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Optimum tuned mass damper approaches for adjacent structures

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Abstract. Pounding of adjacent structures are always a notable reason for damages after strong ground motions, but it is already unforeseen detail in newly constructed structures. Thus, several approaches have been proposed in order to prevent the pounding of structures. By using optimally tuned mass dampers, it is possible to decrease the displacement vibrations of structures. But in adjacent structures, the response of both structures must be considered in the objective function of optimization process. In this paper, two different designs of Tuned Mass Dampers (TMD) are investigated. The first design covers independent TMDs on both structures. In the second design, adjacent structures are coupled by a TMD on the top of the structures. Optimum TMD parameters are found by using the developed optimization methodology employing harmony search algorithm. The proposed method is presented with single degree of freedom and multiple degree of freedom structures. Results show that the coupled design is not effective on multiple degree of freedom adjacent structures. The coupled design is only effective for rigid structures with a single degree of freedom while the use of independent TMDs are effective on both rigid and flexural structures.

Keywords: adjacent structures; structural control; tuned mass damper; optimization; harmony search; pounding

1. Introduction

During strong ground motions, adjacent structures may suffer from the damages resulted from pounding if the seismic gap between the structure blocks is insufficient. In addition to the damages at the point of collision, one or both structures may collapse because of unexpected impact forces and changing seismic behavior of coupled adjacent structures.

Pounding of adjacent structures can be described under five major types (Jeng and Tzeng 2000). These five major types are mid-column ponding, heavier adjacent building pounding, taller adjacent building pounding, eccentric pounding, en building pounding. Mid-column pounding is the most seen type after earthquakes. In this type, if the floor levels of adjacent buildings are not at the same level, the slab of the structure may collide to the column of the other structure. The big impact forces directed from the slab to the column may cause great damage on the column. Since the impact force is directed from a heavier and more rigid element than column, this types of pounding is also called as hammering. In heavier adjacent building pounding, the impact force

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directed from the heavier structure may cause big lateral movement for the lighter structure. If the adjacent structures have very different heights, the lower one may prevent the sway of the taller structure if the seismic gap between them is insufficient. Because of this pounding, the high shear force may occur at the point of the collision. Pounding of adjacent structures are not always occur from lateral movements. In eccentric pounding, torsional story movement may cause damages at the corners of the structures. In the series of adjacent buildings, the structures at the end of the series may suffer big lateral movements because of the pendulum effect resulting from the pounding of the other structures.

In order to prevent the pounding of adjacent structures, several passive, active and semi-active control applications have been proposed by connecting adjacent structures. Adjacent buildings interconnected with nonlinear hysteretic dampers were proposed by Ni *et al*. (2001) for analyzing random seismic response. In modeling process, two adjacent buildings were idealized as multi degree of freedom (MDOF) elastic structures and the hysteretic dampers were modeled as Bouc-Wen (BW) differential model. Numerical studies show that the approach is effective on minimizing random seismic response of adjacent buildings. The stochastic optimal coupling-control of adjacent buildings by using a reduced-order model of coupled structures connected with control devices at any floors was conducted by Ying et al. (2003) and the non-linear optimal coupling-control law was determined by the dynamical programming equation in the analyses. Pounding of base isolated structure with fixed based structure is also an important research area. According to Polycarpou and Komodromos (2010), these structures may also collide due to deformations of structures even if the seismic gap at the base is securely provided. Bharti et al. (2010) proposed Magnetorheological (MR) dampers for connecting adjacent structures in order to prevent the pounding of buildings. The orientation of MR dampers and maximum voltage for semi-active control were also investigated. Xu and Zhang (2002) connected adjacent buildings with hydraulic actuators controlled by Linear Quadratic Gaussian (LQG) controllers in order to increase the modal damping ratios of the system and the improvement of the seismic responses of adjacent buildings is depending on the selection of the parameters of LQG controllers. Bumper-type collision shear walls proposed by Anagnostopoulos and Karamaneas (2008) is also a method of protection from the pounding of adjacent structures. By using these walls, the structures can be protected from impulsive acceleration pulses resulting from the pounding. According to the study, the approach is effective on seismic response although local and repairable damage can be seen on the shear walls at the collision points. Matsagar and Jangid (2005) proposed viscoelastic dampers in order to connect adjacent structures. Different combinations of fixed and base-isolated structures were investigated and the proposed approach is the most effective for adjacent fixed based and base isolated structures. Sheikh *et al*. (2012) proposed the use of MR dampers in order to reduce pounding effect of base isolated multi-span reinforced concrete highway bridges and different control strategies were investigated. Park and Ok (2012) proposed a preference-based optimum design approach for multi objective optimization of actively controlled adjacent structures in order to find the best balance between performance and cost. Rubber shock absorbers can be also used in reducing the pounding effect of adjacent structures. Polycarpou *et al*. (2013) developed a nonlinear impact model for rubber shock absorbers for earthquake induced poundings. Cimellaro and Lopez-Garcia (2011) developed a two-stage design algorithm for the optimization of adjacent structures connected with passive dissipation devices. The algorithm determines an active control law and modifies damper and stiffness coefficients according the difference between the responses of the active and an equivalent passive control system. By controlling damping force of MR dampers, a hybrid control approach was proposed for seismic response control of adjacent buildings by Kim and Kang (2011). Tubaldi *et al*. (2012) proposed a probabilistic performance-based procedure methodology which was incorporated into a performance-based earthquake engineering approach and the method was illustrated on adjacent structures including the use of viscous dampers with different retrofit schemes. Cundumi and Suarez (2008) connected adjacent structures with a Variable Damping Semi-Active (VDSA) control system which contains two dampers connected to the structures by the upper ends and driven by a dynamic actuator. Bigdeli *et al*. (2012) investigated the optimum configuration of passive dampers which couples adjacent structures. Trombetti and Silvestri (2007) investigated the implementation of the Mass Proportional Damping (MPD) system in buildings by direct implementation and indirect implementation (adjacent structures). The optimum insertion of viscous dampers are also provided from the results.

The aim of this paper is to minimize the seismic gap between adjacent structures by using optimum tuned mass dampers. In densely inhabited metropolitan areas, minimization of the seismic gap is important for economical purposes. Also, implementing a tuned mass damper (TMD) on the top of an adjacent structure with inadequate seismic gap is a practical retrofit application. In this paper, two different approaches of TMD design for adjacent structure are proposed. In one of the designs, adjacent structures are connected to each other by using a TMD with different damping and stiffness properties at the connection points. The other approach is to implement independent TMDs on both structures without connecting them. Optimum TMD parameters were obtained for both approach by using a metaheuristic algorithm called harmony search. Optimizing TMDs for adjacent structures is more complicating process than tuning a TMD for a structure. Optimum parameters of a TMD is dependent to seismic response of structure. For adjacent structures, seismic response of both structures must be taken into consideration for the optimum design of TMDs preventing seismic pounding and minimizing seismic gap. In numerical example, Single Degree of Freedom (SDOF) structures with different periods and Multiple Degree of Freedom (MDOF) structures were investigated.

2. Harmony search algorithm

Metaheuristic algorithms, which are developed for finding optimum values of scientific and mathematical problems, are inspired by natural phenomena such as observations of animal species, natural happenings, scientific theories and processes. Harmony Search (HS) algorithm developed by Geem *et al*. (2001) was produced by the observations of musical performances. The final aim of the musical performances and optimization process is similar to each other. In an engineering problem, the designer must find a solution that provides safety and economic conditions at the same time for the users of the project. Similarly, a musician searches a better harmony to gain admiration and enjoyment of audience.

Especially, when derivative of mathematical functions cannot be analytically calculated, HS is a frequently used method. It uses a stochastic random search instead of a gradient search (Geem 2008). The algorithm is suitable for solving problem with discrete and continuous variables (Lee *et al*. 2005; Lee and Geem 2005).

In order to find the optimal solution quickly without entrapped to local optimums, three different options of musical performance is adapted to the algorithm (Yang 2010). Playing any famous part of the music from the memory of the musician is the first option. This option is imitated by the usage of Harmony Memory (HM) matrix, in which the possible optimum solutions are stored. In the second option, the musician can play a part of music similar to the famous one. With this option, the musician tries to find best harmony. In HS algorithm, a new harmony vector can be generated from the values around the existing ones in HM matrix. The third option is to play something randomly. Randomization is the source of creation of values in HS algorithm. The HS algorithm searches new values around the existing vectors with a possibility but it also searches the whole domain for better results. Thus, the local optima problem is prevented.

The HS algorithm has been already used for civil engineering problems including the areas of structural analysis (Lee and Geem 2004; Akin and Saka 2010; Hasancebi *et al*. 2010; Togan *et al*. 2011; Erdal *et al*. 2011; Bekdas and Nigdeli 2013a; Toklu *et al*. 2013), hydraulics (Geem 2009; Geem and Cho 2011; Geem 2011), construction cost and management (Geem 2010) and structural vibration control (Bekdaş and Nigdeli 2011; 2012; 2013b; 2013c; Nigdeli and Bekdaş 2013).

3. Tuned mass dampers

The initial form of tuned mass damper (TMD) without inherent damping is the vibration absorber device invented by Frahm (1911). After this invention, the inherent damping was added to device by Ormondroyd and Den Hartog (1928) and the classical TMD was formed. The use of TMDs in civil engineering includes several practical applications. In order to surpass structural vibrations resulting from external excitations like turbulent winds and strong ground motions, TMDs were installed to Citigroup Centre (New York), Trump World Tower (New York), Taipei 101 (Taipei), LAX Theme Building (Los Angeles), Berlin TV Tower (Berlin) and other important civil structures including high-rise buildings, towers and bridges. Since wind and earthquake excitations have randomly changing frequencies, optimally tuning of TMDs for civil structures is an important and developing research area. Several approaches for tuning of TMDs have been proposed (Den Hartog 1947; Warburton 1982; Sadek *et al*. 1997; Chang 1999) including the use of metaheuristic algorithms (Bekdaş and Nigdeli 2011; 2012; 2013a; 2013b; Nigdeli and Bekdaş 2013; Hadi and Arfiadi 1998; Singh *et al*. 2002; Marano *et al*. 2010; Steinbuch 2011; Leung *et al*. 2008; Leung and Zhang 2009).

In the previous HS approaches for TMD, the aim of the optimization is to reduce displacements (Bekdaş and Nigdeli 2011; 2013a; 2013b) and base shear force (Nigdeli and Bekdaş 2013) of a single structure. Only a TMD is proposed for a slender structure between two structures in the previous TMD optimization approach for adjacent structures (Bekdaş and Nigdeli 2012). In this study, TMDs on both adjacent structures are optimized for two different designs. In one of the designs, all structures contain an independent TMD (design1). In the other approach, two adjacent structures are connected to each other with an optimum TMD (design2).

The equations of motion of two N-story adjacent structure (Fig.1) with independent TMD on the top floor can be written as

$$\begin{aligned} \mathrm{M}\ddot{\mathrm{x}} + \mathrm{C}\dot{\mathrm{x}} + \mathrm{K}\mathrm{x} &= -\mathrm{M}\{1\}\ddot{\mathrm{x}}_{\mathrm{g}} \\ M\ddot{x} + C\dot{x} + Kx &= -M\{1\}\ddot{\mathrm{x}}_{\mathrm{g}} \end{aligned} \tag{1}$$

for earthquake excitation. In the equations, italic terms represent the parameters of the second structure while straight terms are corresponding to the first structure. M, C and K matrices are diagonal lumped mass (Eq. (2)), damping (Eq. (3)) and stiffness matrices (Eq. (4)), respectively.

Vector of structural displacements (Eq. (5)), base acceleration excitation and a vector of ones with a dimension of (N+1,1) are shown with x,   and {1}, respectively.

$$M=\mathrm{diag}[m_1\ m_2\ \ldots\ m_N\ m_d] \quad M=\mathrm{diag}[m_1\ m_2\ \ldots\ m_N\ m_d] \tag{2}$$

$$C=\begin{bmatrix} (c_1+c_2) & -c_2 & & & & & \\ -c_2 & (c_2+c_3) & -c_3 & & & & \\ & . & . & & & & \\ & . & . & . & & & \\ & & . & . & . & & \\ & & & & -c_N & (c_N+c_d) & -c_d \\ & & & & & -c_d & c_d \end{bmatrix}$$

$$C=\begin{bmatrix} (c_1+c_2) & -c_2 & & & & & \\ -c_2 & (c_2+c_3) & -c_3 & & & & \\ & . & . & & & & \\ & . & . & . & & & \\ & & . & . & . & & \\ & & & & -c_N & (c_N+c_d) & -c_d \\ & & & & & -c_d & c_d \end{bmatrix} \tag{3}$$

$$K=\begin{bmatrix} (k_1+k_2) & -k_2 & & & & & \\ -k_2 & (k_2+k_3) & -k_3 & & & & \\ & . & . & & & & \\ & . & . & . & & & \\ & & . & . & . & & \\ & & & & -k_N & (k_N+k_d) & -k_d \\ & & & & & -k_d & k_d \end{bmatrix}$$

$$K=\begin{bmatrix} (k_1+k_2) & -k_2 & & & & & \\ -k_2 & (k_2+k_3) & -k_3 & & & & \\ & . & . & & & & \\ & . & . & . & & & \\ & & . & . & . & & \\ & & & & -k_N & (k_N+k_d) & -k_d \\ & & & & & -k_d & k_d \end{bmatrix} \tag{4}$$

$$x=[x_1\ x_2\ \ldots\ x_N\ x_d]^T \quad x=[x_1\ \ x_2\ \ldots\ x_N\ \ x_d]^T \tag{5}$$

In Fig. 1 and Eqs. (2)-(5), m_i, c_i, k_i and x_i are mass, damping coefficient, stiffness coefficient and horizontal displacement of i^{th} story of structure. The properties of the TMD are shown with m_d (mass), c_d (damping coefficient) and k_d (stiffness coefficient). The displacement of the TMD is shown as x_d.

For the second design of TMD (Fig.2), the coupled equation of motion of two N-story adjacent structures connected with a single TMD on the top floor can be written as

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = -\mathbf{M}\{1\}\ddot{x}_g \tag{6}$$

for earthquake excitation. **M**, **C** and **K** matrices are diagonal lumped mass (Eq. (7)), damping (Eq. (8)) and stiffness matrices (Eq. (9)) of the coupled system, respectively. Vector of structural displacements (Eq. (10)), base acceleration excitation and a vector of ones with a dimension of (N+1,1) are shown with **x**, $\ddot{x}_g(t)$ and {1}, respectively.

$$M = \mathrm{diag}[m_1\, m_2\, \dots\, m_N\, m_d\, m_N\, \dots\, m_2\, m_1] \tag{7}$$

$$\mathbf{C} = \begin{bmatrix} (c_1+c_2) & -c_2 & & & & & & & & & \\ -c_2 & (c_2+c_3) & -c_3 & & & & & & & & \\ & . & . & & & & & & & & \\ & . & . & . & & & & & & & \\ & & . & . & . & & & & & & \\ & & & -c_N & (c_N+c_d) & -c_d & & & & & \\ & & & & -c_d & c_d+c_d & -c_d & & & & \\ & & & & & -c_d & (c_N+c_d) & -c_N & & & \\ & & & & & & & . & . & . & \\ & & & & & & & & . & . & . \\ & & & & & & & & & . & . \\ & & & & & & & & & -c_3 & (c_2+c_3) & -c_2 \\ & & & & & & & & & & -c_2 & (c_1+c_2) \end{bmatrix} \tag{8}$$

$$\mathbf{K} = \begin{bmatrix} (k_1+k_2) & -k_2 & & & & & & & & & \\ -k_2 & (k_2+k_3) & -k_3 & & & & & & & & \\ & . & . & & & & & & & & \\ & . & . & . & & & & & & & \\ & & . & . & . & & & & & & \\ & & & -k_N & (k_N+k_d) & -k_d & & & & & \\ & & & & -k_d & k_d+k_d & -k_d & & & & \\ & & & & & -k_d & (k_N+k_d) & -k_N & & & \\ & & & & & & & . & . & . & \\ & & & & & & & & . & . & . \\ & & & & & & & & & . & . \\ & & & & & & & & & -k_3 & (k_2+k_3) & -k_2 \\ & & & & & & & & & & -k_2 & (k_1+k_2) \end{bmatrix} \tag{9}$$

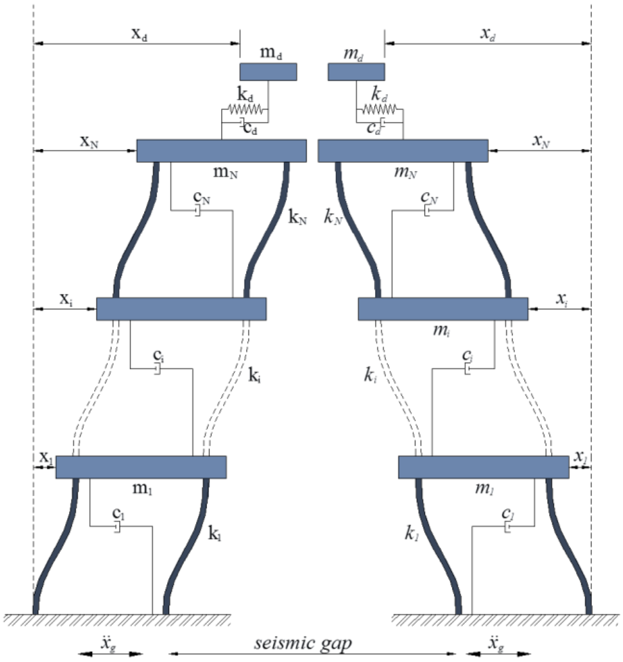

Fig. 1 Physical model of adjacent structures (design1)

$$\mathbf{x}= [\mathrm{x}_1\ \mathrm{x}_2\ \ldots\ \mathrm{x}_N\ \ \mathrm{x}_d\ \ x_N\ \ldots\ x_2\ \ x_1\ ]^T \tag{10}$$

In the optimization, the main Objective Function (OF) is to minimize the minimum required seismic gap given in Eq. (11). The minimum required seismic gap is also the maximum relative displacement of stories.

$$OF=\max(|[\mathrm{x}_1\ \mathrm{x}_2\ \ldots\ \mathrm{x}_N]^T-[x_1\ \ x_2\ \ldots\ x_N\ ]^T|) \tag{11}$$

4. Optimization process

A computer code is generated for the optimization and dynamic analyses of adjacent structures under several earthquake excitations. Matlab with Simulink (2010) was employed for time history analyses by choosing continuous state using Runge-Kutta method with 0.001 s time step. The methodology using HS for optimization can be explained in five steps.

i. In the first step, structural properties of adjacent structures, harmony search parameters, desired maximum seismic gap, earthquake records used in optimization process and solution range for possible TMD parameters is defined.

ii. Then, dynamic analyses of adjacent structures without TMD are done for future comparison of objective function when the stopping criteria of the optimization are decided.

iii. In this step, initial Harmony Memory (HM) matrix is constructed by harmony vectors containing randomly generated values for the TMD parameters. The optimized TMD parameters are the mass, stiffness and damping coefficients of TMD. This matrix contains harmony vectors as many as Harmony Memory Size (HMS).

iv. After initial HM matrix is generated, stopping criteria must be checked. The minimum required seismic gap (maximum difference of displacements of adjacent structures given in Eq. (11)) must be less than desired maximum seismic gap. The program has the ability to increase the desired maximum seismic gap if the criterion is not satisfied after several attempts. Also, another criterion about the frequency response of the structure is checked. The first story acceleration Transfer Function (TF) of the structures must be less than the value of uncontrolled structures. If the criteria are not satisfied, a new vector is generated.

v. A new vector is generated by using the special rules of HS in order to prevent to trap a local optimum value. Newly generated vector can be generated from the whole solution range or it can be generated from a smaller range around an existing harmony vector in HM matrix with a possibility called Harmony Memory Considering Rate (HMCR). The ratio of the smaller and whole solution range can be defined with the parameter called Pitch Adjusting Rate (PAR). The best and the worst vector is defined according to the objective function given in Eq. (11) and if the solution of newly generated vector is better than the worst vector stored in HM matrix, the worst one is replaced with the newly generated vector. The process is repeated from steps v until the stopping criteria are satisfied for all sets of harmony vectors.

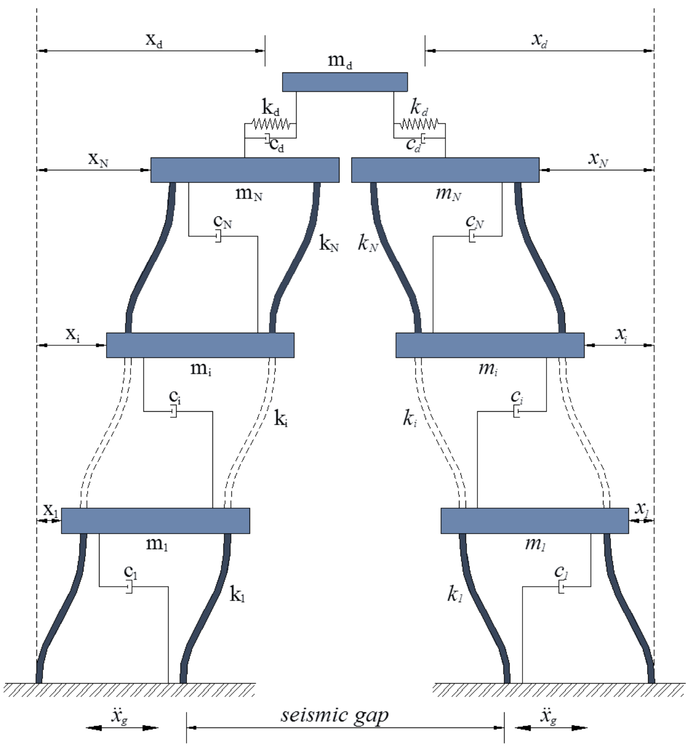

Fig. 2 Physical model of adjacent structures (design2)

Table 1 Earthquake records used in the HS optimization (Pacific Earthquake Engineering Research Center)

Earthquake	Date	Station	Component	PGA (g)	PGV (cm/s)	PGD (cm)
Cape Mendocino	1992	Petrolia	PET090	0.662	89.7	29.55
Kobe	1995	0 KJMA	KJM000	0.821	81.3	17.68
Erzincan	1992	95 Erzincan	ERZ-NS	0.515	83.9	27.35
Northridge	1994	Rinaldi	RRS228	0.838	166.1	28.78
Northridge	1994	24514 Sylmar	SYL360	0.843	129.6	32.68
Loma Prieta	1989	16 LGPC	LGP000	0.563	94.8	41.18

5. Numerical example

The proposed optimization technique is applied to Single Degree of Freedom (SDOF) structures and Multiple Degrees of Freedom (MDOF) adjacent structures. The limit of mass ratio, period and damping ratio of TMD are searched between 1%-5% and 0.8-1.2 times of period of the structure and 1%-20%, respectively. The harmony search parameters; HMS, HMCR and PAR are taken as 5, 0.5 and 0.1, respectively in order to reach the optimum results immediately without entrapped to a local optimum. The optimization is performed under six different earthquake records given in Table 1. The limits of solution range for design2 is configured as equal to the design1 for total mass ($\mathrm{m_d}+m_d$), stiffness coefficients ($\mathrm{k_d}$ and k_d) and damping coefficients ($\mathrm{c_d}$ and c_d) of TMDs.

5.1. Single degree of freedom (SDOF) adjacent structures

Adjacent structures were idealized to single degree of freedom structures in order to examine the success of the proposed optimization and protection technique for combination of adjacent structures with different period and damping ratio. Thus, two different designs for TMDs were compared and a conclusion is done according to the situation of adjacent structures.

Twelve combinations of adjacent structures given in Table 2 were investigated. T_1, T_2 and ξ_s are period of first and second structure and damping ratio of them, respectively. The first six combinations represent adjacent reinforced concrete structures with 5% inherent damping. The other combinations may represent base isolated adjacent structures as seen from damping ratios and periods.

In the analyses, the mass of the SDOF structures was taken as 100 t. The optimum parameters and the required seismic gap for SDOF adjacent structure combinations are shown in Table 3 for design 1 and in Table 4 for design 2.

For the combination of SDOF structures, the reduction percentages vary from 9.24% to 35.58% in design 1. According to the results, the approach is more effective than the other ones for the adjacent structures with close and long periods.

The best reduction of objective function is obtained for the ninth combination in the first design. The time history plots for the Relative Displacement of the structure (RD) which are the displacement of the first structure subtracted by the displacement of the other structure for all earthquake excitations, are shown in Fig. 3 for the combination 9 of the design 1. The optimum TMD is effective for all excitations including the most critical one (Loma Prieta) causing the maximum value of RD.

Table 2 Combination of the SDOF adjacent structures

Combination	T_1 (s)	T_2 (s)	ξ_s (%)
1	0.5	1	5
2	0.5	1.5	5
3	0.5	2	5
4	1	1.5	5
5	1	2	5
6	1.5	2	5
7	2	3	20
8	2	4	20
9	3	4	20
10	2	3	40
11	2	4	40
12	3	4	40

Table 3 Optimum TMD values for SDOF structures (design 1)

Combinations	m_d (t)	k_d (kN/m)	c_d (kNs/m)	*m_d (t)*	*k_d (kN/m)*	*c_d (kNs/m)*	OF without TMD	OF with TMD	Reduction (%)
1	4.83	110.973	0.160	4.94	14.430	0.188	48.84	44.33	9.24
2	2.59	40.351	0.074	4.92	12.890	0.082	62.07	59.19	4.64
3	3.50	41.622	0.102	4.58	3.869	0.069	64.09	57.10	10.91
4	4.95	18.658	0.065	4.83	7.888	0.231	89.13	65.41	26.62
5	4.99	21.589	0.104	4.70	3.867	0.050	93.03	82.97	10.82
6	5.00	9.437	0.062	4.90	4.452	0.105	81.85	55.20	32.55
7	4.82	5.291	0.054	2.86	0.871	0.036	47.63	43.06	9.59
8	4.96	4.132	0.108	4.99	1.159	0.020	76.17	68.35	10.27
9	4.70	2.753	0.025	4.87	0.968	0.037	52.66	33.92	35.58
10	4.91	5.675	0.059	3.79	1.196	0.014	27.49	24.35	11.44
11	4.99	4.876	0.040	4.97	0.958	0.027	44.86	39.75	11.40
12	4.96	3.334	0.037	4.65	0.967	0.025	21.89	16.25	25.75

Table 4 Optimum TMD values for SDOF structures (design 2)

Combinations	m_d (t)	k_d (kN/m)	c_d (kNs/m)	*k_d (kN/m)*	*c_d (kNs/m)*	OF without TMD	OF with TMD	Reduction (%)
1	4.16	67.909	0.513	4.330	0.020	48.84	11.00	77.48
2	4.36	79.099	1.493	1.307	0.009	62.07	10.84	82.54
3	4.31	79.897	0.847	0.787	0.020	64.09	10.53	83.56
4	9.93	17.265	0.084	11.098	0.368	89.13	38.78	56.49
5	9.73	14.644	0.092	7.435	0.046	93.03	38.12	59.03
6	9.86	11.085	0.071	7.214	0.435	81.85	51.59	36.97
7	9.88	4.801	0.058	3.332	0.200	47.63	35.71	25.03
8	9.71	4.717	0.037	1.793	0.251	76.17	35.30	53.66
9*	19.68	5.867	0.181	3.324	0.583	52.66	39.44	25.11
10*	18.56	7.691	0.234	5.990	0.822	27.49	26.07	5.17
11	7.27	1.949	0.021	1.745	0.339	44.86	28.24	37.04
12	-	-	-	-	-	-	-	-

*maximum mass ratio is 10%

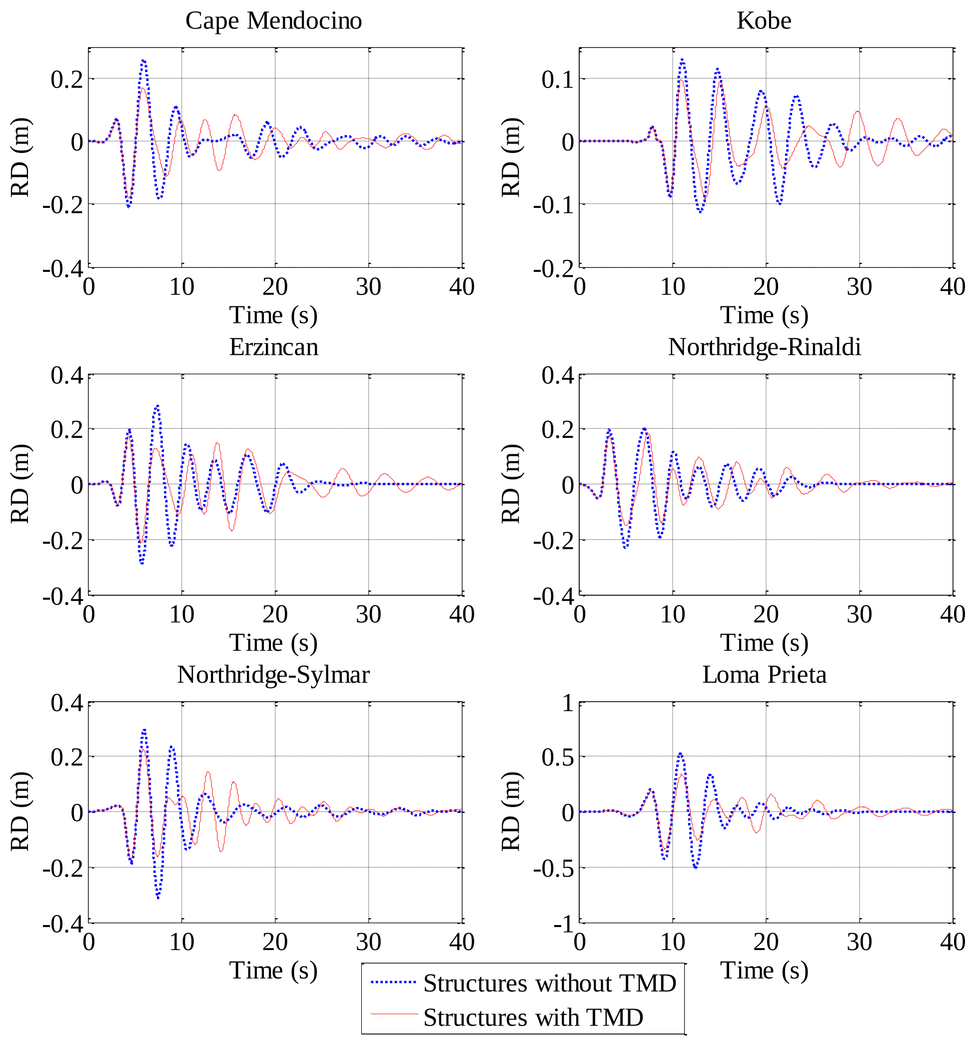

Fig. 3 Relative displacements for the combination 9 of the design 1

By using maximum 5% mass ratio, it is not possible to obtain a reduction on objective function for combinations 9-10 and 12 in design 2. For that reason, a reduction on objective function is provided with 10% mass ratio limits for combinations 9 and 10, respectively in design 2. A reduction cannot be provided for the combination 12 in design 2 even by using 10% mass ratio limit. Except the structures with long period and 40% damping, the reductions of objective function are generally more effective for design 2 than design 1.

For the second design, the reductions vary from 5.17% to 83.56%. The time history plots of RD are given in Fig. 4 for the combination 3 of the design 2 which is the best reduction of objective function is provided.

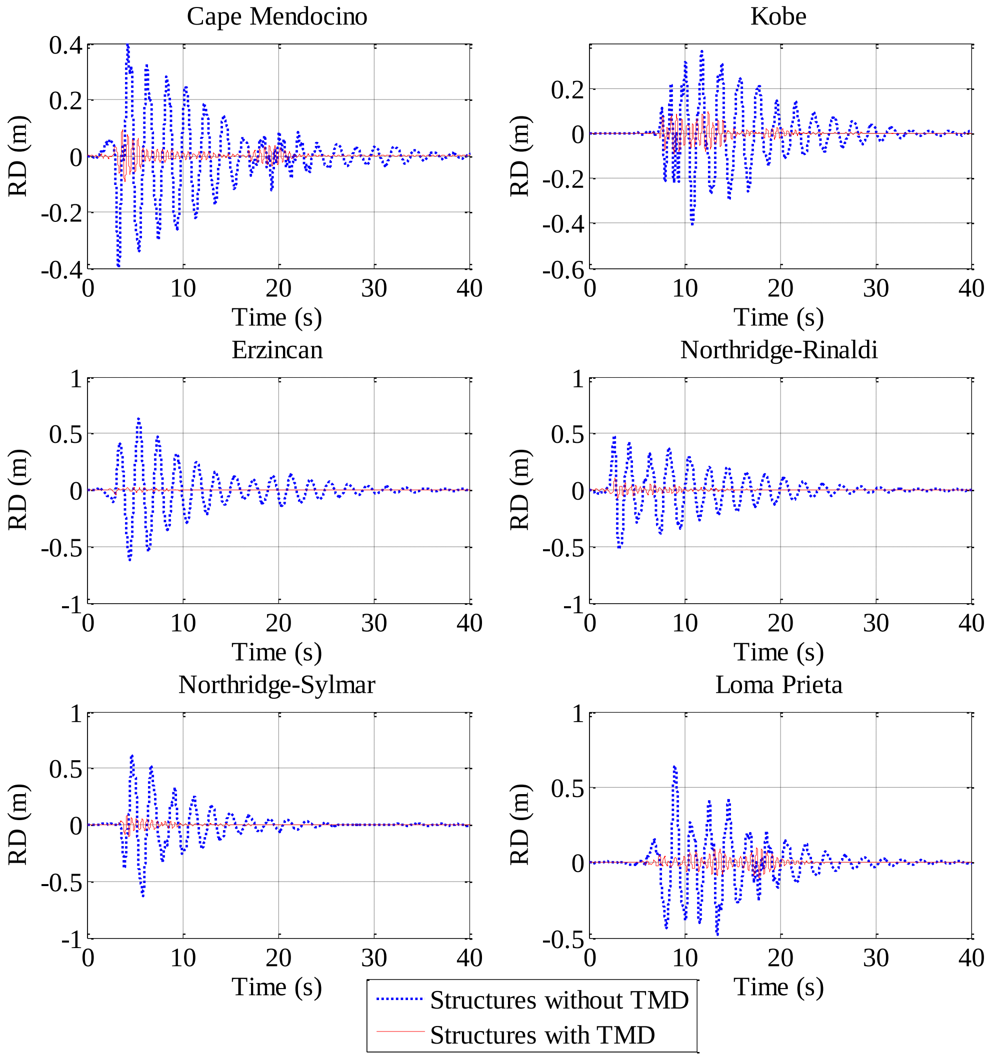

Fig. 4 Relative displacements for the combination 3 of the design 2

5.2. Multiple degrees of freedom (MDOF) adjacent structures

Two ten story adjacent structures were examined for optimum TMD design. Structures have different characteristics. First structure has 1 s critical period and the second one has nearly 2 s natural period. The first structure is significantly stiffer than the other structure. The pounding of these structures may cause great problems. The properties of these structures are given in Table 5. The first structure has same story properties while the mass and the stiffness of the other structures is decreasing by the increase of the stories.

Table 5 Properties of MDOF adjacent structures

Story	Structure 1			Structure 2		
	m_i (t)	k_i (kN/m)	c_i (kNs/m)	m_i (t)	k_i (kN/m)	c_i (kNs/m)
10	360	650000	6200	98	34310	442.599
9				107	37430	482.847
8				116	40550	523.095
7				125	43670	563.343
6				134	46790	603.591
5				143	49910	643.839
4				152	53020	683.958
3				161	56140	724.206
2				170	52260	674.154
1				179	62470	805.863

Table 6 Optimum TMD parameters

	Design 1		Design 2	
	Case 1	Case 2	Case 1	Case 2
$\mathrm{m_d}$ (t)	178.42	354.94	193.20	326.10
$\mathrm{k_d}$ (kN/m)	8177.08	16630.54	10034.35	12161.37
$\mathrm{c_d}$ (kNs/m)	44.44	249.91	46.00	662.46
m_d (t)	65.55	135.40	-	-
k_d (kN/m)	699.25	1067.66	133.44	120.19
c_d (kNs/m)	9.18	107.15	12.90	7.04

Optimization of TMD was done for two cases of mass ratio range. In the second case, the maximum mass ratio limit was taken as 10% while all other ranges were taken as the same as single degree of freedom example in case 1. Table 6 shows the optimum TMD parameters.

In Fig. 5, maximum Relative Displacements (RD) for stories are given for design 1. The optimization objective is the highest one of these displacements which occurs at the top story and under Northridge-Rinaldi excitation. The plots given in Fig. 5 prove that the optimum TMD is not only effective for the critical excitation and story. The case with lower mass ratio than other one (Case 1) is not effective for Northridge-Sylmar excitation. The optimum TMD has the minimum effect on the results of Loma Prieta excitation. For Cape Mendocino excitation, the same performance is nearly obtained according to the results of the both cases.

The time history plots of RD at the tenth (top) story of the structure under Northridge – Rinaldi excitation are given in Fig. 6 for Case 1 and 2 of design 1. The second case is absolutely more effective than other case in damping of vibrations.

The maximum values of displacement and acceleration of structures for design 1 are given in Table 7 and 8, respectively. According to the results, optimum TMDs for both cases are effective to reduce maximum responses for the critical excitation which is Rinaldi record of Northridge earthquake.

Fig. 7 shows the relative displacements for the design 2. Differently from the first design, optimum TMD is not so effective on reduction of optimization objective. Also, increases on relative displacements are observed for several stories and excitations such as Kobe, Erzincan, Northridge – Sylmar and Loma Prieta. Whereas, design 2 is very effective for SDOF structures with 1 s and 2 s period (combination 5). This shows that the second design is only feasible for rigid structures with a single degree of freedom. Additionally, several critical responses may increase (as seen in Table 9 and 10) for the slender structure in design 2. For that reason, the design of structure must be suitable to increase of the responses in the design combining both structures. The only benefit of the second design is the reduction of seismic gap.

Table 7 Maximum displacement responses for design 1

Earthquakes	Structure 1			Structure 2		
	Without TMD	Case 1	Case 2	Without TMD	Case 1	Case 2
Cape Mendocino	0.34	0.29	0.26	0.64	0.58	0.51
Kobe	0.52	0.39	0.36	0.57	0.49	0.45
Erzincan	0.28	0.23	0.24	0.98	0.76	0.67
Northridge (Rinaldi)	0.65	0.50	0.50	0.94	0.82	0.74
Northridge (Sylmar)	0.27	0.27	0.30	0.96	1.01	0.72
Loma Prieta	0.34	0.36	0.33	1.01	0.84	0.78

Table 8 Maximum acceleration responses for design 1

Earthquakes	Structure 1			Structure 2		
	Without TMD	Case 1	Case 2	Without TMD	Case 1	Case 2
Cape Mendocino	15.44	13.05	11.12	12.04	10.67	9.42
Kobe	23.63	17.44	14.38	13.76	13.60	12.75
Erzincan	11.75	9.07	8.26	10.25	8.91	8.33
Northridge (Rinaldi)	27.73	19.88	19.26	18.96	17.94	16.68
Northridge (Sylmar)	14.81	13.50	13.22	11.98	12.52	10.27
Loma Prieta	13.58	15.48	15.08	14.90	15.79	13.51

Table 9 Maximum displacement responses for design 2

Earthquakes	Structure 1			Structure 2		
	Without TMD	Case 1	Case 2	Without TMD	Case 1	Case 2
Cape Mendocino	0.34	0.29	0.27	0.64	0.65	0.68
Kobe	0.52	0.38	0.36	0.57	0.89	0.94
Erzincan	0.28	0.25	0.23	0.98	1.05	1.11
Northridge (Rinaldi)	0.65	0.50	0.50	0.94	1.21	1.22
Northridge (Sylmar)	0.27	0.28	0.29	0.96	1.13	1.18
Loma Prieta	0.34	0.39	0.31	1.01	1.15	1.21

Table 10 Maximum acceleration responses for design 2

Earthquakes	Structure 1			Structure 2		
	Without TMD	Case 1	Case 2	Without TMD	Case 1	Case 2
Cape Mendocino	15.44	12.65	12.06	12.04	18.51	19.17
Kobe	23.63	16.29	16.10	13.76	24.57	25.81
Erzincan	11.75	8.94	8.72	10.25	13.50	13.78
Northridge (Rinaldi)	27.73	19.98	17.25	18.96	29.06	28.81
Northridge (Sylmar)	14.81	13.67	13.31	11.98	19.60	19.81
Loma Prieta	13.58	15.64	11.21	14.90	26.94	27.90

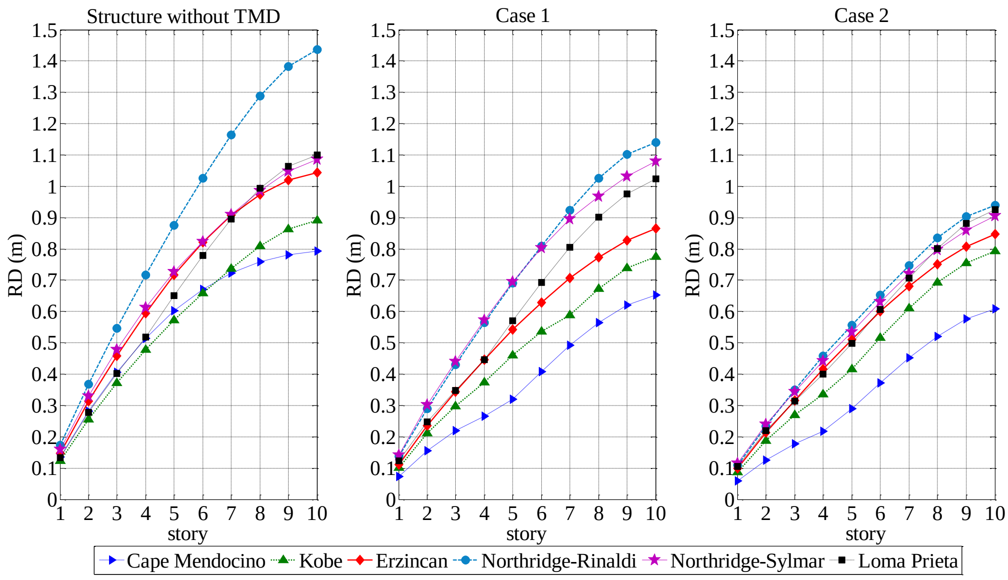

Fig. 5 Maximum relative displacements for design 1

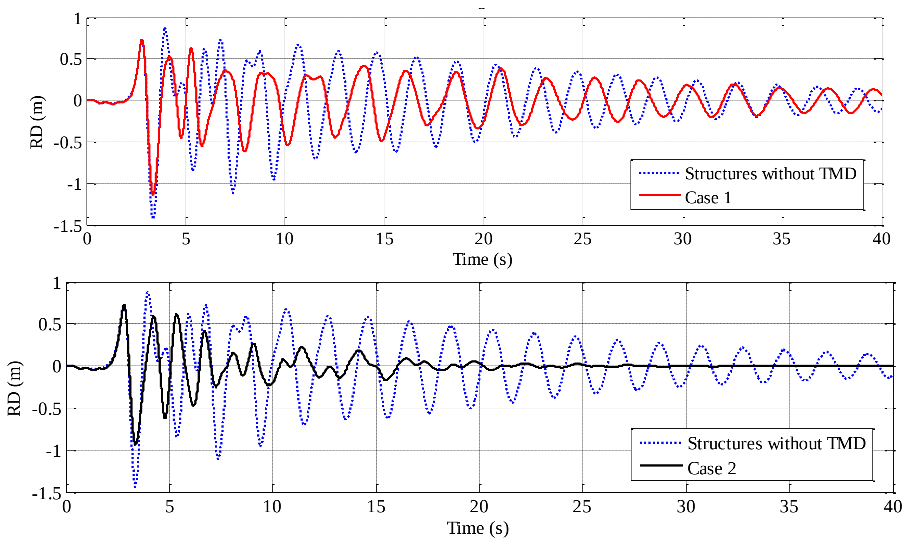

Fig. 6 RD of the tenth story under Northridge-Rinaldi excitation (design 1)

For the first design, first story acceleration Transfer Function (TF) plots are given in Fig. 8 and Fig. 9 for case 1 and 2, respectively. The optimum TMDs are effective on reduction of peak values occurred at natural frequencies of adjacent structures. The results of case 2 show the effectiveness of using a heavier TMD than the first case.

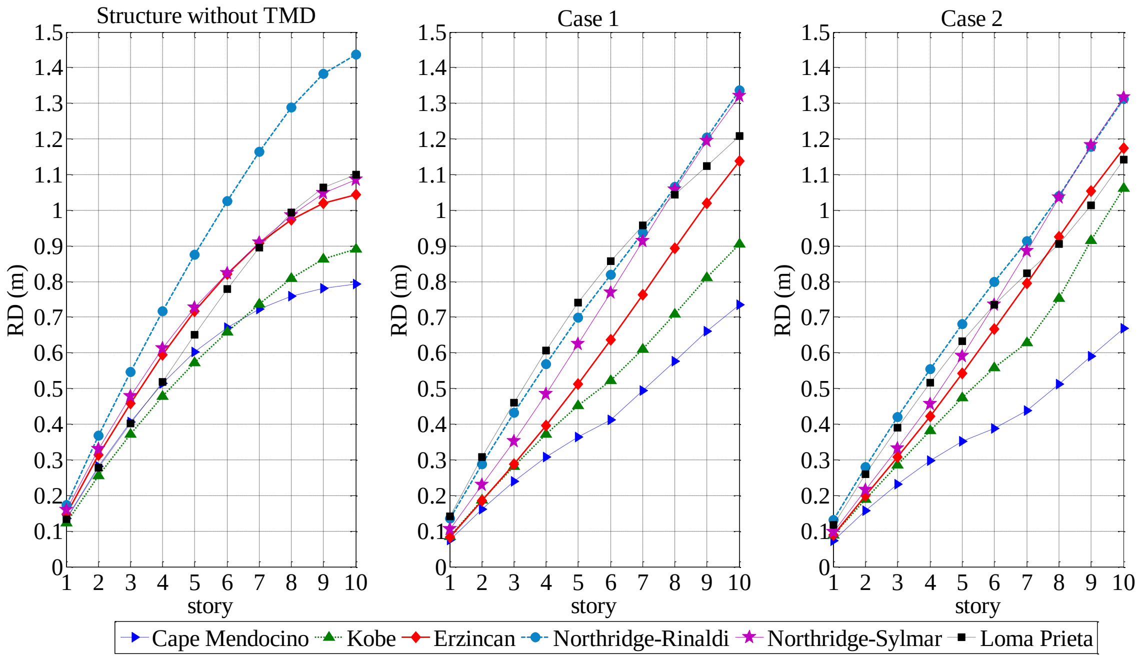

Fig. 7 Maximum relative displacement for design 2

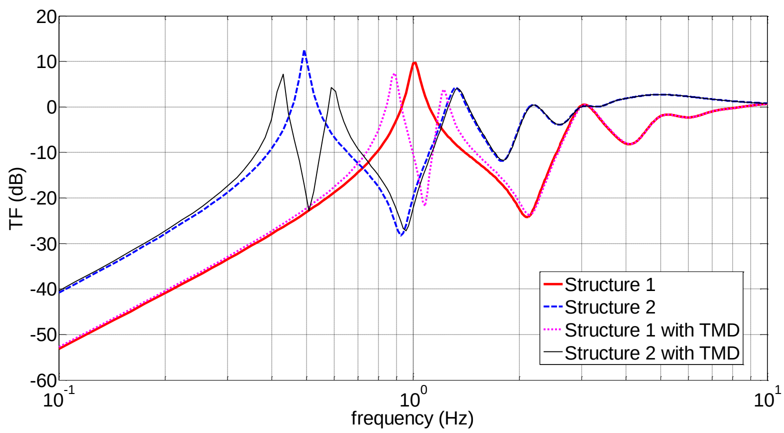

Fig. 8 Acceleration transfer function plot for design 1 (case 1)

The optimum TMD is also effective on the maximum TF for the design 2 as seen in Fig. 10 and Fig. 11. Due to TMD, first resonance peak of structure 2 is not critical for the coupled structures. For the coupled structures, the first natural frequency which is near to the first and critical frequency of the structure 2, is not critical as seen in the transfer function plots. The critical frequency is close to the first frequency of the first structure which is more rigid than the other one.

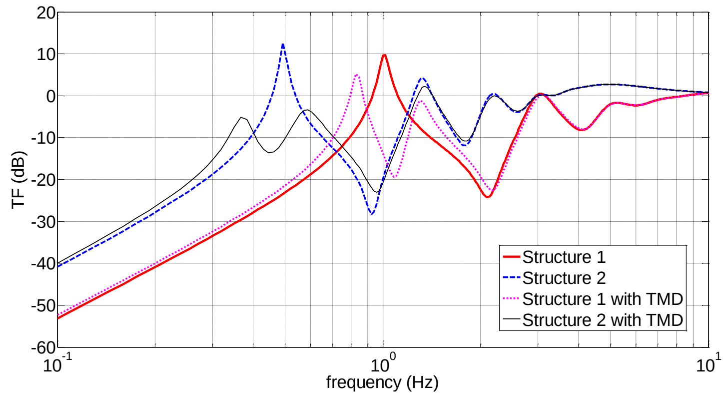

Fig. 9 Acceleration transfer function plot for design 1 (case 2)

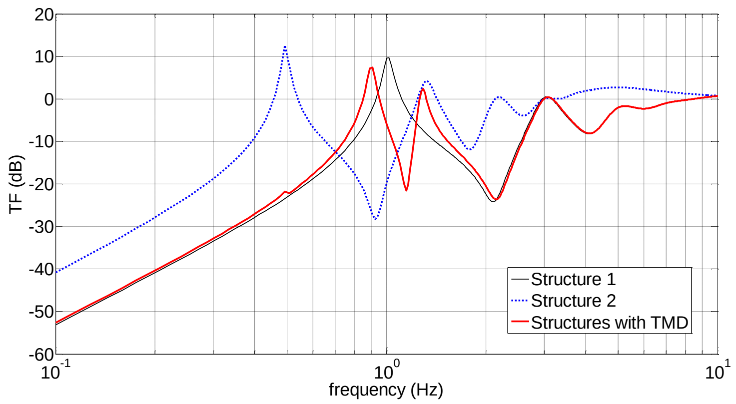

Fig. 10 Acceleration transfer function plot for design 2 (case 1)

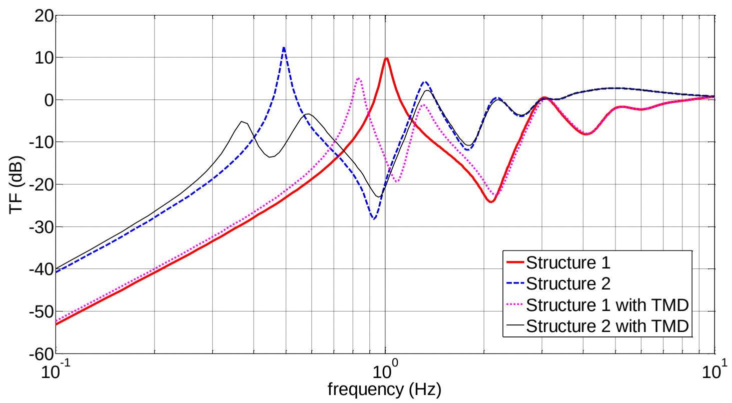

Fig. 11 Acceleration transfer function plot for design 2 (case 2)

6. Conclusions

The use of TMDs for adjacent structures seems as an effective method on preventing of pounding. In tuning of TMD parameters, a methodology different than optimizing TMDs for single structures must be considered because in adjacent structures, the relative displacements of structures with respect to each other is more important than the displacement of structures with respect to ground. The generated algorithm considering the dynamic response of both structures is effective on finding the optimum TMD parameters for two different designs. In design 1 and 2, the adjacent structures are not coupled and coupled, respectively.

First, the methodology was applied for single degree of freedom adjacent structures with different periods and damping ratios. According to the results of design 1 for SDOF structures, best reductions are obtained for the structures with long and close periods. In design 2, the maximum mass ratio limit used in the study is not sufficient for several combinations of the structure. By increasing the mass ratio limit, a possible solution for optimum TMD were found for several combinations, but a physical solution cannot be obtained for the structures with long period (3s and 4s) and high damping (40%).

According to the results, coupling of SDOF adjacent structures with a TMD is generally more effective than using independent TMDs on the structures. But, coupling of flexural multiple degree of freedom adjacent structures is not a good idea as seen in numerical example of two adjacent 10-story structures. Additionally, other structural responses may be negatively affected in design 2 while optimum TMD used separately on the both structures are very effective on the other responses such as displacement and acceleration.

As a conclusion, it is possible to reduce the value of minimum seismic gap and to prevent the pounding of adjacent multi-story structures by using optimally tuned TMDs on both structures. A TMD coupling both structures is an optimum choice for only rigid structures with a single degree of freedom like rigid adjacent bridges. For adjacent single degree of freedom structures with high damping, coupling of adjacent structures with a TMD is also not effective.

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