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EFFECTIVENESS OF DISTRIBUTED TUNED MASS DAMPERS IN SEISMIC RESPONSE CONTROL OF BRIDGE WITH SOIL-STRUCTURE INTERACTION

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ABSTRACT: The effect of soil-structure interaction (SSI) on the peak responses of three span continuous deck bridge seismically controlled by distributed tuned mass dampers (d-MTMDs) is investigated. A concrete type of bridge is idealized using finite element model, controlled by installing tuned mass damper(s) (TMD/s). The TMDs are placed where the mode shape amplitude of the bridge is the largest or larger in the particular mode and each tuned with the corresponding modal frequency, while controlling up to first few modes as desirable. The importance is placed on gauging the significance of physical parameters that affect the response of the system and identify the circumstances under which it is necessary to include the SSI effects in the design of seismically controlled bridges. However, the soil surrounding the foundation of pier is modeled by frequency independent coefficients and the complete dynamic analysis is carried out in time domain using direct integration method. It is observed that the soil surrounding the pier has significant effects on the response of the controlled bridges. In addition, it is also seen that the seismic responses of bridge with d-MTMDs are seen to be considerably altered. Further, it is concluded that the installation of the d-MTMDs in bridges in accordance with their modal properties reduce maximum pier base shear and deck displacement effectively.

1. Introduction

Bridges are the most important lifeline structures and their failure as a result of seismic event seriously obstructs relief and rehabilitation work. They are the most important link for surface transportation networks. There are number of bridges which are collapsed due to the past earthquakes all over the world. Bridges are especially vulnerable to damage and can easily collapse as a result of earthquake ground motions. However, the reasons of failure are their structural simplicity (lesser redundancy) and the fundamental time period. The fundamental time period of vibration of most of the bridges is found to be in the range of 0.2 to 1 sec. From the past earthquakes, it is noted that the predominant time periods had been in this range, thereby it has caused the seismic response of bridges to amplify. Earlier, the effectiveness of tuned mass dampers (TMDs) for vibration control of long span bridges and tall buildings due to wind and earthquake excitations were extensively studied. Optimal linear vibration absorber for linear damped primary system was determined by Randall et al. (1981) using graphical solution. It was also reported that small offset in tuning of the frequency could result in decreased efficiency of a single TMD. Use of multiple tuned mass dampers (MTMDs) was also studied earlier amply, showing that MTMDs are more effective than single TMD (STMD). Luu et al. (2012) have shown the effectiveness of MTMDs to control the vibration of bridges caused due to trains moving at high speed. Recently, Matin et al. (2014) reported that d-MTMDs are the best solution to control the bi-direction response of the concrete bridges subjected to earthquake ground excitations. However, the above mentioned studies ignored the effect of soil-structure interaction (SSI). Tongaonkar and Jangid (2002) reported that the soil surrounding the pier has significant effects on the response of the isolated bridges and under certain circumstances the bearing displacements at abutment locations may be underestimated if the SSI effects are not considered in the response analysis of the system. Their investigation shows that consideration of SSI in the analysis will result in enhancement of safety and reduction in design costs. However, no study is seen by the authors wherein placement and tuning of the TMDs has been done in accordance with their modal properties with considering of SSI. Hence, objectives of this study include (i) the effective placement of distributed multiple tuned mass dampers (d-MTMDs) and (ii) tuning of the d-MTMDs to higher modal frequencies for seismic response mitigation of a concrete bridge with considering of SSI.

2. Structural Model of Bridge without/with TMDs

In this study a concrete continuous span bridge is considered. The assumptions for this study are made as following

1. Uncontrolled bridge and controlled with TMDs systems are assumed to remain in elastic range.
2. The structures with/without TMDs are modeled as a finite element model divided into a number of small discrete segments and a node will connect two adjacent segments together. Degrees of freedom at each node considered to be two and masses of each segment assumed to be d-between two adjacent nodes.
3. Mass contribution of non-structural elements such as parapet walls, kerbs and wearing coat is considered because they are producing inertial forces, however their stiffness is neglected.
4. Because the horizontal and vertical components of an earthquake are generally uncorrelated. Thus, the bridge with/without TMDs is subjected to two horizontal components of ground motion and the effect of vertical component is not considered.
5. At least ninety nine percent of total mass included in the controlled modes.
6. The soil supporting the pier foundation is modeled as spring and damper acting in the horizontal and rotational directions. Viscous damping is used to simulate the radiation damping in the soil, which is developed through the loss of energy emanating from the foundation in the semi-infinite soil medium (Tongaonkar and Jangid, 2002).
7. The foundation is represented for all motions using a spring–dashpot-mass model with frequency-independent coefficients. The modeling of the foundation on deformable soil is performed in the same way as that of the structure and is coupled to perform a dynamic SSI analysis (Tongaonkar and Jangid, 2002).

Table 1- Dynamic properties of soil

Bulk modulus, G (MPa)	3.57
Shear wave velocity, C_s (m/s)	394
Horizontal stiffness of soil medium, K_h ($10^8 N/m$)	4.29
Rocking stiffness of soil medium, K_r ($10^8 N/m$)	1.8
Horizontal damping coefficient, C_h ($10^7 N\ s/m$)	1.04
Rocking damping coefficient, C_r ($10^7 N\ s/m$)	0.967

Fig. 1(c) shows a finite element model of the concrete bridge installed with TMDs, duly considering the flexibilities of both, the bridge deck and piers. The bridge bearings are supported on reinforced concrete piers and rigid abutments. In Fig.1, $k_{d\text{-}x}$, $k_{d\text{-}y}$, $c_{d\text{-}x}$ and $c_{d\text{-}y}$ are the stiffness and damping of a TMD in longitudinal (E-W) and transverse (N-S) directions. The mass of TMD is m_i which are installed at locations determined to be most optimal.

Table 2- Section and material properties of concrete bridge

Member Properties	Three Span Continuous Bridge	
	Deck	Piers
Area (m^2)	3.57	4.09
Moment of inertia in N-S direction (m^4)	2.08	0.64
Moment of inertia in E-W direction (m^4)	2.08	0.64
Young's modulus of elasticity (KN/m^2)	2.02×10^7	2.02×10^7
Mass per unit volume (KN/m^3)	23.536	23.536
Length/height (m)	3@30 = 90	8
Fundamental time period in N-S direction (sec)	0.45	0.1
Fundamental time period in E-W direction (sec)	0.45	0.1
Damping ratio	5%	5%
Shape	Rectangular	Circular

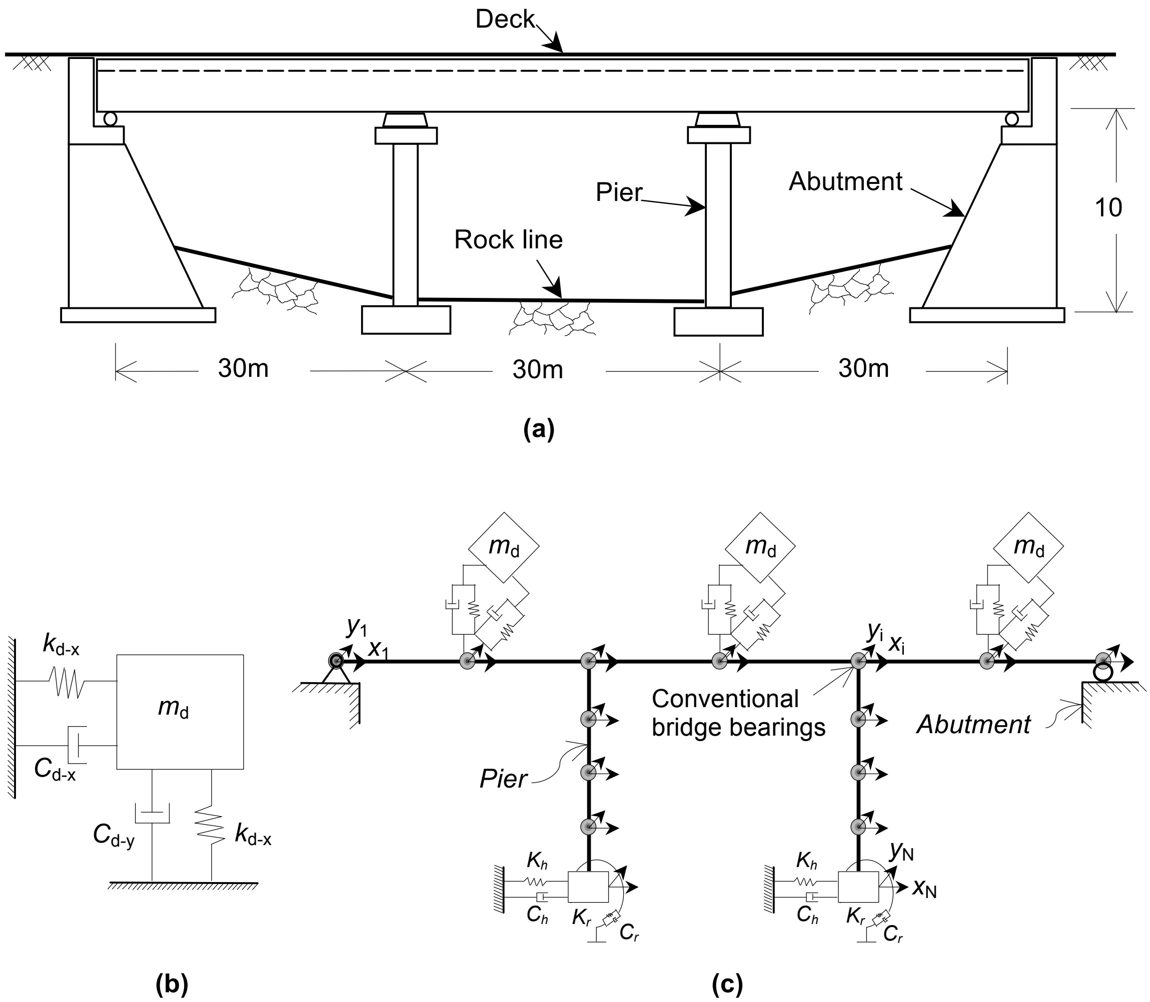

Fig. 1 – (a) General elevation of three span continuous concrete bridge, (b) Schematics of TMD, (c) Finite element model of three span continuous bridge with SSI.

The system has additional degrees-of-freedom at the base of pier due to flexibility of foundation or SSI effects (refer degrees-of-freedom x_n and y_n in Fig. 1(c)). The above assumptions will lead to the mathematical model of the bridge as shown in Fig. 1(c).

A sufficiently accurate consideration of soil behavior can be obtained if the soil stiffness and damping coefficients of a circular mass less foundation on soil strata are evaluated by the frequency independent expressions, Spyrakos (1990). The stiffness and damping coefficients of soil medium are expressed by

$$K_h = \frac{8Ga}{2-\gamma}\left(1+\frac{1}{2\bar{H}}\right) \quad \text{for } \bar{H} > 1 \tag{1}$$

$$K_r = \frac{8Ga^3}{3(2-\gamma)}\left(1+\frac{1}{6\bar{H}}\right) \text{for} 1 < \bar{H} \le 4 \tag{2}$$

$$C_r = \frac{0.4Ga^2}{(1-\gamma)C_s} \tag{3}$$

$$C_h = \frac{4.6Ga^2}{(2-\gamma)C_s} \tag{4}$$

Where K_h and K_r represent the horizontal and rocking stiffness of soil medium, respectively; C_h and $C_r$are the horizontal and rocking viscous damping coefficients for radiation soil damping, respectively; G is the soil shear modulus; C_s is the shear wave velocity for soil; a is the radius of circular footing, g is Poisson's ratio for the soil and H is the depth of the soil stratum overlying a rigid bedrock, and $\bar{H} = H / a$ The above expressions are also valid for the limiting case of a large soil stratum, in which the term $\bar{H}$ diminishes, (N.P. Tongaonkar and R.S. Jangid, 2002).

The equations of motion of the bridge installed with TMDs, under two horizontal components of an earthquake ground motion expressed in matrix are,

$$[M]\{\ddot{Q}\} + [C]\{\dot{Q}\} + [K]\{Q\} = -[M]\{r\}\{\ddot{Q}_{\text{g}}\} \tag{5}$$

Where $[M]$, $[C]$ and $[K]$ are the mass, damping and stiffness matrices, respectively, of the bridge of order $(2N+2n)\times(2N+2n)$, with N indicating degrees of freedom for the bridge and n indicating degrees of freedom corresponding to the TMDs; $\{Q\} = \{X_1, X_2, ...X_{\text{N}}, ...x_{\text{n}}, Y_1, Y_2, ...Y_{\text{N}}, ...y_{\text{n}}\}^{\text{T}}$ $\{\dot{Q}\}$ and $\{\ddot{Q}\}$ are the displacement, velocity and acceleration vectors, respectively; $\{\ddot{Q}_{\text{g}}\}$ is the earthquake ground acceleration vector, including $\ddot{x}_{\text{g}}$ and $\ddot{y}_{\text{g}}$ as earthquake ground acceleration in E-Wand N-S direction, respectively; and $\{r\}$ is the vector of influence coefficients. Further, $\{X_{\text{i}}\}$ and $\{Y_{\text{i}}\}$ are the displacement of the i^{th} node of the bridge in N-S and E-W directions, respectively. The modal frequencies and mode shapes of the bridge are determined by conducting free vibration analysis, solving Eigen value problem for tuning the dampers and deciding their placement. The TMDs are placed where the mode shape amplitude of the bridge is the largest/larger in the particular mode and each tuned to the corresponding modal frequency, while controlling up to first few modes. Not more than one TMD is placed at a location and are the stiffness and damping parameters of the TMDs in N-S and E-W directions, $k_{\text{d-x}}$, $k_{\text{d-y}}$, $c_{\text{d-x}}$ and $c_{\text{d-y}}$ are calculated based on the modal frequencies. The equations of motion in Equation (5) are solved using the modified modal superposition method for obtaining time history of seismic response. The bi-directional interaction of the seismic response has been duly considered under two horizontal components of the earthquakes, applied simultaneously.

2.1. Modeling of STMD/TMDs/d-MTMDs

Earthquake excitation often dominate multimode of the bridges. Due to this effect the bridge can be damaged and it is unsafe for riding. The performance of the d-MTMDs can be improved by effective placement, tuning and optimal parameters. Fig. 2 shows the first four-mode shape of the bridges with/without TMDs in E-W direction. The TMDs are placed where the amplitude of the mode shapes are

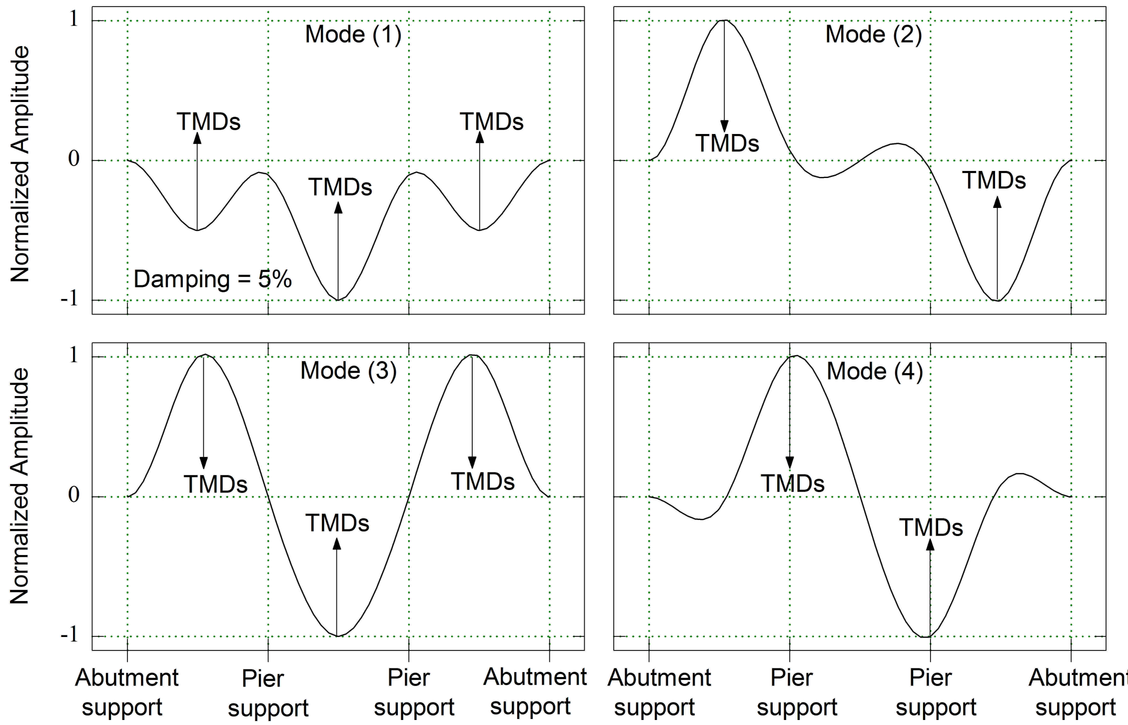

Fig. 2–Mode shape of three span continuous bridge and placement of TMDs, (Chang in mode shape duo to different placement of TMDs).

maximum and each tuned to the corresponding modal frequency. The first four modal frequencies are controlled in both directions simultaneously and frequency of each TMD is calculated from Equation 6.

$$f_1 = \frac{\omega_1}{\Omega_1}, f_2 = \frac{\omega_2}{\Omega_2}, f_3 = \frac{\omega_3}{\Omega_3}, f_4 = \frac{\omega_4}{\Omega_4} \tag{6}$$

Where all tuning frequencies ratios are, $f_1 = f_2 = f_3 = f_4 = 1$. ω_1 to ω_4 and Ω_1 to Ω_4 are the frequencies of the TMDs and first four natural frequencies of the bridge respectively. For design of the d-MTMDs/MTMDs devices, we consider a set of TMD units with equal stiffness, $k_1 = k_2 = k_3 = \ldots = k_n$, rather than identical masses. The stiffness (k_i) of TMDs can be calculated as,

$$k_i = \frac{m_n}{\left(\frac{1}{\omega_1^2} + \frac{1}{\omega_2^2} + \cdots \frac{1}{\omega_n^2}\right)} \qquad i = 1 \text{ to } 4 \tag{7}$$

Here, m_n is calculated for a particular mass ratio, μ. The masses are used for adjusting the frequency of each TMD unit such that,

$$m_i = \frac{k_i}{\omega_i^2} \qquad i = 1 \text{ to } 3 \tag{8}$$

Fig. 3 shows the first four mode shape of the bridge with/without TMDs in transverse direction. The TMDs are placed where the amplitude of the mode shapes are maximum and each tuned to the corresponding modal frequency.

2.2. Solution of Equations of Motion

The classical modal superposition technique cannot be employed in the solution of equations of motion here because (i) the system is non-classically damped because of the difference in the damping in TMDs system compared to the damping in the superstructure. Therefore, the equations of motion are solved by modified modal superposition method. The time interval for solving the equations of motion is taken as 0.02 sec.

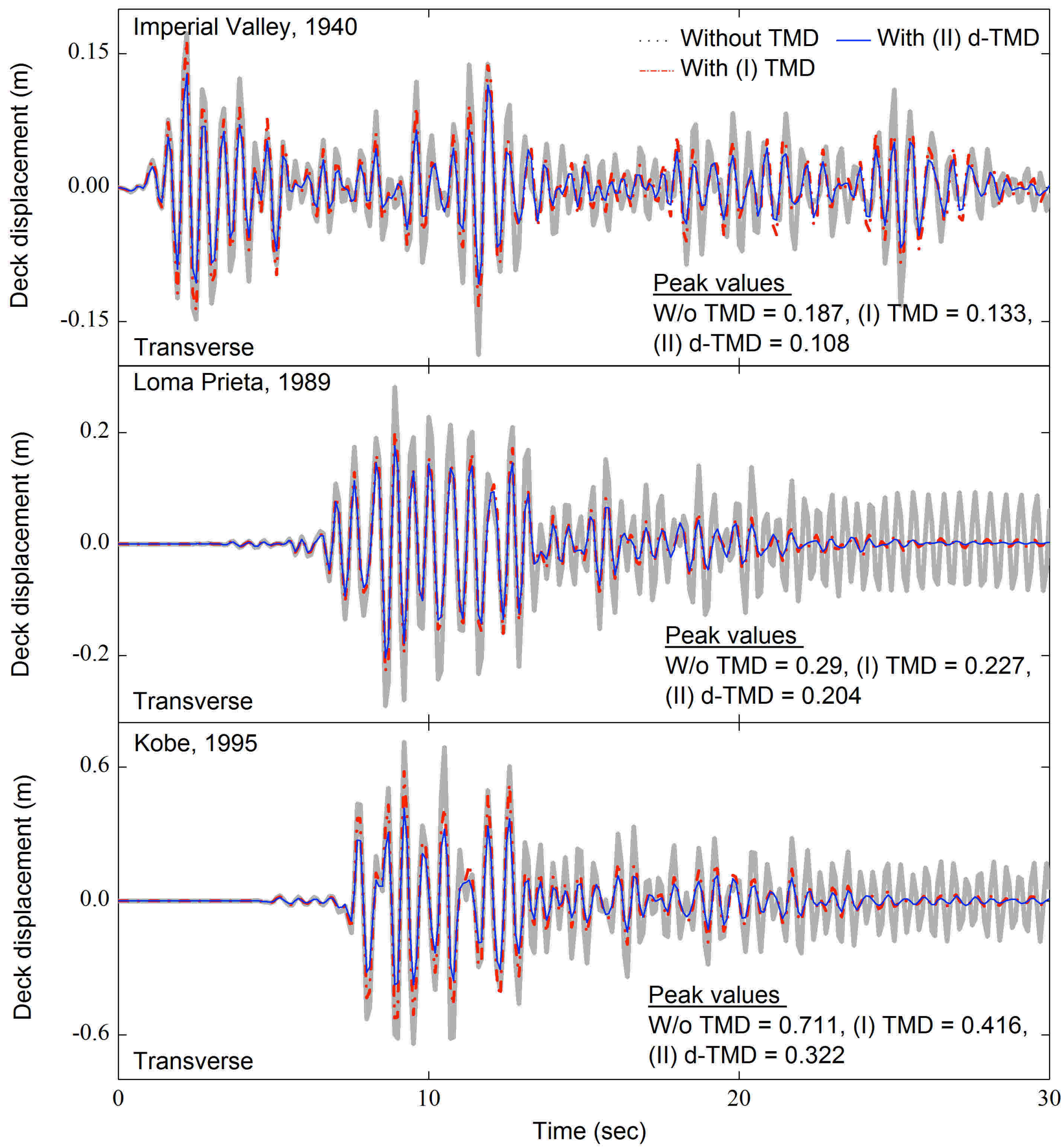

Fig. 3– Time variation for deck displacement of concrete bridge without TMD and with d-MTMDs under three different earthquakes ground motion in N-S direction.

3. Numerical Study

Seismic response of a three span continuous deck bridge with reinforced concrete piers and a pre-stressed concrete box girder is investigated under various real earthquake ground motions. The following earthquake ground motions: Imperial Valley in 1940 (recorded at El- Centro station), Kobe in 1995 (recorded at JMA) and 1989 Loma Prieta earthquake (recorded at Los Gatos Presentation Center). The peak ground acceleration (PGA) of Imperial Valley, Kobe and Loma Prieta earthquake motions are 0.348g, 0.86g and 0.57g respectively. The material properties and dynamic properties of this bridge are given in Table 1. The properties of this bridge are taken from the bridge studied by Wang et al. (1998) using the sliding isolation system, then by Kunde and Jangid (2006). The Fundamental periods of the bridge in both directions are 0.45 sec. the other frequencies of the structure in both directions longitudinal and transverse respectively are, (0.426, 0.19 and 0.1 sec) and (0.43, 0.1626 and 0.1617 sec). The dynamic soil properties of the bridge are given in Table 2. The TMDs designed as coupled to control the frequencies for both directions (longitudinal and transverse) may have two springs and dash-pots and single mass attached to them.

Table 3 - Uncontrolled responses of the bridge under different earthquakes without/with SSI.

Earthquakes	SSI	Imperial Valley, 1940		Loma Prieta, 1989		Kobe, 1995	
Directions		N-S	E-W	N-S	E-W	N-S	E-W
Maximum Base Shear/Deck Weight	Without	1.95	8.01	4.39	6.02	6.92	5.96
	With	1.04	1.18	1.67	3.99	2.45	4.46
Maximum Deck Displacement (m)	Without	0.12	0.14	0.25	0.14	0.45	0.14
	With	0.19	0.20	0.29	0.26	0.71	0.29
Maximum Deck Acceleration (g)	Without	0.25	0.36	0.42	0.32	0.89	0.28
	With	0.14	0.07	0.16	0.26	0.31	0.20

To show the performance of d-MTMDs in response control of bridges three reduction criteria are reduction of pier base shear (J_1, %), reduction of deck displacement(J_2, %) and reduction of deck acceleration(J_3, %). These are defined with the help following Equations.

$$J_1 = \left\{1 - \frac{F_\mathrm{p}}{F_\mathrm{po}},\right\} \times 100 \tag{9}$$

$$J_2 = \left\{1 - \frac{x_\mathrm{d}}{x_\mathrm{do}}\right\} \times 100 \tag{10}$$

$$J_3 = \left\{1 - \frac{\ddot{x}_\mathrm{d}}{\ddot{x}_\mathrm{do}},\right\} \times 100 \tag{11}$$

Where F_p, x_d and $\ddot{x}_\mathrm{d}$ are the peak pier base shear, peak displacement and peak acceleration of deck with control, while, F_p, x_do and $\ddot{x}_\mathrm{do}$ are the peak pier base shear, peak displacement and peak acceleration of deck without control respectively.

3.1. Effectiveness of SSI

It is a common practice that the bridge piers are assumed to be rigid (fixed in solid rock) and there has not been any attempt to investigate the effects of SSI on the response of bridges installed with TMDs. Table 3 shows the peak response of the bridge without/with SSI. It is noted that the acceleration and base shear are reduced in bridge with SSI under different real earthquake ground excitations. However, the displacement at the center of the deck increased significantly. Therefore, the designers have to be careful while designing bridges. Thus, the authors have implied TMDs to control the undesirable responses under different real earthquake ground motions. Time variation of deck displacement of the bridge installed with d-MTMDs is shown in Fig. 3 respectively. The devices designed with same damping ratio of 5% as of the main structure. The mass ratios for the cases were remained same for comparisons and the value is 1% of the total mass of the bridge. The Figure is showing the responses of bridge controlled with TMDs and MTMDs with the configuration of having equal masses. It is observed the significant reduction in deck displacement after installing dampers.

3.2. Effectiveness of STMD and Distributed MTMDs Systems

Optimal placement of TMDs can help the system to perform better during earthquake motion. Base shear in the piers, displacement and acceleration at the center of the bridge deck are the response quantities of interest in the bridge system under consideration. The pier base shears, displacement and acceleration of the deck are directly proportional to the forces exerted in the bridge system due to earthquake ground motion. The suitable places for TMDs are chosen as per maximum amplitude of mode shapes and each tuned to the corresponding modal frequency. The reduction in peak values of the responses for the cases, which were mentioned previously, for the concrete bridge with distributed MTMDs, are presented in Table 4. It is observed that d-MTMDs are effective and responses are considerably reduced as compared to that of uncontrolled bridge and STMD. It implies that all the TMDs systems are effective in response reduction of the bridges due to earthquake forces. In addition, it is observed that MTMDs systems are much effective for bi-direction earthquake excitations as compared to STMD.

Table 4 -Percentage reductions of the peak responses of the concrete bridge installed with STMD, MTMDs and d-MTMDs.

Earthquake		Percentages reduction response *J*(%)					
		Imperial Valley (1940)		Loma Prieta (1989)		Kobe (1995)	
Direction		N-S	E-W	N-S	E-W	N-S	E-W
$J_1(\%)$	NC	0.00	0.00	0.00	0.00	0.00	0.00
	(I) TMD	22.73	18.71	20.90	14.59	35.53	9.84
	(II) TMD	30.43	22.94	23.90	15.67	38.55	11.18
	(II) d-TMD	33.70	22.94	26.29	15.67	38.71	11.18
	(III) TMD	33.61	28.02	27.85	16.37	38.96	22.40
$J_2(\%)$	NC	0.00	0.00	0.00	0.00	0.00	0.00
	(I) TMD	29.01	13.00	21.61	5.47	41.54	23.23
	(II) TMD	31.59	20.50	23.83	8.88	46.68	26.66
	(II) d-TMD	42.48	20.50	29.67	8.88	54.71	26.66
	(III) TMD	45.60	27.50	28.71	9.13	52.11	31.11
$J_3(\%)$	NC	0.00	0.00	0.00	0.00	0.00	0.00
	(I) TMD	21.43	21.34	31.25	17.42	34.10	12.94
	(II) TMD	28.79	31.68	37.50	23.86	37.70	16.42
	(II) d-TMD	36.07	31.68	45.63	23.86	40.66	16.42
	(III) TMD	39.64	36.91	45.00	26.14	40.00	22.89

3.3. Effectiveness of Number of TMDs and Effective Placement

The response quantities of the bridge in longitudinal as well as transverse direction are plotted for the uncontrolled controlled with STMD and d-MTMDs. The number of TMDs increased up to three and tuned to the corresponding modal frequency and optimally placed. Base shear in transverse for the case of STMD are reduced about 23%, 21% and 36% respectively for Imperial Valley, 1940, Loma Prieta, 1989 and Kobe, 1995. Fig.5 is implying that the d-MTMDs are more effective comparing to STMD. Base shear in transverse for the case of d-MTMDs are reduced about 34%, 26% and 39% respectively for Imperial Valley, 1940, Loma Prieta, 1989 and Kobe, 1995. The deck displacement in transverse is also reduced about 22 - 42% under different earthquakes for the case of STMD. The d-MTMDs are shown improved performance over STMD and under different earthquakes the deck displacement is reduced about 30 - 55% respectively for transverse direction. The systems are shown to be effective for the control of deck acceleration under the ground motions and it is reduced about 21 - 34% when the STMD is installed and about 36 - 46% when the d-MTMDs are installed. It is noted that the base shear reduced significantly in both directions after installation of STMD, TMDs and d-MTMDs. Base shear in longitudinal are reduced about 10 – 19%, 16 – 28%, and 11 – 23% respectively for STMD, TMDs, and d-MTMDs under different earthquake ground excitations.

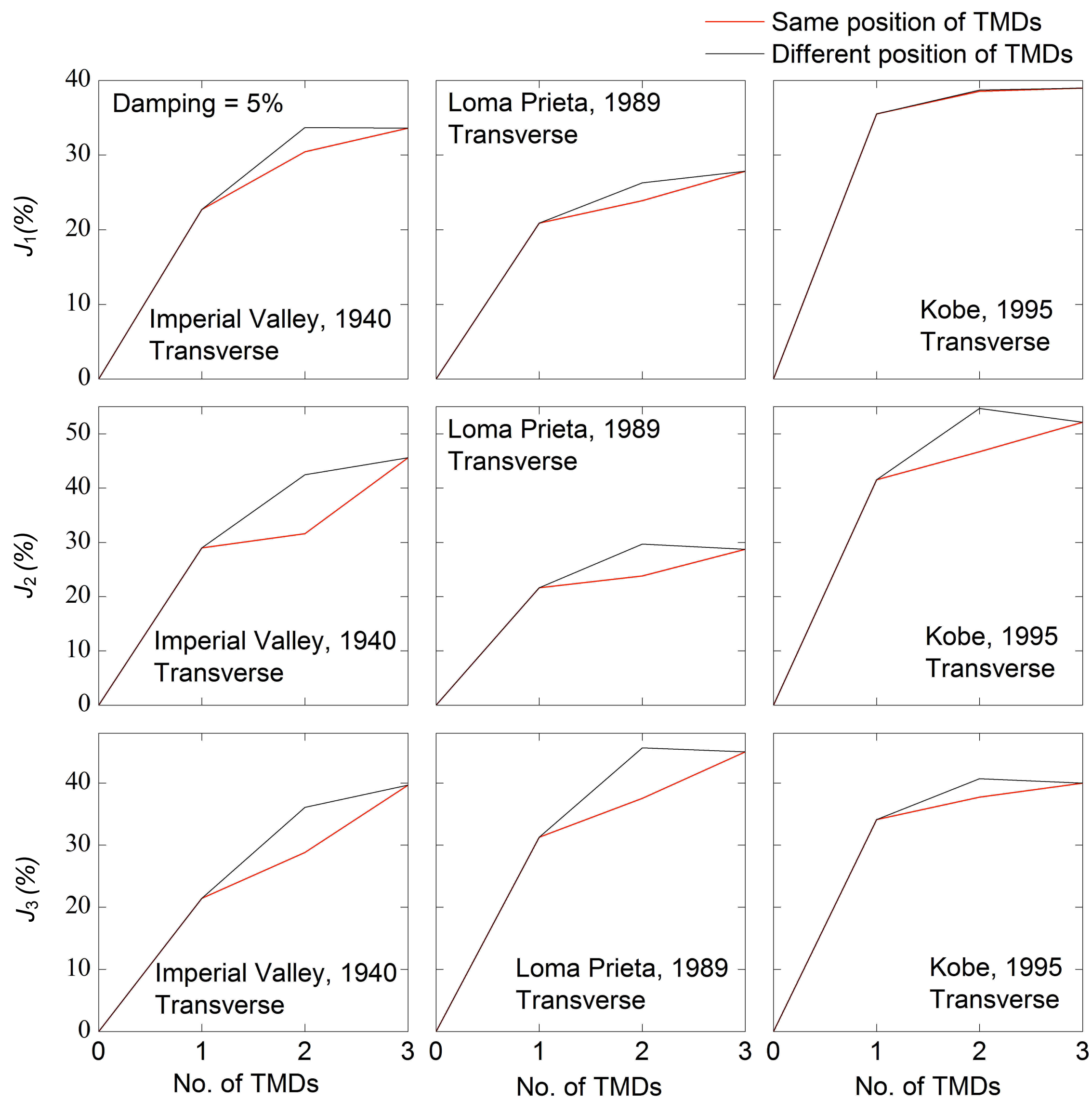

Fig. 5 – Comparing the percentages reduction response of concrete bridge from (pier base shear, deck displacement and deck acceleration) with different number of TMDs and different earthquakes ground motion (Imperial Valley, 1940, Kobe, 1995 and Loma Prieta, 1989) in transverse direction.

4. Conclusions

The effectiveness of MTMDs for seismic response control of three span continuous concrete bridges with SSI is presented. The following conclusions are drawn from the numerical study.

1. The displacement at the center of the deck increased significantly in bridges considered SSI.
2. Significant reduction in deck displacement is observed after installing dampers.
3. Base shear reduced significantly in both directions after installation of STMD, TMDs and d-MTMDs.
4. Base shear in transverse are reduced about 21 – 36%, 27 – 40%, and 26 – 40% respectively for STMD, TMDs, and d-MTMDs under different earthquake ground excitations.

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