Performance Improvement of Tall Reinforced Concrete Structures with Multiple Tuned Mass Dampers (MTMD)

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SUMMARY:
This paper aims at investigating the effectiveness of using linear multiple tuned mass dampers (MTMD) to improve performance of tall reinforced concrete structures with nonlinear behavior. A new analytical approach was developed for seismic control of reinforced concrete structures in the inelastic range of deformations. The effect of inelasticity on structural stiffness is discussed along with the procedure employed in formulating local stiffness matrix. Important aspects of inelastic response are presented in terms of member-end springs incorporating post yield behavior, and hysteretic models introducing inelastic member stiffness under reversed cyclic loading caused by seismic excitations. Placement of the multiple TMDs is studied to give the best structural performance. Numerical simulations are performed to study the energy responses of structures with and without TMD installed. The optimization of placement of the multiple TMDs is considered by two different criteria: i) maximum of the peak structural displacement and ii) average Hysteretic dissipated energy.

Keywords: Multiple Tuned Mass Dampers (MTMD), Reinforced concrete structure, Hysteretic model

1. INTRODUCTION

Tuned mass damper is a passive vibration control device that is widely used to reduce the dynamic response of the structure. The main parameters of the TMD which are considered by designers are tuning ratio, mass ratio and damping ratio whereas the tuning ratio which shows the ratio of TMD period to the building period is more important than others. Many researches in elastic domain have been carried out to survey TMD sensitivity to the earthquake. The investigation by Villaverde and Koyoama showed a weakness of TMD. They expressed that TMD affects only on the building performance while the earthquake record is limited to the narrow band frequency and long duration (Villaverde and Koyama 1993). To overcome this weakness, researchers have suggested using MTMD instead of TMD (Jangid 1999, Abé and Fujino 1994). In inelastic domain Wong and Johnson showed that performance of a building equipped with TMD is very sensitive to the earthquake vibration characteristics (Wong and Johnson 2009). It should be mentioned that most researchers have considered the elastic response of structures and the researchers whose researches are related to the inelastic behavior of structures, published their research in the last few years (Wong and Harris 2010, Wong and Johnson 2009). Reviewing the recent researches shows that most buildings investigated are the steel structures with medium height and elasto-plastic behavior whereas the concrete structures with inelastic behavior were not paid attention to very much. Sgobba and Marano obtained optimum parameters of linear tuned mass dampers just for a single degree of freedom concrete structure with nonlinear behavior (Sgobba and Marano 2010). In the present research, the performance improvement of tall reinforced concrete buildings has been investigated while the structural elements are allowed for developing into the inelastic behavior base on Takeda hysteretic model.

2. EQUATION OF MOTION

The equation of motion of a structural system with n degrees of freedom with TMD is expressed in Eqn. 2.1 (Shooshtari 2005):

$$M\,\ddot{x} + C\,\dot{x} + K\,x = E\,\ddot{x}_g(t) \tag{2.1}$$

Where M , C and K are mass, damping and stiffness matrices, respectively and $\ddot{x}$, $\dot{x}$ and x are presented as response acceleration, velocity and displacement, respectively. The ground acceleration is denoted by $\ddot{x}_g(t)$ and and E is a vector representing the degree of freedom in which the ground excitation is applied. To solve Eqn. 2.1, one can change it to the state-space form which is shown in Eqn 2.2:

$$\dot{Z}(t) = AZ(t) + B_r \ddot{x}_g(t) \tag{2.2}$$

Where

$$Z(t) = \begin{bmatrix} x \\ \dot{x} \end{bmatrix};\ \dot{Z}(t) = \begin{bmatrix} \dot{x} \\ \ddot{x} \end{bmatrix};\ B_r = \begin{bmatrix} 0 \\ M^{-1}E \end{bmatrix};\ A = \begin{bmatrix} 0 & I \\ -M^{-1}K & -M^{-1}C \end{bmatrix} \tag{2.3}$$

The dynamic response of structures by solution Eqn. 2.2 is obtained as follows:

$$Z(t) = Td(t-\Delta t) + \frac{\Delta t}{2} B_r \ddot{x}_g(t) \tag{2.4}$$

Where

$$d(t-\Delta t) = e^{\Omega \Delta t} T^{-1} [Z(t-\Delta t) + \frac{\Delta t}{2} B_r\, \ddot{x}_g(t-\Delta t)] \tag{2.5}$$

Where T is a matrix whose columns are eigenvectors of matrix A and

$$\Omega = T^{-1} A T \tag{2.6}$$

The stiffness matrix, K in Eqn. 2.3 remains constant for all steps of time and is not suitable for the investigation of inelastic response. During inelastic response, the structural stiffness changes as inelastic deformations are generated due to element yielding. This is modelled by means of rotational springs introduced at the ends of elements. Therefore, another approach is needed to accommodate yielding in the system and corresponding changes in stiffness K in each step of time. Matrix $\overline{S}(t)$ is introduced for this purpose (Shooshtari 2005):

$$\dot{Z}(t) = \overline{A} Z(t) + B_r \ddot{x}_g(t) + \overline{S}(t) \tag{2.7}$$

Where

$$\overline{A} = \begin{bmatrix} 0 & I \\ 0 & -M^{-1}C \end{bmatrix} \quad ; \quad \overline{S}(t) = \begin{bmatrix} 0 \\ -M^{-1} F_S(t) \end{bmatrix} \tag{2.8}$$

$$F_S(t) = F_S(t-\Delta t) + K(t)\left[x(t) - x(t-\Delta t)\right] \tag{2.9}$$

The solution for Eqn. 2.7 can be expressed as indicated in Eqn. 2.10:

$$Z(t) = \overline{T}\overline{d}(t - \Delta t) + \frac{\Delta t}{2}(B_r \ddot{x}_g(t) + \overline{S}(t)) \tag{2.10}$$

Where $\overline{T}$ is a matrix whose columns are eigenvectors of matrix $\overline{A}$ and;

$$\overline{d}(t - \Delta t) = e^{\overline{\Omega}\Delta t}\overline{T}^{-1}(Z(t - \Delta t) + \frac{\Delta t}{2}(B_r \ddot{x}_g(t - \Delta t) + \overline{S}(t - \Delta t))) \tag{2.11}$$

$$\overline{\Omega} = \overline{T}^{-1}\overline{A}\overline{T} \tag{2.12}$$

3. ELEMENT STIFFNESS MATRIX

When bending moment at the end of a flexural member reaches yield moment M_y, the member develops inelasticity. Subsequently, a plastic hinge is created within this region whose properties can be represented by a rotational spring. Typically, a frame member is modeled by two springs, one at each end. This is shown in Fig. 3.1:

Figure 3.1. Inelastic springs at member ends

Where k_a and k_b are spring stiffnesses at member ends. During elastic response, spring stiffnesses are assigned an infinitely large value so that inelastic deformations are prevented. Upon yielding, the springs are assigned their appropriate values and rotate, developing inelastic deformations. The stiffness matrix is expressed as follows (Holzer 1985):

$$k = \alpha \begin{bmatrix} \frac{AL^2}{I} & 0 & 0 & -\frac{AL^2}{I} & 0 & 0 \\ & 12S_1 & 6LS_2 & 0 & -12S_1 & 6LS_4 \\ & & 4L^2S_3 & 0 & -6LS_2 & 2L^2S_5 \\ & & & \frac{AL^2}{I} & 0 & 0 \\ & Sym. & & & 12S_1 & -6LS_4 \\ & & & & & 4L^2S_6 \end{bmatrix} \tag{3.1}$$

Where

$$\alpha = \frac{EI}{L^3} \ ; \ S_1 = \frac{1}{2S}(S_a + S_b + 4S_bS_b) \ ; \ S_2 = \frac{S_a}{S}(1 + 2S_b) \ ; \ S_3 = \frac{S_a}{2S}(3 + 4S_b) \tag{3.2}$$

$$S_4 = \frac{S_b}{S}(1 + 2S_a) \ ; \ S_5 = \frac{2}{S}(S_aS_b) \ ; \ S_6 = \frac{S_b}{2S}(3 + 4S_a) \ ; \ S = 2(1 + S_a)(1 + S_b) - \frac{1}{2} \tag{3.3}$$

$$S_b = \frac{k_bL}{4EI} \ ; \ S_a = \frac{k_aL}{4EI} \ ; \ k_a = k_b = \frac{6EI}{L} \times \frac{c}{1 - c} \tag{3.4}$$

Assuming a constant post-yield stiffness, consistent with the assumption of bi-linear primary moment-rotation relationship, a fraction of the initial elastic stiffness may be assigned to the post-yield stiffness. This ratio is defined as “c”, as shown in Fig. 3.2. Takeda hysteretic model was employed in the current research (Takeda et al. 1970).

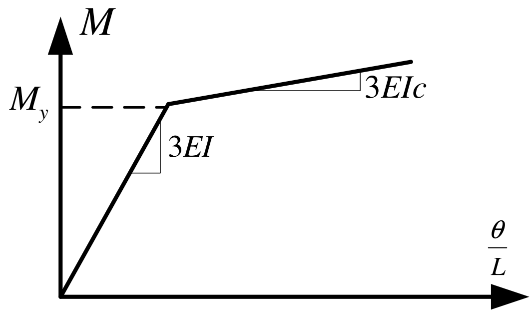


Figure 3.2. Relationship between member-end moment and chord angle before and after yielding.

4. STRUCTURAL MODEL

Three twenty-story moment resistance frames are considered in this investigation. As shown in Fig. 4.1, each frame is 80 meters high by 18 meters wide consisting of three equal bays. All frames are two dimensional R.C buildings whose member sizes as well as rebar reinforcement are presented at Table 4.1.

Table 4. 1. Specifications of Structural Elements

Story	Beam Section (mm^2) and Reinforcement	Column Section (mm^2) and Reinforcement	
		Interior Column	Exterior Column
1 ~ 5	400*500 3#25 top , 2#25 bottom	600*600 16#25	500*500 12#25
6 ~ 7	400*500 2#25 top , 2#20 bottom	600*600 16#25	500*500 12#20
8 ~ 10	400*500 2#25 top , 2#20 bottom	600*600 12#25	500*500 12#20
11	300*500 3#20 top , 2#20 bottom	600*600 12#25	400*400 8#20
12 ~ 15	300*500 3#20 top , 2#20 bottom	500*500 12#20	400*400 8#20
16 ~ 20	300*400 2#20 top , 2#20 bottom	400*400 12#20	400*400 8#20

The difference of these three frames is in the location of TMDs as shown in Fig. 4.1. In Case 1, the frame is equipped with only one TMD the mass of which is 7% of the frame mass and is located at the roof. In Case 2, two TMDs are used; one is located at the roof and the other one is installed at the 10^{th} floor but the total mass of two TMDs is the same as that in Case 1; 7% of the building mass. In Case 3; 14 TMDs are installed from 7^{th} floor to roof whereas their total mass is equal to the previous cases. All beams in three cases except the roof beams are subjected to 29 kN/m uniform gravity loads, including dead and live load whereas the uniform loads of the roof beams are 14 kN/m.

The fundamental period of vibration of twenty-story frame without TMD is 4.96 second and the tuning period of each TMD is set to be equal to the fundamental period of buildings. All frames are subjected to the Kobe earthquake record which is shown in Fig. 4.2. The damping ratio of frames as well as the damping ratio of each TMD is set to 5% and it is assumed that all TMDs remain in linear behaviour during earthquake vibration.

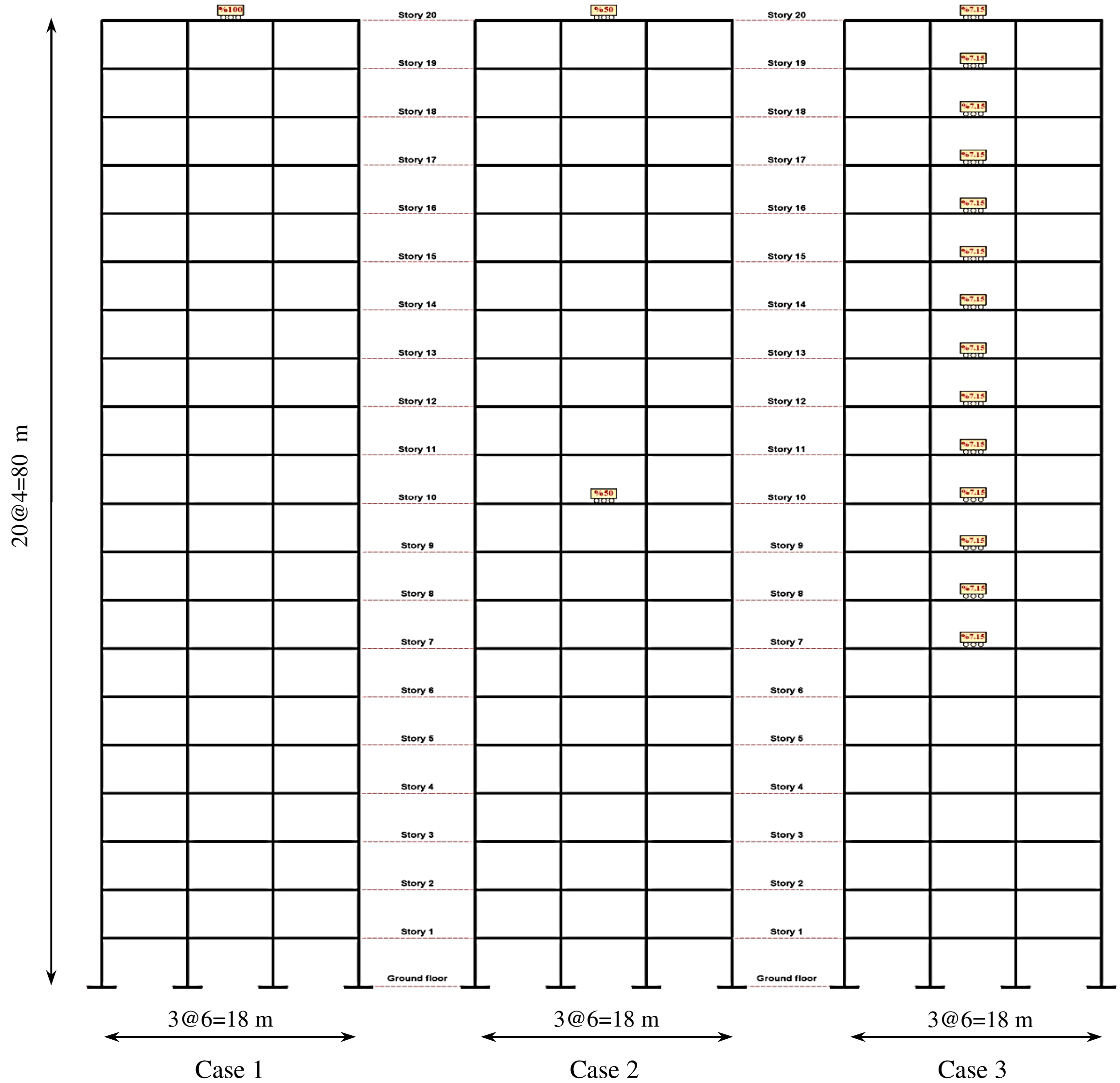


Figure 4.1. Twenty-story moment resistance frame showing location of TMDs

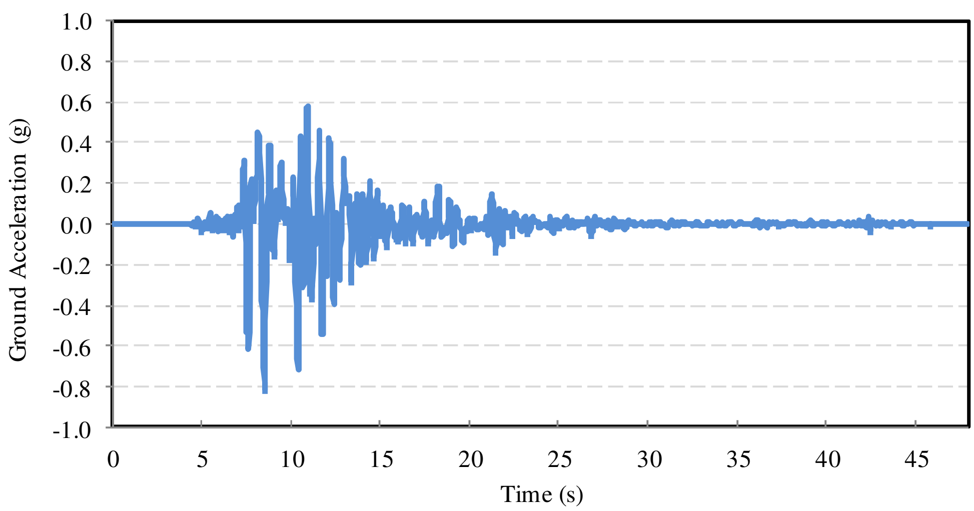
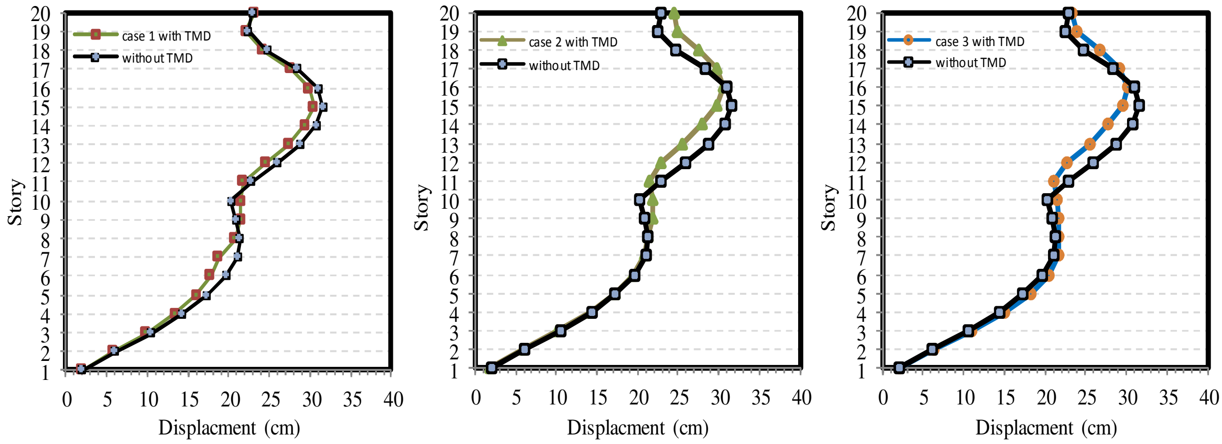


Figure 4. 1. Kobe earthquake record, Japan.

5. RESULTS

Three frames which were described in the previous section were analyzed to compare their performance. The first criterion which should be considered is the maximum lateral displacement of floors. Fig. 5.1 shows the maximum lateral displacement of each case compared with the lateral displacement of the frame without TMD. To have better idea about the performance of these three cases, Fig. 5.2 is presented which shows the displacement of all three cases just in one figure.



Case 1 Case 2 Case 3

Figure 5.1. Comparing maximum lateral displacement in three cases.

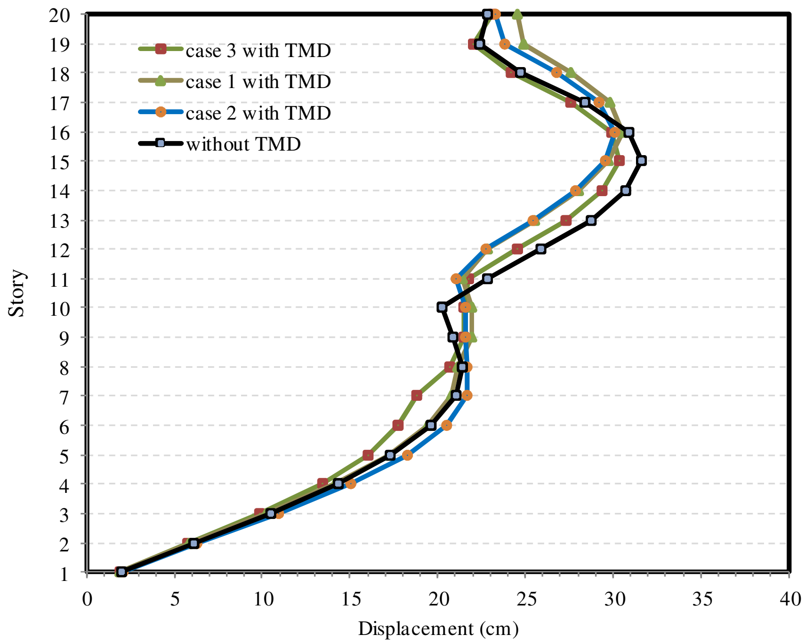


Figure 5.2. Comparing maximum lateral displacement for all cases

The second criterion which can show the performance of TMD is the total hysteretic dissipated energy in structure due to earthquake vibration. The total amount of that energy as well as the percentage of increasing/decreasing of both criteria is presented in Table 5.1 through Table 5.3.

Table 5.1. Maximum of Lateral Displacement and Plastic Energy Dissipation for case 1

Maximum of	With TMD	Without TMD	Percent of Increase	Percent of Decrease
Lateral Displacement (m)	0.305	0.316	-	4.7%
Total Plastic Energy dissipation (KJ)	464.77	502.51	-	7.5%

Table 5.2. Maximum of Lateral Displacement and Plastic Energy Dissipation for case 2

Maximum of	With TMD	Without TMD	Percent of Increase	Percent of Decrease
Lateral Displacement (m)	0.312	0.316	-	3.5%
Total Plastic Energy dissipation (KJ)	456.33	502.51	-	9.2%

Table 5.3. Maximum of Lateral Displacement and Plastic Energy Dissipation for case 3

Maximum of	With TMD	Without TMD	Percent of Increase	Percent of Decrease
Lateral Displacement (m)	0.304	0.316	-	3.8%
Total Plastic Energy dissipation (KJ)	467.40	502.51	-	7.0%

6. CONCLUSION

In this research, the performance of three tall concrete buildings is considered. The difference of these cases is just in the location of TMDs. Based on the presented results one can conclude that using TMDs can create a reduction in the lateral displacement. However, the amount of the reduction is not significant, but the results show that using just one TMD in the roof has better performance compared with the other cases. These results also indicate that the main purpose of using TMDs is to reduce the plastic energy dissipation which causes the buildings to have better performance in earthquake event.

At the bottom line, it should be mentioned that the main advantage of this research is to consider inelastic behavior of R.C. buildings which has not paid attention to much.

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