 2-3. Rayleigh damping(12) approach has been used to calculate the damping of the uncontrolled structure.

From the modal analysis, the 1st modal(natural) frequency is obtained and value is 2.4 rad/sec.To calculate the mass of RTGD partially saturated has been considered which have unit weight 13.22 N/m^3. The RTGD property (mass, spring stiffness and damping) has been calculatedusing the equation 2, 3 and 4and provided in **Table.4**. The total stiffness and damping is divided among the sixteen column joint because in every column joint one spring and damping device has been installed. The spring location is marked in plan view of building which is mention in **Fig.3a** and spring damper shown in **Fig.3b**.

Table.2: Materials Details of structure

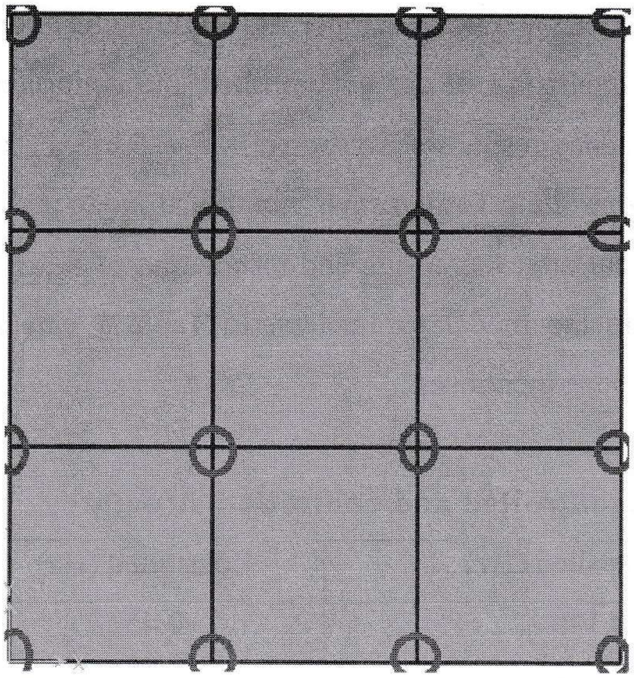
Item	Value	Unit
Modulus of Elasticity(E)	2.486? 1010	Pa
Poisson's Ratio	0.2	-
Density	23563.122	kg/m^3
Shear Modulus G	1.036? 1010	--
Compressive Strength of concrete (fc')	27579032.	Pa

Table.3: Element Details of Structure

Item	Width(mm)	Depth (mm)
Beam	300	500
Column	600	600
Floor Slab or roof	----	162.5
Roof Top Garden slab	----	200

Table.4: Roof Top Garden Dampers properties

Item	RTGD properties
Mass (Kg)	575075
Stiffness(N/m)	2029054
Damping(N-Sec/m)	616969.4

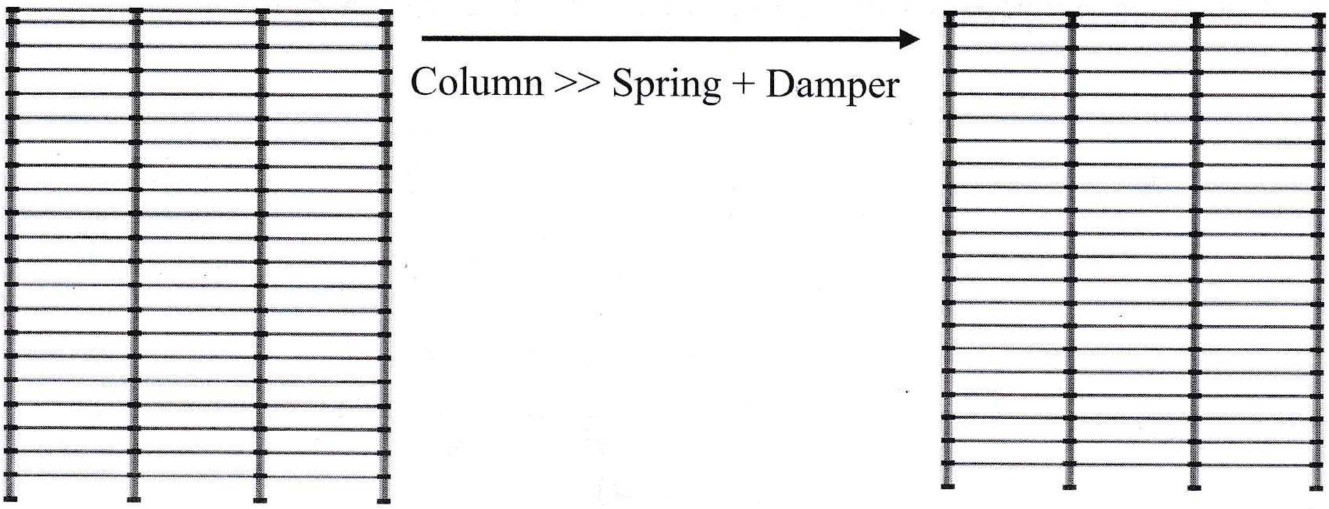


a) Damper locations

b) Damper details in 3D view

Fig.3: Spring damper device locations and details

In **Fig.4**, the building model of Open Sees is represented. **Fig.4(a),** represents an uncontrolled twenty story building with roof top garden. **Fig.4(b),** illustrates a twenty-story building with roof top garden damper as a functioning tuned mass damper (TMD). The columns of roof top garden in uncontrolled structure has been replaced with the springs and viscous dampers. After modeling the structure with and without RTGD the El-Centro, California and Northridge earthquakes has been applied as a time history function. One directional excitation has been performed. Newark's method has been used for time history analysis.



a) Uncontrolled Model

b) Controlled Model

Fig.4: Transfer Model from Uncontrolled to Controlled in OpenSees

4.0. Results and Discussion

To establish the effectiveness of the roof top garden damper (RTGD), the different types of structural responses were compared. The comparisons of the uncontrolled and controlled responses for displacement, base shear force, and frequency response, are shown in Fig. 5 to7. From these figures, it is clear that the RTGD is capable of controlling fundamental vibrational mode under earthquake excitations. A modal analysis was carried out after the design and installation of RTGD, to determine the effect of the RTGD with respect to the modal parameters. In Table 5 natural frequency of uncontrolled and controlled structure has been mentioned.

Table.5: Natural frequencies (Hz) of the uncontrolled and controlled structures.

Mode No.	Uncontrolled (Hz)	Controlled (Hz)
First Mode	0.338	0.454
Second Mode	1.055	1.252
Third Mode	1.885	2.181

Fig.5 shows the displacement response of the structure under sine wave acceleration. Sine wave acceleration has been created with signal frequency 0.3382 Hz which is the first modal frequency to create the resonance effect on the structure. From the resulting displacement, it can be clearly observed that under this signal, the uncontrolled displacement is huge about 277.00 cm under the condition partial saturated roof top garden soil whereas controlled displacement is 32.822 cm. It is clear that this structure cannot be sustained under this sine wave or if the same kind of earthquake will impose to this structure but when the roof top garden is modified and used to control vibration, displacements is mitigated significantly and structure might be sustaining.

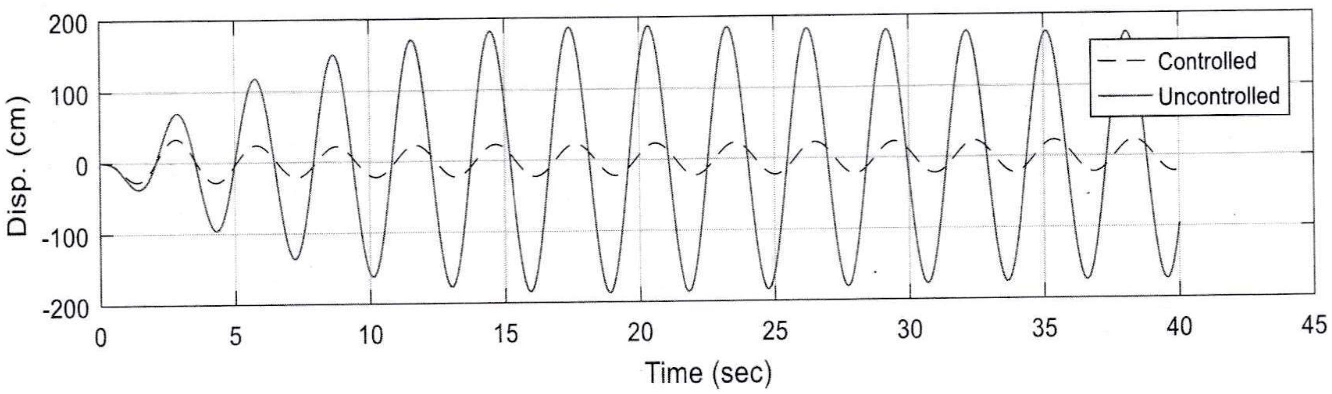
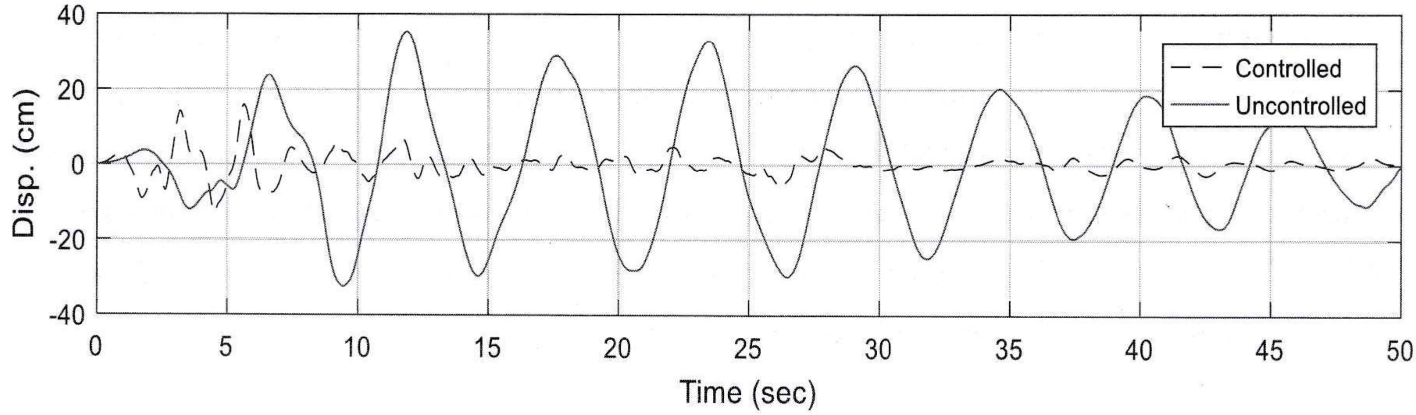
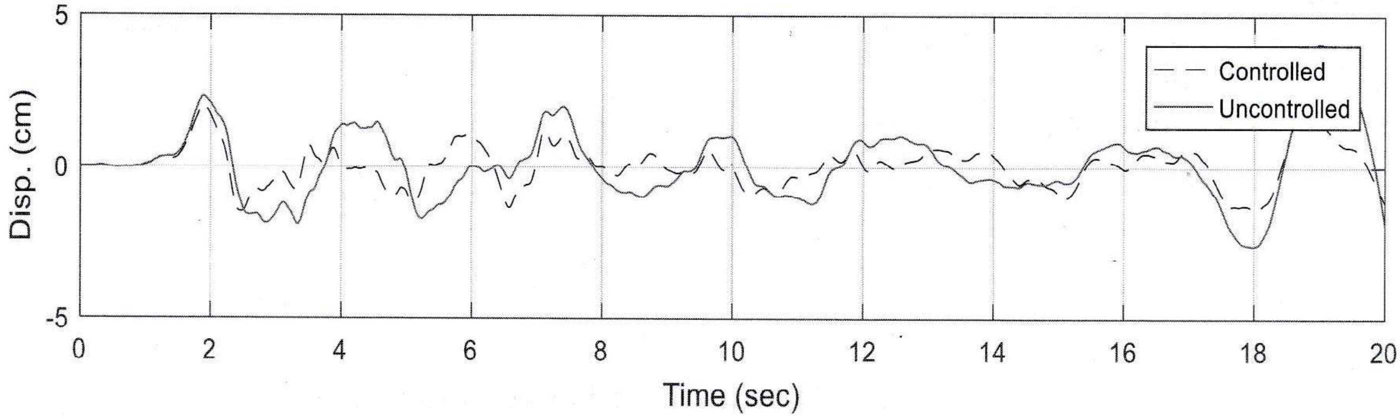


Fig.5: Top floor Displacement under the Sine wave signal with frequency 0.3382 Hz

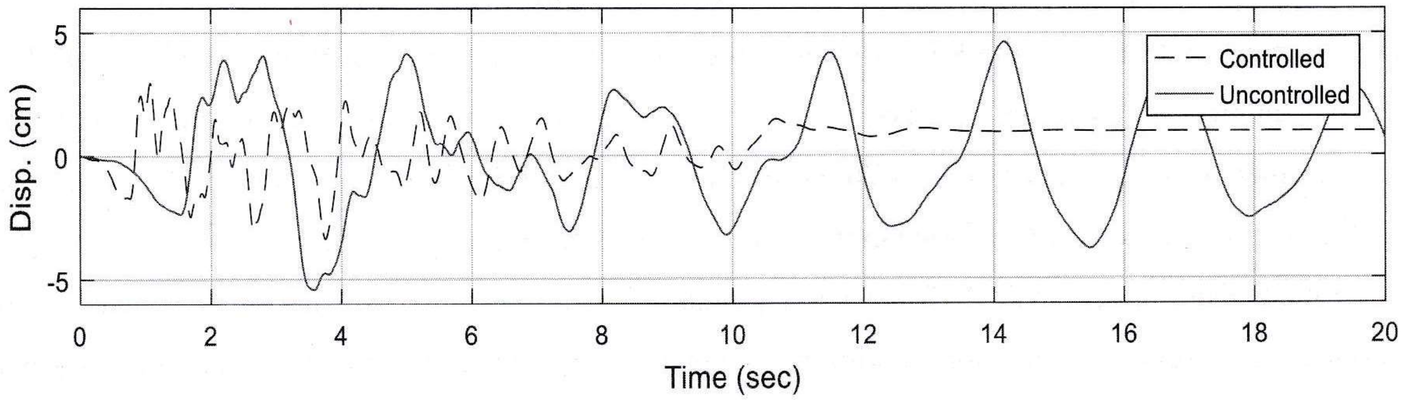
Fig.6 shows the result of top floor displacement for the uncontrolled structure and the structure controlled by the RTGD when subjected to the El-Centro, California, and North-Ridge ground motions. From the obtained results, it was observed that the uncontrolled maximum displacement of the structure was 34.76 cm whereas controlled displacements were 15.96 cm for the El-Centro. And under the California earthquake, the uncontrolled displacement was 4.40 cm, but the controlled displacement was 2.23 cm correspondingly. Moreover, the Northridge earthquake also applied with previously mention conditions to get the dynamic responses of the structure, in that case, the uncontrolled maximum displacement was 5.55 cm and controlled displacement was 3.34cm. The reductions rate of maximum displacement is 54.64 %, 49.52 % and 40.17 % under for the El-Centro, California, and North-Ridge earthquakes, respectively. The average RTGD stroke length was 22.02 cm, 3.59 cm and 5.01 cm which is an acceptable limit for the El-Centro, California, and North-Ridge earthquakes, individually.



a) Top floor Displacement under the El-Centro Earthquake



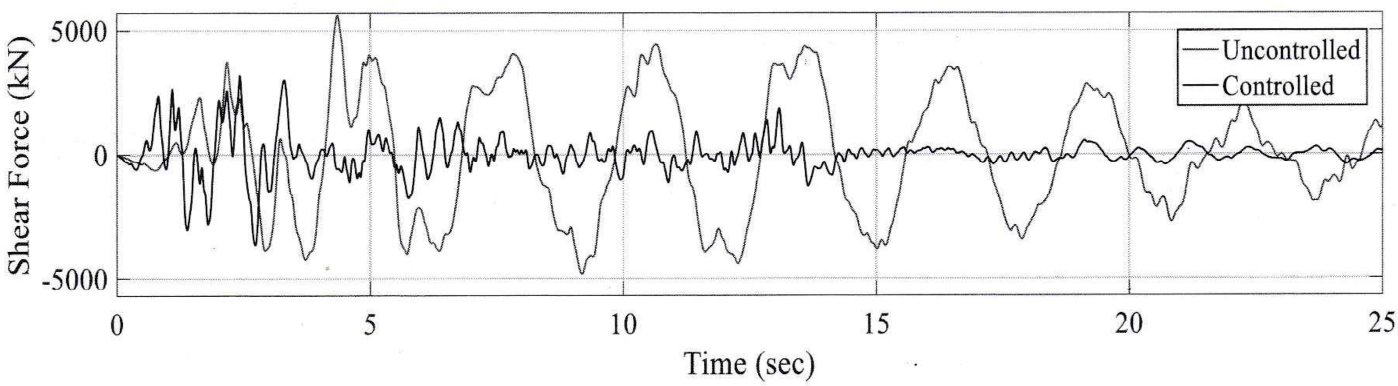
b) Top floor Displacement under the California Earthquake



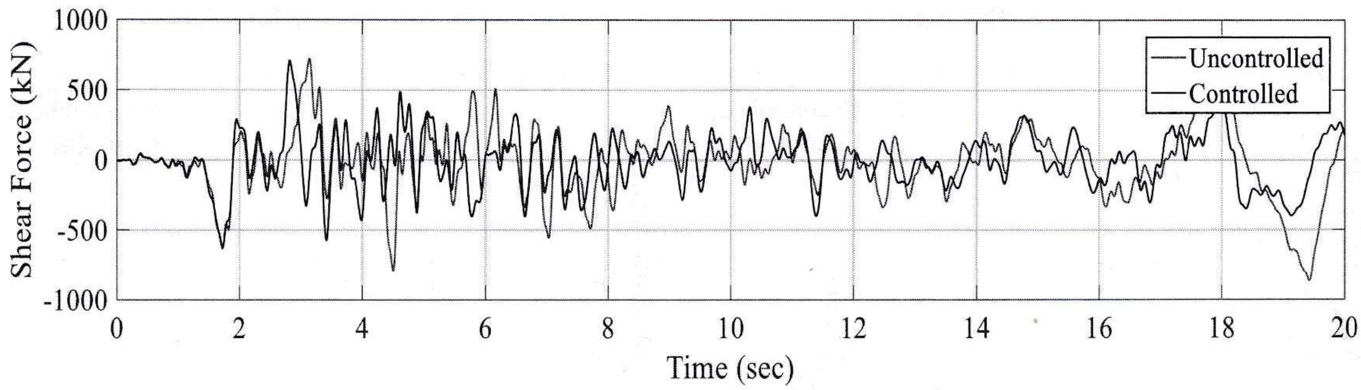
c) Top floor Displacement under the North-ridge Earthquake

Fig.6: Top floor displacement of the Uncontrolled structure and the controlled structure by RTGD under different Earthquakes

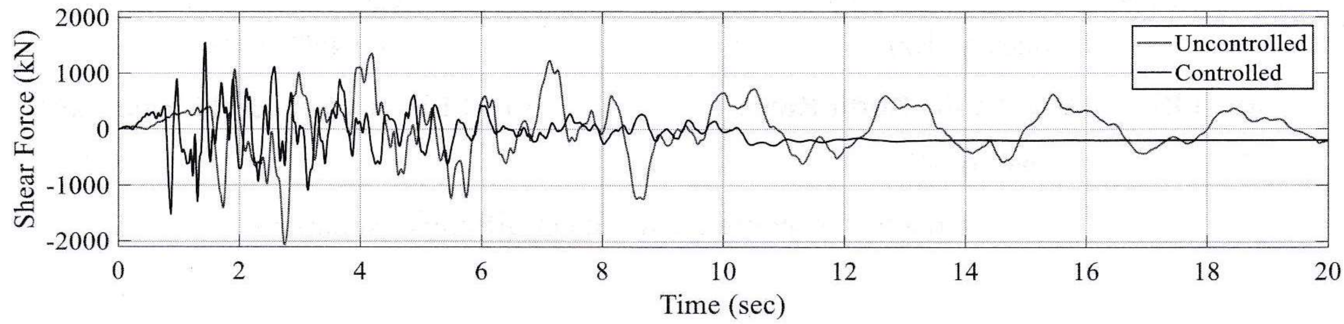
Fig.7 shows the time history response of the base shear for the uncontrolled structure and the structure controlled by RTGD for different earthquake excitations. From the time history response of each curve of the earthquake, it is clear that the RTGD is capable of reducing the base shear force effect of the structure under a broad range of earthquake excitations. The maximum uncontrolled base shear force is 5620 kN, 864.1 kN, and 2060 kN under El-Centro, California and Northridge earthquakes respectively whereas the controlled shear forces are 3641kN, 709.2 kN and 1547 kN. The percent reduction in the maximum base shear force was 32.21%, 18.00 % and 24.90 % for the El-Centro, California, and North-Ridge earthquakes, respectively.



a) Base shear forces under the El-Centro Earthquake



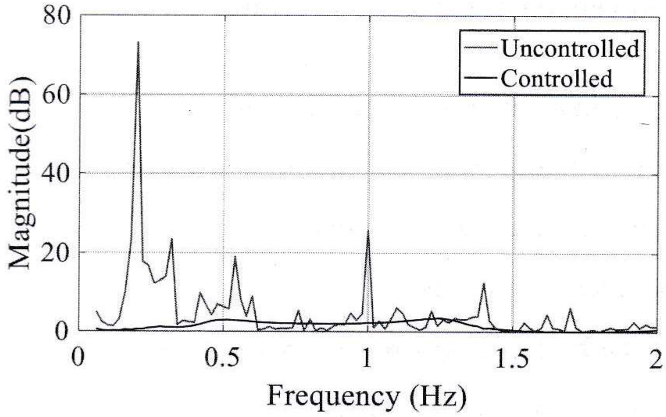
b) Base shear forces under the California Earthquake



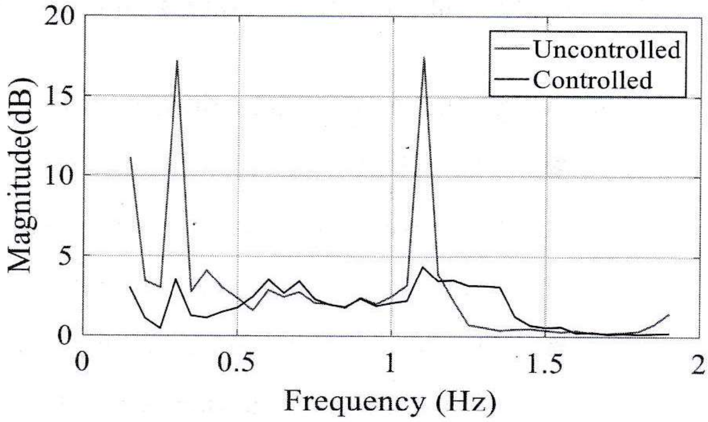
c) Base shear forces under the North-Ridge Earthquake

Fig.7: Base Shear Force comparison of the Uncontrolled structure and the controlled structure under different Earthquakes

Fig.8 shows the results of the acceleration frequency response curve for the uncontrolled structure and the structure controlled by the RTGD under the El-Centro, California and North-Ridge Earthquakes, and Sine wave signal (where the signal frequency 0.3328Hz). The maximum first modal frequency response amplitudes of the uncontrolled structure were 73.09 dB,17.17 dB, **9.30** dB and 11.18 dB, whereas controlled amplitude was 0.95 dB, 3.5 dB, 0.52 dB and 2.06 dB under the El-Centro, California, North-Ridge Earthquakes, and the Sine acceleration, respectively. The performance demonstrated that the RTGD is one of the capable devices of controlling modal frequencies of the uncontrolled structure.



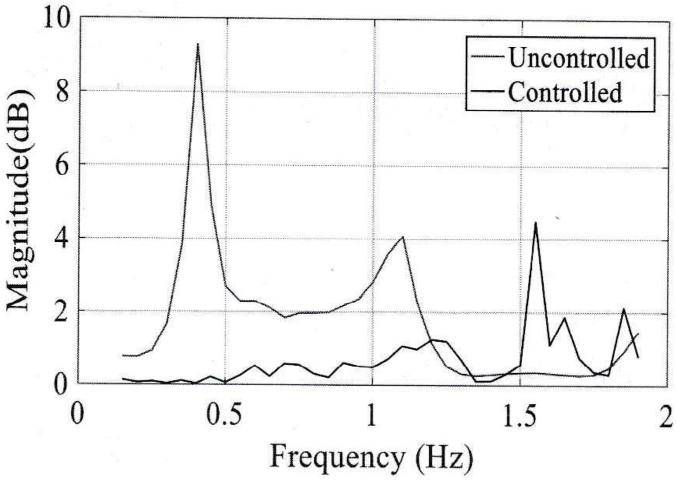
a) FRF curve under the El-Centro



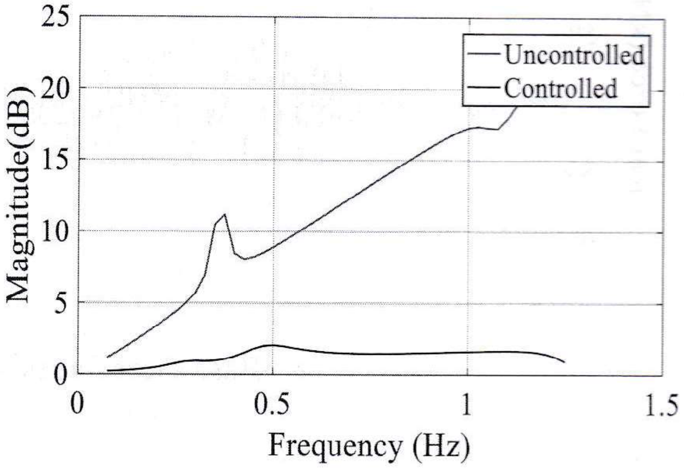
b) FRF curve under the California

earthquake

earthquake



a) FRF curve under the North-Ridge earthquake



c) FRF curve under the Sine wave

Fig.8: Frequency response curves under different excitations

5.0. Conclusions

This work proposed a new vibration control system, which was modified the roof top garden as a Tuned Mass damper which was able to control dominate modal vibrations under random vibration conditions like earthquakes or wind forces. This damper system is called the Roof Top Garden Damper (RTGD). The roof top garden mass is used as a damper mass so additional mass did not require in this system. This system can be served as a roof top garden as well as a damper for mitigating the vibration of building the structure at the same time. The damper stiffness and damping were design under partially saturated soil conditions of roof top garden. The modeling of the structure and the simulation were carried out using OpenSees. Accordingly, the proposed RTGD has a great potential of achieving very satisfactory, innovative vibration control performance. To evaluate the performance of the RTGD, the test results of the structure controlled with the RTGD were compared with those obtained for an uncontrolled version of the same structure.

The following conclusions can be drawn out from the trend of the results of this study.

- The RTGD was significantly more efficient and practical solution for reducing the response of the building. The results indicate there was a remarkable reduction in the maximum top displacement under earthquake excitations.
- The dominant mode shape amplitude of the building was primarily considered to select the locations of the dampers. The design and installation of the RTGD focused on the modal parameters of the first mode which carried about 85 % of the structural mass.

- The rate of response reduction in the uncontrolled top displacement was 54.64 %,49.52 % and 40.17 % for the El-Centro, California and North-Ridge earthquakes, respectively.
- The base shear response reduction rate of the uncontrolled structure was 32.21%, 18.00 % and 24.90 % for the El-Centro, California, and North-Ridge earthquakes, correspondingly.
- The amplitude of the frequency response reduction for first mode frequency was about 98.70%,79.62%, 94.62% and 81.57% under the El-Centro, California, Northridge earthquakes, and the sine wave, respectively. These results clearly indicate that the RTGD is capable of controlling frequency earthquake forces.

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