Periodica Polytechnica
Mechanical Engineering

59(4), pp. 189-198, 2015
DOI: 10.3311/PPme.8327

RESEARCH ARTICLE

Vibration Mitigation of Shazand Railway Bridge Induced by Train Using Tuned Mass Damper (TMD)

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Received 13 June 2015; accepted after revision 03 September 2015

Abstract

In the last two decades, tuned mass damper has been used as a passive control method against earthquakes on tall buildings and bridges. In this paper, the effect of tuned mass damper system on the seismic behavior of 10 story structure with moment frame has been investigated taking into account the floor rigidity. In addition, the effect of TMD system is investigated on Shazand Railway Bridge located in Markazi province, Iran in north to south-west railway. The bridge consists of ten spans. In 1984, during Iran-Iraq war, that's main span, which has a length of 72 m, was attacked and severely damaged. Eight month later, damaged span replaced with a steel deck. The deck is straight in plan, but the railway is curved and this causes eccentricity. Excessive vibration was observed during the train passage. Although the bridge was retrofitted in two stages, the problem has not been solved yet, and the trains' speed should be reduced to around 10 km/hr in order to avoid excessive vibration. The present study addressed the effectiveness of tuned mass damper (TMD) in reducing train-induced vibrations of Shazand Railway Bridge. A three dimensional finite-element model of the bridge is developed and dynamic time history analyses under train passage in both as-built and passively controlled with TMD are conducted. Sensitivity analyses are performed to demonstrate the effects of the damper parameters on structural response. The results show that considerable reduction in acceleration response of the bridge can be achieved by employing proper TMD.

Keywords

Shazand, Railway Bridge, Vibration, Tuned mass damper

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1 Introduction

Given recent advances in the area of structural control against earthquakes and wind, the need for doing research on passive control systems such as TMD and finding their strengths and weaknesses in earthquake-prone countries is most felt [1]. TMD, like other control system for structures, aims at reducing the need for energy dissipation in the load-bearing elements of a structure against lateral loads. This decrease is achieved by movement of a heavy load on the load-bearing elements of the structure. TMD operation is based on dynamic vibration absorbers invented by Frahm in 1909. In 1956, Den Hartog [2] completed and corrected Frahm's work and later investigated damped and undamped dynamic absorber systems with damping of the original system. With the aim of improving the performance of dynamic vibration absorbers, Snowdon (1960) investigated solid absorber behavior in reducing the main system response. Results of this study showed that the dynamic absorber that used materials with stiffness proportional to frequency and a constant damping factor could significantly reduce the resonant vibration in the main system. In 1977, Jennige and Frohrib investigated a translatory–rotary absorber system numerically to control both bending and torsional modes in a building [3]. Most early studies focused on the use of dynamic absorbers in mechanical systems where only one operating frequency in resonance with the fundamental frequency of the machine. Since building structures are affected by environmental and natural loads, they have different frequency components. Selecting a TMD system can be based on several important parameters [4] such as efficiency, compactness and dimensions; economic cost of the system; implementation and installation costs; and maintenance and security of the system for buildings. In engineering design of a TMD, the dynamic response that can be achieved in practice is applied with a series of implementation considerations.

Shazand Railway Bridge is one of the most famous historical stone arch bridges in Iran. Shazand Railway Bridge was constructed in 1937 and opened to traffic in 1939. This bridge is located in of Markazi Railway route, between Samangan and noorabad. The bridge is near Shazand station, and it is

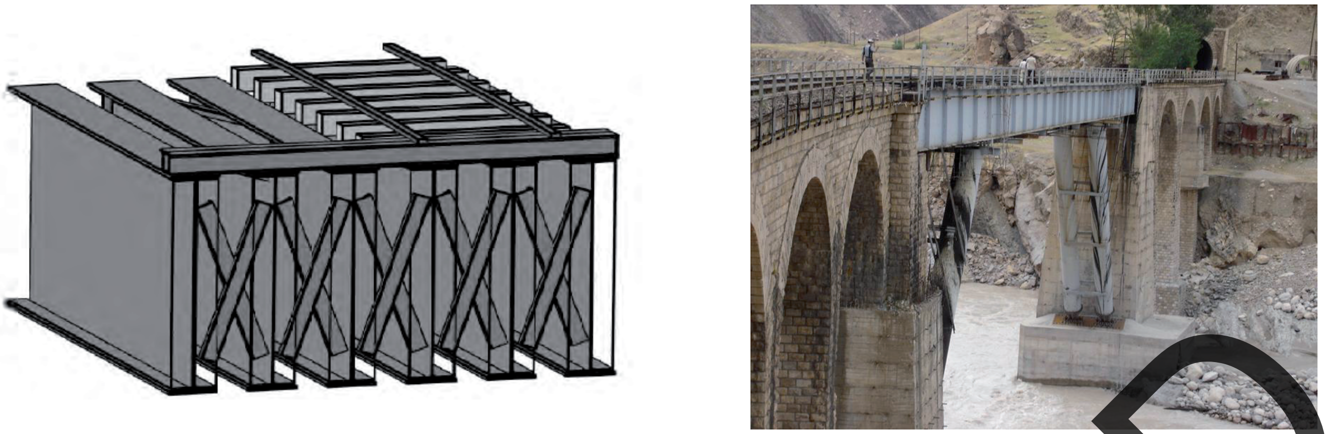


Fig. 1 L.H.S: Cross section of the bridge and railway. R.H.S: Shazand bridge after two retrofitting

constructed across a river. The bridge located between two tunnels and has 9 spans of 10 m and 1 main span with 72 m length. Railway of North to Southwest is one of the strategic routes of Iran. This route has a single rail line and has a light terrific density. This bridge had a significant effect in War between Iran and Iraq. In that war, the bridge repeatedly attacked by air strike but was not damaged significantly. In 1987, air attack caused severe failure of the main span of the bridge and the railroad was disconnected. It was necessary to reopen the supply route immediately. New steel deck designed, and implanted in 1990. The steel deck is straight in a plan, but the railway is curved and this caused eccentricity. Fig. 1 shows cross-section of the bridge and eccentricity of the rail line and bridge after two retrofitting. The length of the steel deck is 72 m and consists of six 2.5 m depth steel plate girders connected by cross brace and chord. At each abutment, the deck rests on six non-seismic elastomeric pads. The steel bridge had excessive vibration due trains passing; therefore, speed of trains limited to 6 kilometres per hour. During the war, the steel bridge was attacked several times, but was not seriously damaged. After the war, the bridge retrofitted in two stages. In the first stage, the bridge post-tensioned with 16 cables to reduce negative stresses. In the second stage, four oblique columns added to the bridge. These columns divided the bridge to 3 spans with length of 15, 42 and 15 m.

The problem did not solve entirely with these retrofitting, just speed of trains arose to 10 kilometers per hour. Field tests on the bridge showed that the acceleration of middle of the bridge reaches more than 1g. The ratio of maximum displacement of the system with damper to that of the system without damper was largely dependent on the form of earthquake input records. In the computer modeling (with SAP2000) of the TMD system, Non-Linear Link was used. Two Nllink members with damper character was placed on each other between the mass of TMD and the TMD junction with the structure. One of these has a stiffness equal to the stiffness needed in the TMD and a damping of zero and the other has a damping equal to that of the TMD and a stiffness of 0.01 (to prevent computational error). The TMD point mass is assumed to which the damper mass is allocated and the unnecessary degrees of freedom are bound. The behavior of the structure is estimated in the range of linear behavior. *Combined structure-TMD system* is shown in Fig. 2.

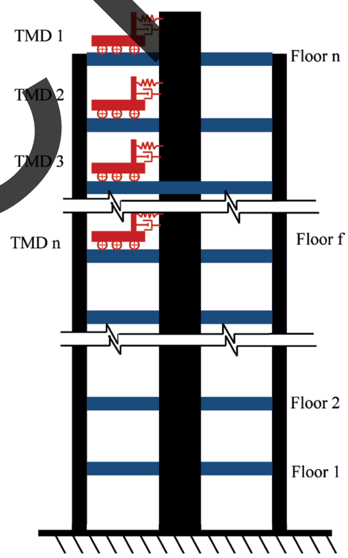


Fig. 2 The combined structure-TMD system

Using Tuned Mass Damper (TMD) is one of the techniques to suppress vibrations. TMD is a classical control device and it is made of a mass, a spring and a viscous damper. The concept of TMD was proposed by Frahm [1]. Den Hartog added a damper to Frahm's devices and proposed an optimal design for TMDs under harmonic load [2]. Philosophy of using TMD is a reduction of structural vibration energy. This reduction is accomplished by transferring some of the vibration energy of structure to the TMD. This devise has a variety of merits; it has permanent servise time, requires easy management and maintenance, and it can be simply incorporated into an existing structure with less interference on main performance of the structure. In summary, its advantages include compactness reliability, efficiency and low cost.TMDs have been extensively studied and applied to attenuate wind and earthquake induced vibrations. Most of the theoretical analyses, experimental researches and practical uses have been shown that TMD is an effective to suppress structural vibrations. Several researchers have investigated using of TMD in reducing the vibration of a bridge under traffic loads. Kwon et al. studied the use of TMD to suppress vibrations of a three-span bridge under TGV train passage. They concluded that TMD can reduce 21% of maximum displacement, and quickly suppress the free vibrations [3]. Wang et al. created numerical and analytical of Taiwan High-Speed Railway and investigated the performance of TMD to suppress the train-induced vibrations. The numerical results showed TMD can reduce up to 40% of acceleration of the moderately long bridge under train's passage. They concluded that interaction effect between bridge and train is of no significant effect on TMD's performance [4]. In present study the effectiveness of TMD in reducing train-induced vibrations of Shazand Railway Bridge is addressed. A three dimensional finite-element model of the bridge is developed and dynamic time history analyses under train passage in both as-built and passively controlled with TMD are conducted. Sensitivity analyses are performed to demonstrate the effects of the damper parameters on structural response.

2 Equations of Motion of the Bridge and TMD

For the formulation of bridge responses, these assumptions are used: 1- the bridge is idealized as having permanent stiffness in its length, 2- train is modeled as moving forces, 3- the train moves at a constant speed on the bridge, 4- only vertical modes of the bridge considered. The equation of motion can be expressed as

$$M\ddot{z}(x,t) + C\dot{z}(x,t) + Kz(x,t) = F_v(x,t) + F_T(t) \tag{1}$$

in which M= matrix of mass, C= matrix of damping, K= matrix of stiffness and $F_v(t)$ and $F_T(t)$ are interaction force between bridge and train and between TMD and the bridge, respectively. In all cases, initial set of loads are considered to be these of the first axle of the train when it enters the bridge. Let *ai* be the longitudinal distance from the *i*th train load set to the initial train load set on the bridge. R and K stand for the number of first and last train axles on the bridge. At time *t*, $F_v(x,t)$ and $F_T(t)$ are given by

$$F_v(x,t) = \sum_{i=R}^{k} \delta\left[x - (vt - a_i)\right] \times p_i \tag{2}$$

$$F_T(t) = C_T\left(\dot{z}\left(\frac{L}{2},t\right) - \dot{y}(t)\right) + K_T\left(z\left(\frac{L}{2},t\right) - y(t)\right) \tag{3}$$

where δ = dirac delta function, v = speed of a train p_i = static load of *i*th wheel of train, C_T and K_T = damping and spring constant of the TMD, respectively, $z(\frac{L}{2},t)$ = vertical vibration displacement of middle of the bridge, $y(t)$ = vertical vibration displacement of the TMD and a dot represents derivative with respect to time.

Jianzhong et al. investigated a simply supported girder bridge under high-speed trains with an emphasis on the resonant vibration. They proposed Eq. (4) for resonant speed of trains and concluded that the number of wheel-axle of train passage across the bridge is also an important factor in bridge resonant vibration [5].

$$v = \frac{\omega}{2\pi j} L_v \qquad (j = 1,2,3,\ldots) \tag{4}$$

in this equation L_v = distance between train axles, ω = fundamental frequency of the bridge.

3 Optimum parameters of Tuned Mass Damper

Den Hartog proposed formulas for obtaining optimum parameters of TMD. These formulas are obtained for reduction responses of structure with single degree of freedom under harmonic load excitation [2]. Den Hartog formulations are given as

$$f_{opt} = \frac{1}{1+\mu} \tag{5}$$

$$\xi_{opt} = \sqrt{\frac{3\mu}{8(1+\mu)}} \tag{6}$$

where μ is mass ratio of TMD. Having ξ_{opt} and f, optimum values of damping coefficient C_T and stiffness K_T of the TMD can be calculated as

$$K_{opt} = f_{opt}^2 \omega_s^2 m \tag{7}$$

$$C_{opt} = 2\xi_{OPT}\omega_s m \tag{8}$$

in which ω_s = Main structure frequency, m = TMD mass.

Sadak et al. proposed TMD optimum parameter for multi degree of freedom (MDOF) structure under earthquake excitation. Results shown that with the use of optimum parameter, significant reduction of structural responses can be achieved [6]. Sadak et al. formulations are given as

$$f_{opt} = \frac{1}{1+\mu\Phi}\left[1 - \beta\sqrt{\frac{\mu\Phi}{1+\mu\Phi}}\right] \tag{9}$$

$$\xi = \Phi\left[\frac{\beta}{1+\mu} + \sqrt{\frac{\mu}{1+\mu}}\right] \tag{10}$$

where β is the damping ratio of main structure, Φ is the amplitude of fundamental mode of vibration for a unit modal participation factor computed at location of TMD and μ is the mass ratio for MDOF structure. μ Can be calculated as

$$\mu = \frac{m}{\Phi_n^T M \Phi_n} \tag{11}$$

in which m is the TMD mass, M is the mass matrix of structure and Φ_n is fundamental mode shape normalized to have a unit participation factor.

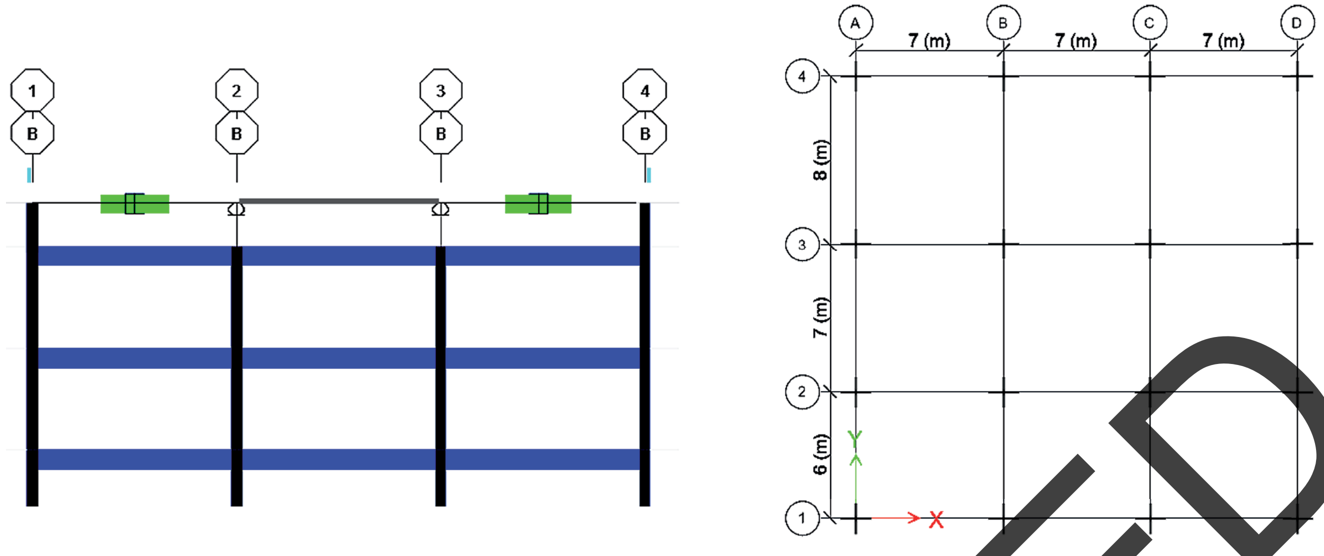


Fig. 3 Right Hand Side(R.H.S.): The building plan and L.H.S.: TMD System on the roof

4 Steel moment resisting structure (10-story)

In the first case, a steel structural system with moment connections and rigid diaphragm is studied. The plan and section of the building are shown in Fig. 3 (3m height of each story). Regarding the installation of TMD, it should be noted that because the oscillating mass applies force to the system at the top of the structure, the TMD movement direction and the point where the TMD force is applied should be around the stiffness center so that torsion is not created in the structure.

After modal analysis of the structure (mass limits for each story along x and y axes are assumed to be 79300 kg taking into account 500(kg/m^2) dead load and 200 (kg/m^2) live load). The dominating period for the structure along x was 2.54 seconds. The structure was investigated both with and without damping. The amount of μ (the ratio of the TMD mass to the total mass of the structure) is assumed to be 5%. Therefore, given the dominance of the first mode, the parameters of the optimum damper are as follows.

If $\mu = m / M$ equals 5%, then $0.05 = m / 79300$ and the damper mass will be 3965 kg. From (6), α_{opt} will be 0.952 and from (8), ξ_{opt} will be 0.1336. If the calculations are done along x, the TMD stiffness will be 21900(kg/s^2) and the TMD damping equals 2320(kg/s). The structure period became 3.11 after adding the damper as a result of the damper oscillation. In this case, the ratio of peak displacement after adding the damper to the peak displacement before adding the damper was 65%.

In this case, the TMD_structure system was investigated under El Centro earthquake (from 0 to 15 seconds) and Manjil-Rudbar (from 0 to 20 seconds) with a scale of 0,25g. Figures 4 and 5 respectively represent the roof displacement of under El Centro and Manjil-Rudbar earthquakes. Figures 6 and 7 are the base shears under earthquake. As can be seen in Fig. 4 and 5, in El Centro earthquake, the TMD with ratios of 3%, 5%, and 10% reduced 41%, 51%, and 60% of the maximum displacement respectively. In Manjil-Rudbar earthquake, it reduced 52%, 59%, and 70% of the maximum displacement respectively. Using TMD was also effective in reducing the base shears. As seen in Fig. 6 and 7, in El Centro earthquake, using TMDs of 3%, 5%, and 10% reduced 33%, 39%, and 44% of the base shear respectively. In Manjil-Rudbar earthquake, it reduced 10%, 20%, and 35% of the base shear respectively.

As can be seen, using the TMD was effective in reducing the amount of the base shear. In addition to balancing the structural vibration the TMD reduced the amount of shear force ate the structure base. This will require the use of lighter sections in the structure frames.

However, if the structure has a damping of 5% and a period of 2.54 seconds, the optimum damper parameters in Table 1 for ξ_s=5%, μ = 3% and μ = 0,05 obtained via interpolation between the values of μ = 0,03 and μ = 0,1 are as follows:

Table 1 Optimum parameters of TMD

5%	3%	μ	
0.912	0.938	α_{opt}	
0.1048	0.08567	ξ_{opt}	
20160	129500	K_{TMD}(kg/s^2)	
1875	955	C_{TMD}(kg/s)	
3965	2400	M_{TMD}(kg)	
3.113	2.963	T (sec) after TMD installation	
0.60	0.66	El Centro	Maximum ration

Figures 8 and 9 represent the absolute roof displacement of a damped structure under the two earthquakes. Comparing Figures 4, 5, 8 and 9 show that the effectiveness of TMD in an undamped structure is more tangible. The reason is that as shown in the diagrams (Fig. 4 and 5), in an undamped structure, due to lack of damping in the structure after starting vibration, the vibration increases and a resonant situation is created.

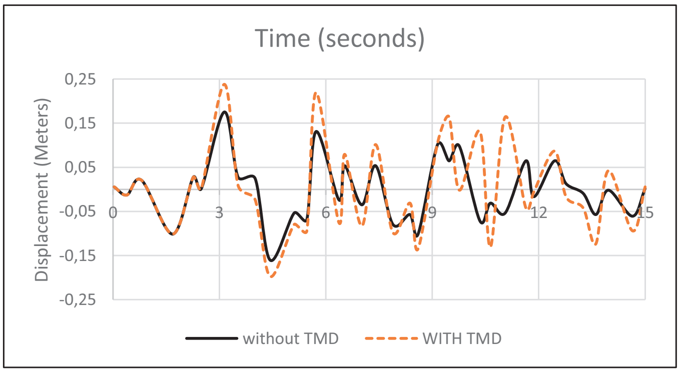


Fig. 4 Roof displacement of a point in undamped structure under El Centro earthquake

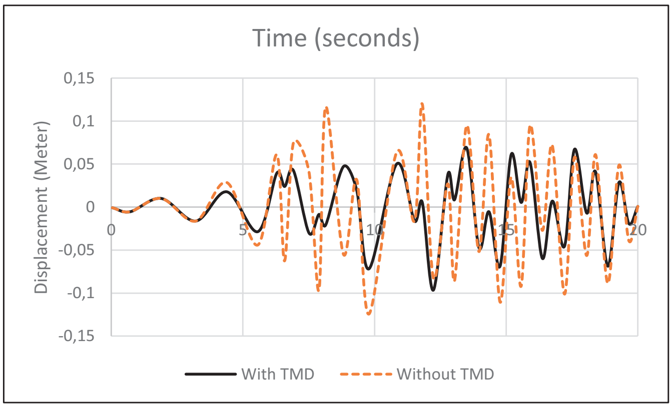


Fig. 5 Roof displacement of a point in undamped structure under Manjil earthquake

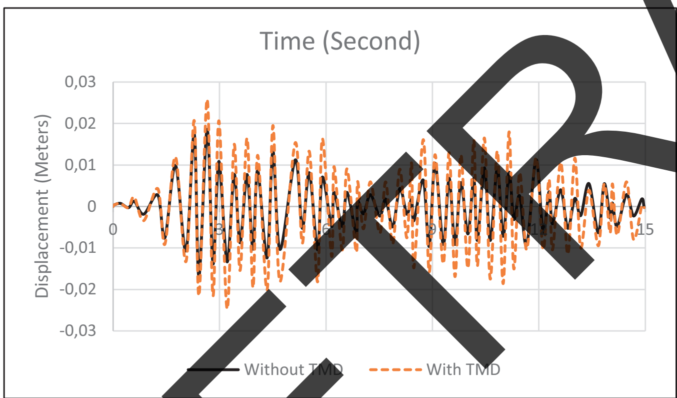


Fig. 6 Base shear in the undamped structure under El Centro earthquake.

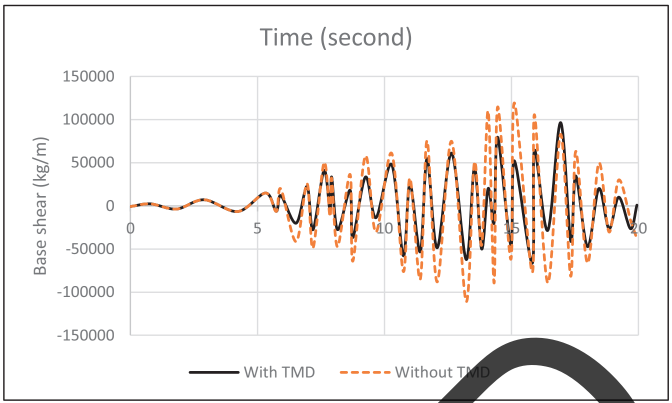


Fig. 7 Base shear in the undamped structure under Manjil earthquake.

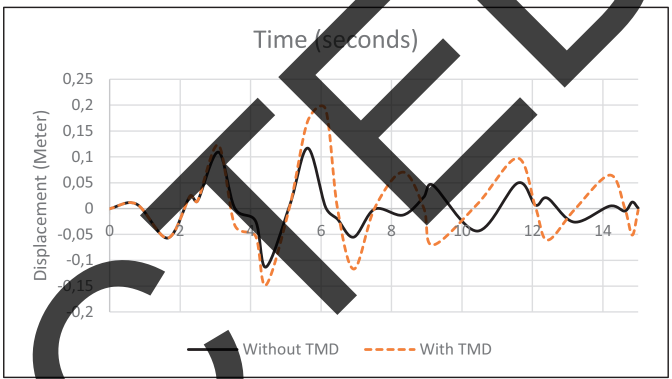


Fig. 8 Roof displacement of a point in the damped structure (ξ_s=5%) under El Centro earthquake

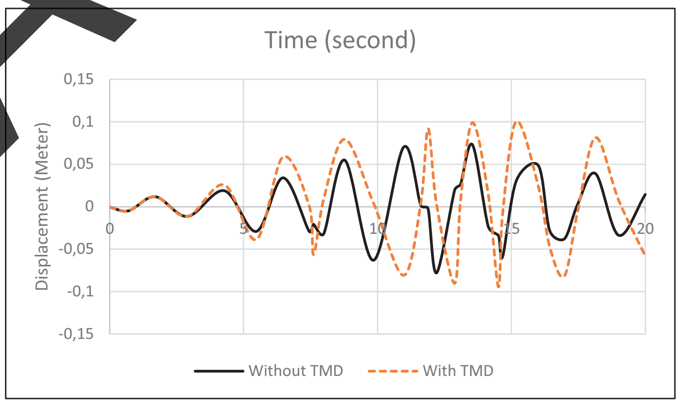


Fig. 9 Roof displacement of a point in the damped structure (ξ_s=5%) under Manjil earthquake

Using a TMD significantly dampen these vibrations. However, in the damped structure, damping of the structure itself help to reduce the maximums and keep away from a resonant situation. The maximum amounts established in the damped structure is lower and thus the effectiveness of TMD in the structure is significantly less than in the undamped structure.

4.1 Response of the bridge under train induced vibration

The external moving point loads that acting on the bridge and the longitudinal distance between axles of the train are shown in Fig. 10. The maximum fast Fourier transform (FFT) acceleration versus speed at midspan of the bridge is shown in Fig. 11. Figure 12 shows the time history of dynamic deflection and their FFT at midspan of the bridge when the train moves over the bridge at the resonant speed. The time histories of dynamic acceleration and their FFT at midspan in directions transverse and vertical directions of the bridge at the speed of 32 km/hr and 40 km/hr are shown in Fig. 13. The System parameters used in simulation are shown in Table 2.

ISO purposed that maximum sensitivity of human to acceleration in vertical direction is in the frequency range of 4 to 8 Hz [7]. These excessive vibrations have a disturbing effect on passengers.

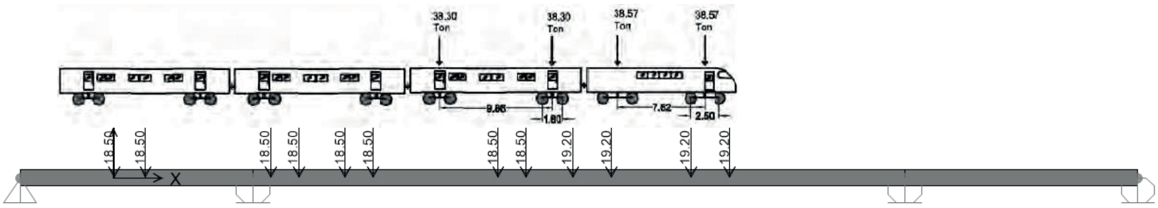
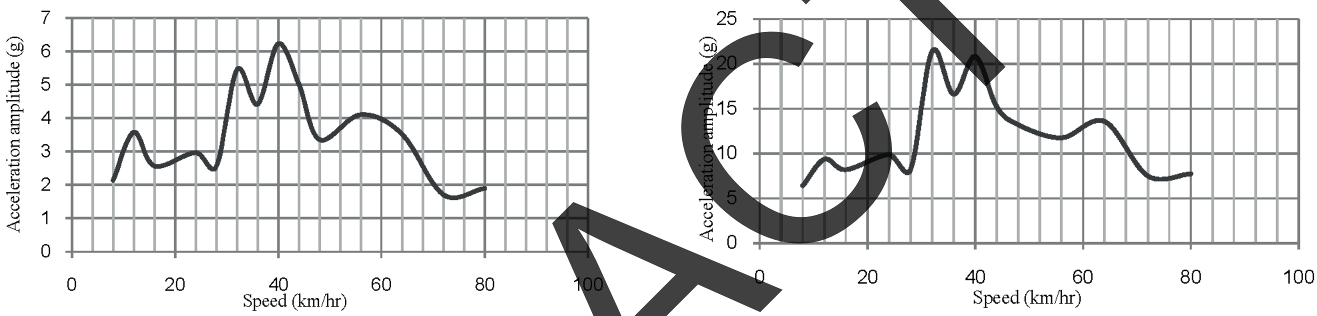


Fig. 10 moving point loads and longitudinal distances between axle of used train (ton-m)

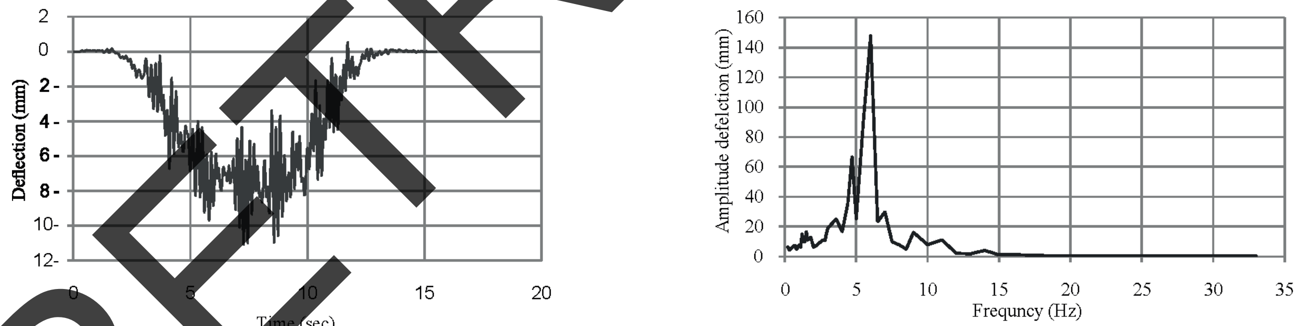
Table 2 System parameters used in simulation.

System	item	value
Bridge	*Mass(ton/m)*	18
	main span(m)	72.0
	Other spans (m)	10.0
	Damping ratio	1.0%
TMD	Mass ratio	2.0%
	Damping ratio	5.0%

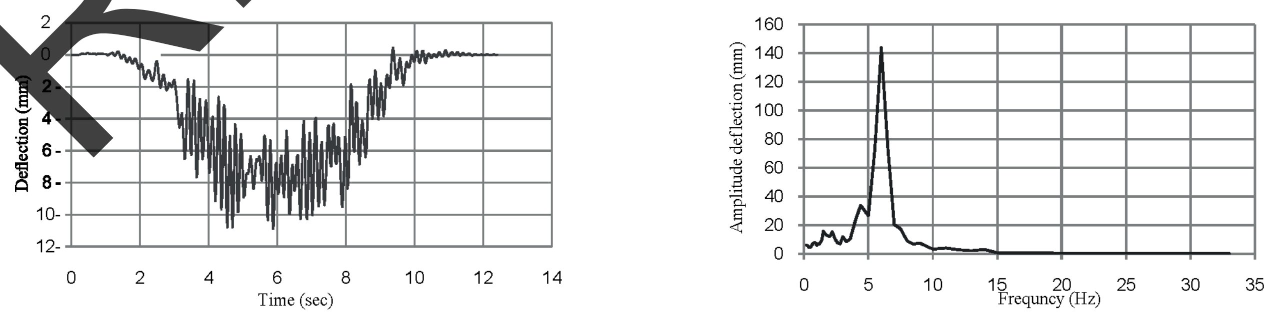


(a) (b)

Fig. 11 Acceleration-speed curves at midspan of bridge without TMD: (a) Transverse direction (b)Vertival direction.



(a)



(b)

Fig. 12 Time-histories of vertical deflection and their FFT at midspan of the bridge without TMD: (a) v = 32(km/hr) and (b) v = 32(km/hr)

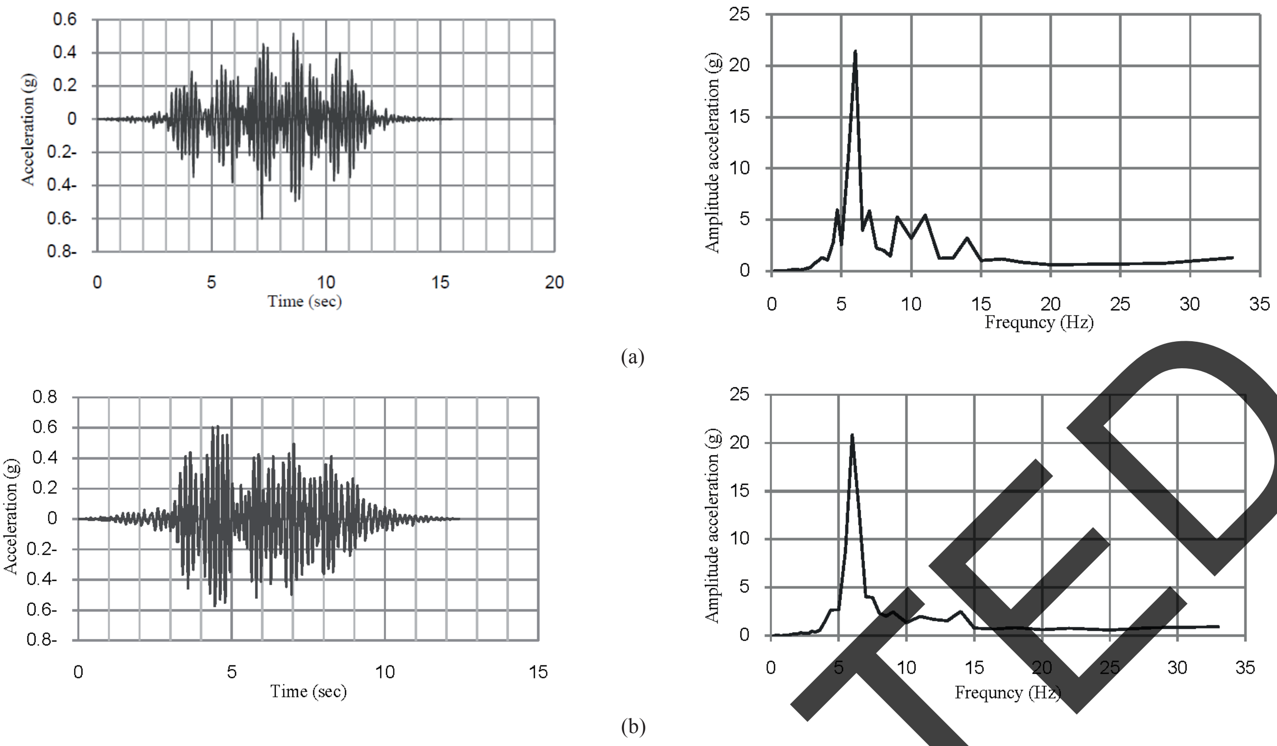


Fig. 13 Time histories of vertical acceleration and their FFT at midspan of the bridge without TMD: (a) v = 32(km/hr) and (b) v = 32(km/hr)

5 Response of the bridge with TMD

In this section, TMD is installed to reduce governing mode of vibrations of the bridge. Governing modes of bridge are shown in Table 3. The bridge has very closed two governing modes.

Table 3 Governing mode of bridge

Mode number	Frequency (HZ)	Mode shape
1	6.23	twisting
2	6.21	flexural

TMD is tuned to the frequency of vibration and its performance with 0.005 and 0.01 are investigated. Figure 14 shows the performance of TMD to reduce excessive vibrations.

5.1 Effect of tuned frequency in performance of TMD

In this section, the sensitivity of TMD to various tuning frequency is studied. Trains with different mass can change frequency of fundamental mode of the bridge. Effects of TMD's tuning frequency of are shown in Fig. 15.

TMD has effective performance when is tuned to the vicinity of the frequency of the governing mode of the bridge. Various types of trains with different masses have detuning effect because the bridge frequency is changed under trains passing.

5.2 Effect of Damping Ratio

Figure 16 shows the changes in the acceleration-frequency amplitude response curves of the bridge due to different values of damping ratios of TMD. Mass ratio and frequency of TMD are respectively, selected as 0.5% and 6.22 Hz.

From Figure 16, it can be seen that the optimum damping ratio of TMD is very small.

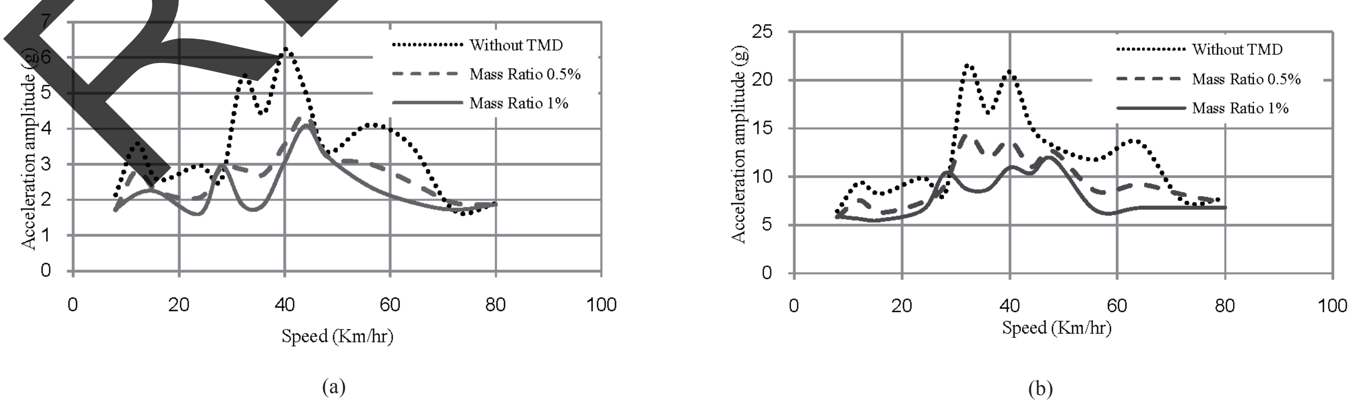
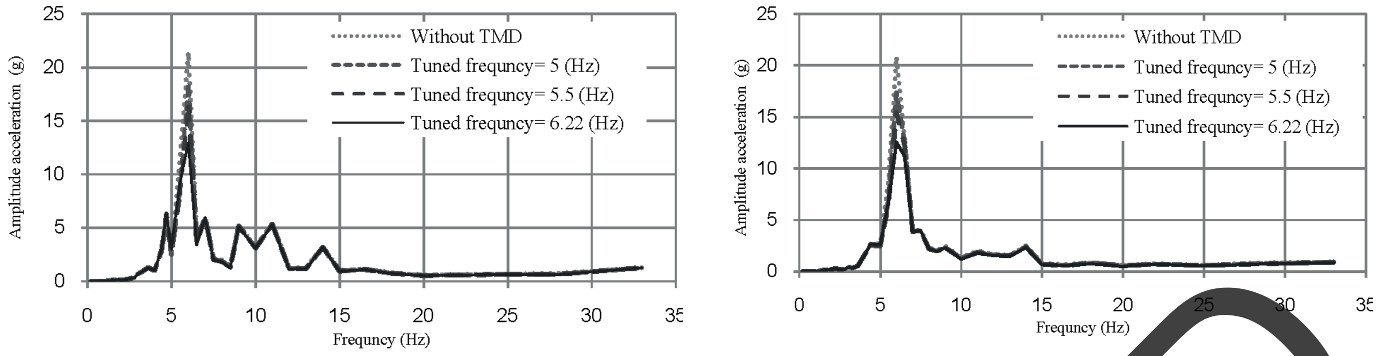
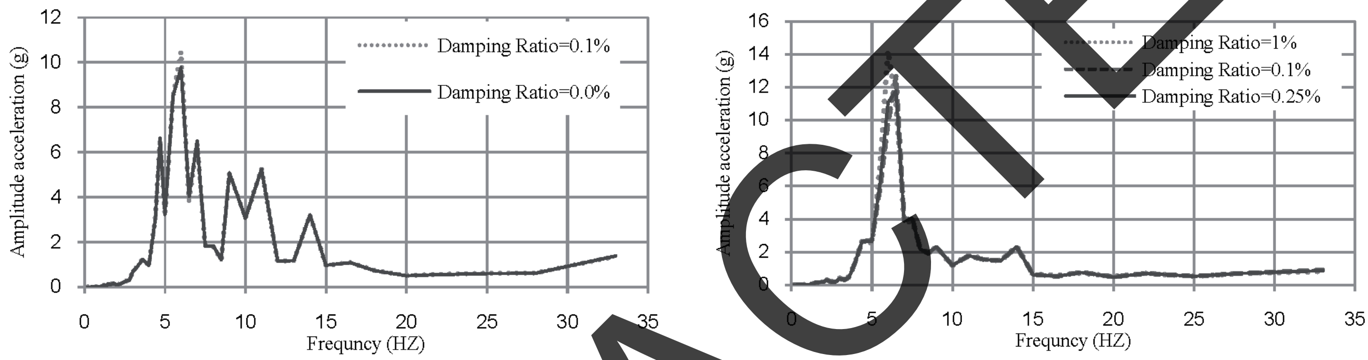


Fig. 14 Acceleration-speed curves at midspan of bridge with and without TMD: (a) Transverse direction; and (b) vertical direction.



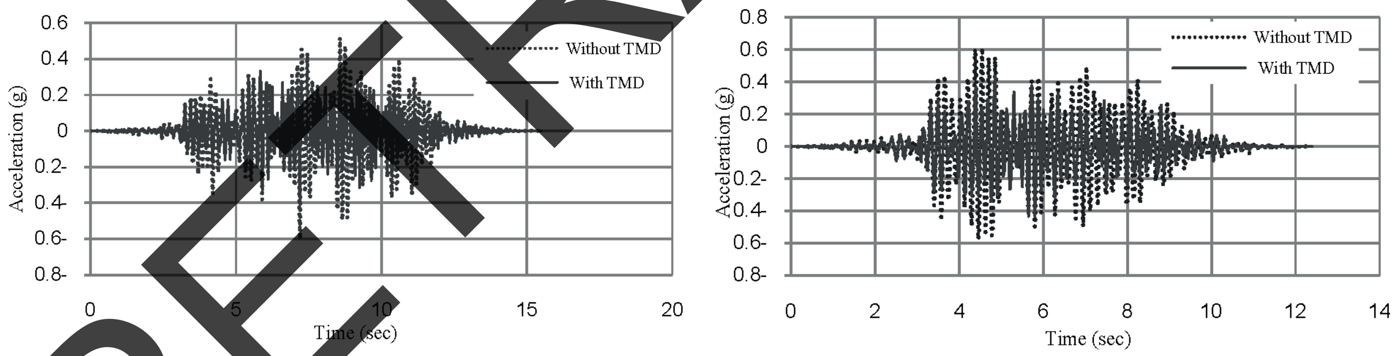
(a) (b)

Fig. 15 Acceleration-frequency curves at midspan of the bridge mass ratio of TMD=0.5%: (a) v = 32(km/hr) and (b) v = 40(km/hr)



(a) (b)

Fig. 16 Acceleration-frequency curves at midspan of bridge mass ratio of TMD=0.5% and tuned frequency =6.22 (HZ): (a) v = 32(km/hr) and (b) v = 40(km/hr)



(a) (b)

Fig. 17 Acceleration-time amplitude response curves at midspan of the bridge mass ratio of TMD=1% tuned frequency =6.22 (HZ): (a) v = 32(km/hr) and (b) v = 32(km/hr)

5.3 Effectiveness of TMD for Suppression of Resonant Phenomenon

The analyzed time–histories of deflection and acceleration at midspan of the bridge at the speed of 32 km/hr and 40 km/hr are shown in Figs. 17 and 18.

The results of the analyses are summarized in Table 4. The result shows that TMD is effective in reducing acceleration of the bridge, but it has not considerable effect on reducing displacement response of the bridge.

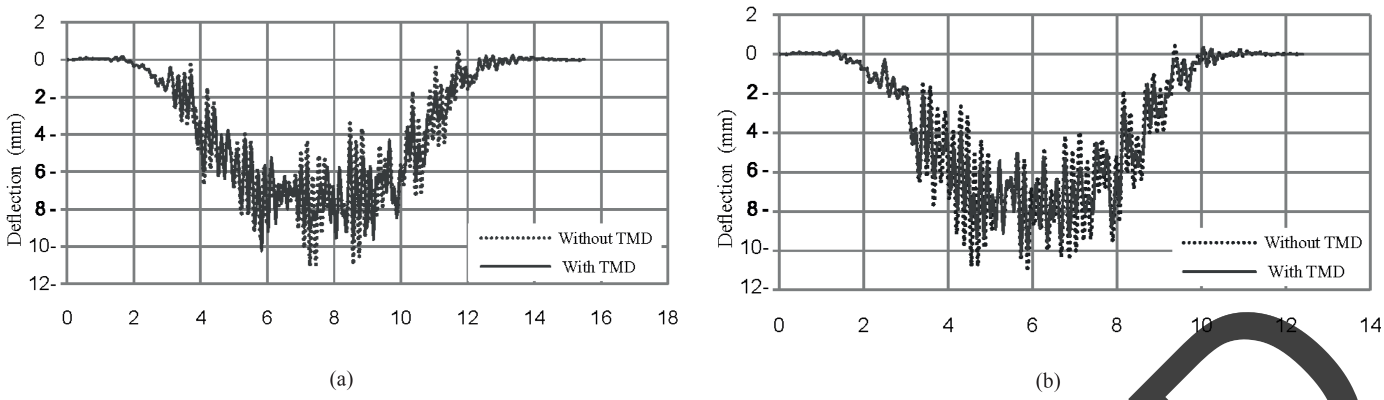


Fig. 18 Deflection-time amplitude response curves at midspan of the bridge mass ratio of TMD=1% and tuned frequency=6.22 (HZ): (a) v = 32(km/hr) and (b) v = 32(km/hr)

Table 4 maximal response of the bridge with and without TMD

Speed (km/hr)	TMD Condition	Maximal vertical Acceleration (g)	Maximal transverse acceleration (g)	Maximal vertical Displacement (mm)	maximal transverse displacement (mm)
32	Without TMD	0.52	0.18	-10.71	3.56
	TMD With 0.5% mass ratio	0.42	0.15	-10.47	3.36
	reduction percentage	18.24	17.01	2.22	5.49
	TMD With 1% mass ratio	0.34	0.13	-10.25	3.38
	reduction percentage	35.14	27.75	4.23	5.17
40	Without TMD	0.61	0.18	-10.89	3.28
	TMD With 0.5% mass ratio	0.48	0.15	-10.35	3.09
	reduction percentage	22.06	18.65	4.93	5.73
	TMD With 1% mass ratio	0.44	0.15	-10.23	2.98
	reduction percentage	27.76	15.78	6.07	8.96

6 Conclusion

The results of this study show the tangible and positive effect of TMD on structural vibrations. The important point regarding the effectiveness of TMD is the dominance of a mode in the structure (first mode), for which the TMD is designed and installed. Certainly, these relations work better for SDOF structures and the TMD designed for these structures perform better. Therefore, high-rise SDOF structures with more concentrated mass in high altitude such as telecom towers are the best choice for TMDs to reduce the vibrations caused by wind and earthquake in them. Other results show more effectiveness of TMD system in undamped structures. Regarding the behavior of the TMD it should be noted that the damper is activated after the first maximum input load (or acceleration) and has the ability to control the next maximums. Thus, the ratio of maximum displacement of the system with TMD to that of a system without TMD depends on the shape of the input record (earthquake).

In this second case study, TMD is used to suppress the vibration of Shazand Railway Bridge under passing train. The performance of TMD with parameters of the tuned frequency, damping ratio and mass ratio are investigated. From the results the following conclusions may be drawn:

- When natural frequencies of the bridge are multiples of the frequency of the train excitation, the resonant effect will occur and cause excessive vibrations of the bridge.
- The bridge has two close natural modes that cause two close resonant speeds. The resonant speeds of tested trains are 32 km/hr and 40 km/hr.
- TMD with optimum parameter, significantly reduces acceleration of the bridge, but it has not very effective in suppressing deflection of the bridge.

Acknowledgments

The work was executed with financial support from petrochemical industries development Management Company of the Iran.

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