Earthquakes and Structures, *Vol. 18, No. 5 (2020)625-635*
DOI: https://doi.org/10.12989/eas.2020.18.5.625

TMD parameters optimization in different-length suspension bridges using OTLBO algorithm under near and far-field ground motions

Hamed Alizadeh[a] and H.H. Lavasani*

Department of Civil Engineering, Kharazmi university, No.43. South Mofatteh Ave., Iran

(*Received March 11, 2019 Revised October 1, 2019, Accepted November 6, 2019*)

Abstract. Suspension bridges have the extended in plan configuration which makes them prone to dynamic events like earthquake. The longer span lead to more flexibility and slender of them. So, control systems seem to be essential in order to protect them against ground motion excitation. Tuned mass damper or in brief TMD is a passive control system that its efficiency is practically proven. Moreover, its parameters i.e. mass ratio, tuning frequency and damping ratio can be optimized in a manner providing the best performance. Meta-heuristic optimization algorithm is a powerful tool to gain this aim. In this study, TMD parameters are optimized in different-length suspension bridges in three distinct cases including 3, 4 and 5 TMDs by observer-teacher-learner based algorithm under a complete set of ground motions formed from both near-field and far-field instances. The Vincent Thomas, Tacoma Narrows and Golden Gate suspension bridges are selected for case studies as short, mean and long span ones, respectively. The results indicate that All cases of used TMDs result in response reduction and case 4TMD can be more suitable for bridges in near and far-field conditions.

Keywords: suspension bridge, tuned mass damper, meta-heuristic optimization algorithm, near-field ground motions, far-field ground motions

1. Introduction

The extended in plane configuration of suspension bridges makes them vulnerable against ambient vibration like traffic, wind and earthquake loading. In a suspension bridge, the vertical vibration can be created by the vertical component of ground motions and the longitudinal oscillation of support points due to horizontal component of the earthquake. Vibration problem of suspension bridges contains two distinct parts. In part one, the vibration of the pylon-pier system dominates while in the second part the vibration of suspended structure is dominant (Abdel-Ghaffar and Rubin 1983).

Nowadays, this fact is admitted that severe collapses are seen in the vicinity of fault (Maniatakis *et al.* 2008). The specifications of ground motions variate based on the fault distance, and according to distance they are divided to two various types called near and far-field ground motions. Also, there are other features distinguishing the near field ground motions, like directivity, hanging-wall, fling step, and velocity pulse, vertical and rotational component from far fault ones (Grimaz and MaliSan 2014). Vertical component of an earthquake tends to concentrate its energy on a high frequency narrow band that can be destructive for structures which their fundamental period lies between in the mentioned band (Elnashai and Papazoglou 2007). Peak ground acceleration is one of the most important features of seismic excitation (Colliera and Elnashai 2010), and it minimizes based on the increasing of distance from the fault (Memarpour *et al.* 2016).

In the last decades, vibration control of structures like tall buildings and bridges was studied by many researchers. Vibration control strategies are divided to four groups named active, semi-active, passive, and hybrid systems. TMD as a passive control system contains a mass, spring and damper in the simplest form (Elias and Matsagar 2017). Wong and Chee (2004) reported that TMD could reduce the maximum strain and kinetic energies of structures. Pourzeynali and Datta (2002a) addressed the effect of TMD parameters on increasing of flutter velocity, and finally provided the optimum value of each parameter to maximum increase. Wang *et al.* (2003) reported that TMD is a proper control strategy to decrease the vertical amplitude of vibration of bridges. Yau and Yang (2004) utilized multi TMD systems to reduce the multiple resonant response of cable stayed bridges due to oscillation caused by travelling of high speed train. Poorzeynali and Esteki (2008) used trial and error method to mitigate the vertical vibration of the Vincent Thomas suspension bridge under earthquake loading, and eventually they reported optimum value for each parameter. Chen and Wu (2008) studied the performance of MTMD in controlling of the vibration of bridges under wind load. Casciati and Giuliano (2009) purposed a multi TMD system needing an innovative parameter namely frequency rang instead of choosing single value to tuning frequency as customary procedure, and addressed its efficiency in the towers of suspension bridge under gust loading. It found out that selecting an appropriate frequency range having effective bandwidth can

*Corresponding author, Assistant Professor
E-mail: lavasani@khu.ac.ir
[a]Ph.D. Student
E-mail: std_h.Alizadeh72@khu.ac.ir

ISSN: 2092-7614 (Print), 2092-7622 (Online)

considerably suppress the vibration. Tubino *et al.* (2016) attached two TMDs to the footbridge in order to control the vibration of it under various conditions of loading. The results indicated that TMDs could suitably reduce the ultimate responses in all conditions of loading. Alizadeh *et al.* (2018) investigated the sensitivity of flutter velocity into the gyration radius and placement of TMDs.

Chen and Huang (2014) specified an upper limit for the mass ratio which was 15% and stated that exceeding from the mentioned limit decreased the performance of TMD. But too heavy TMDs are not recommended and change the natural characteristics of structures, so the optimum values should be selected (Tao *et al.* 2017). The performance of control systems is improved by minimizing of whole energy under specific condition or response quantities like displacement, and this minimization is the basis of optimization algorithms. Determination of characteristics of control system is the integral aid of optimization methods. Control systems can be classically optimized by gradient-based search method. Meta-heuristic optimization algorithm like genetic is another method providing the optimum value in a feasible and complex discontinues space (Pourzeynali *et al.* 2007). Rao *et al.* (2011) firstly introduced the teaching-learning-based optimization as a Meta-heuristic algorithm. It includes two independent part namely teaching and learning phase. In the first part the mean scientific level of the students is improved by the teacher while in the second part the students improve their level by interaction between together. Although usefulness of the mentioned method, introducing a surplus phase called observer would provide a new solution to improve the level of the student briefly called OTLBO (Shahrouzi *et al.* 2017).

Li *et al.* (2010) reported that efficiency of MTMDs optimized by the random optimization is more appropriate in compare to others optimized by classical methods. Ubertini *et al.* (2015) addressed different vibration control strategies single TMD, MTMD and MTMDs in deterministic and non-deterministic design conditions. Both design procedures eventuated about same value for optimum tuning frequency and position of TMD. Also, the non-deterministic method tended to increase the mass ratio of TMD in all of three conditions, because it would decrease the failure probability. Miguel *et al.* (2016) optimized TMDs in vehicle-bridge using robust design optimization. The results showed that two or three TMD could result in further improvement and caused ultimate less vertical displacement of the center span's middle node in compare to using single TMD. Pisal and jungid (2016) reported that optimized MTMD placed at the middle point of span and distributed along bridge length is more effective than optimized STMD in response reduction of bridge. Miguel *et al.* investigated the robust optimized TMDs in controlling the vibration of buildings and bridges.

In this paper, the performance of TMD is optimized under a complete set of ground motions compressing twenty near and far-field ground motions by OTLBO algorithm. In this regard, TMD devices are attached to the deck in three different cases. The Vincent Thomas, Tacoma Narrows and Golden Gate suspension bridges as the short, mean and long span bridges, respectively, are chosen for numerical analysis. The structural properties matrices and motion equations can be written by the finite element method and using energy principle. Finally, the numerical analysis is done and the results will be provided.

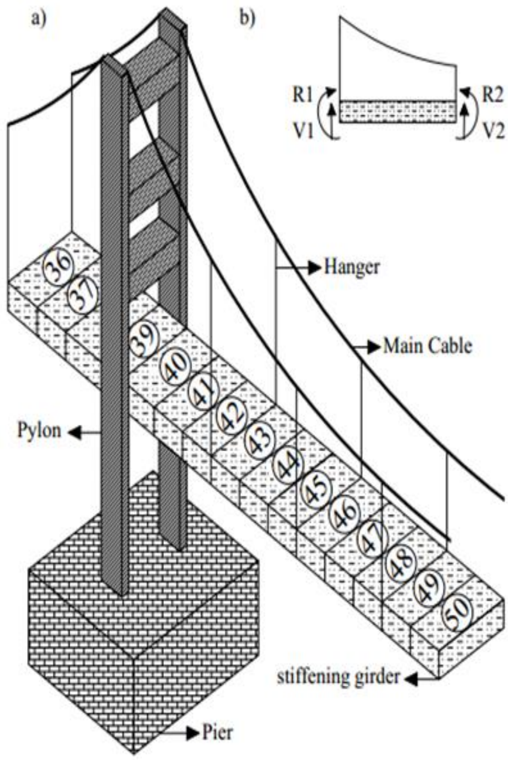


Fig. 1 Finite element model of bridge

2. Equation of motion

In order to dynamic analysis of suspension bridges, 2-D and 3-D models can be utilized. When the behavior of all parts of bridge like suspended deck, main cables, towers and piers is considered as a unique system, the 3-D model will be mostly adapted (Hosseini Lavassani et. al. 2020). It has been indicated that 2-D model can successfully provide the natural modes shapes and frequencies (Pourzeynali and Datta 2002b). In this study, the lumped mass matrix is used to obtain the natural characteristics of 2-D models. In this regard, the suspended structure is discretized to some certain beam elements. Each element containing stiffening structure (girder), main cables, and at least two hangers shown in Fig. 1. The hangers are assumed to be inextensible, so the displacement of the main cables and stiffening structure is the same, and consequently considering one node at the centerline of the girder is enough. Hence, each element compresses two nodes at the end parts. Each node has two degrees of freedom that one of them is vertical displacement $(V_1 . V_2)$ while another one is the bending rotation $(R_1 . R_2)$.

Total potential energy of the structure is divided into three independent parts which are (i) the strain energy stored in the girder due to effects of the bending moment, shear and normal forces, (ii) the strain energy stored in the cables due to the tension load, and (iii) gravity energy of the cables resulted in by the lowered position of the dead loads (see the formulas stated by Abdel-Ghaffar (1980)).

The interpolation functions related to the vertical and rotation degrees of freedom are presumed to be cubic Hermitian polynomials. In this regard, the deflection shape function of an element can be written as follows

$$v_e(x.t) = \{f(x)\}_e^T\{q(t)\}_e \tag{1}$$

In which, $f(x)$ and $q(t)$ are the vector of polynomials and nodal displacement. In order to evaluate the structural property matrices of bridges, the potential and kinetic energies should be written in account to the nodal displacement.

The results are represented here and the more details of evaluation of matrices were stated by Abdel-Ghaffar (1980). The stiffness matrix is formed from three independent parts namely k_{cg}, k_{ce} and k_{ge} denoting gravity stiffness and elastic stiffness matrices of cable, and elastic stiffness of girder, respectively. Also, the mass matrix has a single part and is represented by m_t.

$$[k_{ge}]_e = \frac{E_g I_g}{L^3}\begin{bmatrix} 12 & -6L & -12 & -6L \\ -6L & 4L^2 & 6L & 2L^2 \\ -12 & 6L & 12 & 6L \\ -6L & 2L^2 & 6L & 4L^2 \end{bmatrix}. \quad [k_{cg}]_e = \frac{-H_w}{30L}\begin{bmatrix} 36 & -3L & -36 & -3L \\ -3L & 4L^2 & 3L & -L^2 \\ -36 & 3L & 36 & 3L \\ -3L & -L^2 & 3L & 4L^2 \end{bmatrix}$$

$$[k_{ce}]_e = \frac{E_c A_c}{L_E} * \left(\sum_{i=1}^{3}\frac{W_d}{H_w}\{\hat{f}\}_{N_i}\right) * \left(\sum_{i=1}^{3}\frac{W_d}{H_w}\{\hat{f}\}^T{}_{N_i}\right). \quad \{\hat{f}\}^T = \left[\frac{L}{2}.\frac{-L^2}{12}.\frac{L}{2}.\frac{L^2}{12}\right] \tag{2}$$

$$[m_t]_e = \frac{m_t L}{420}\begin{bmatrix} 156 & -22L & 54 & 13L \\ -22L & 4L^2 & -13L & -3L^2 \\ 54 & -13L & 156 & 22L \\ 13L & -3L^2 & 22L & 4L^2 \end{bmatrix}$$

In which, L, L_E, A_c, and I_g are the span length, virtual length, Cross section of the cable and moment inertia of the girder, respectively. W_d and H_w are the Dead load of the whole suspended structure and Cable force. Also, E_g and E_c show the Modulus of elasticity of the girder and cable, respectively.

In this regard, the structural properties matrices of one element should be evaluated using Eq. (2). the stiffness matrix is $[k_{ge}]$e + $[k_{cg}]$e + $[k_{ce}]_e$ and mass matrix is $[m_t]_e$, and this computation should be done for another elements of left side span. Center and right side spans have the similar evaluation. Eventually, after assembling the computed matrices the rows and columns corresponded to vertical degree of freedom of final elements of each span should be eliminated from the matrices, because two-hinged girder are utilized.

Now, by using the potential and kinetic energy of the structure and applying the Hamilton's principal, the

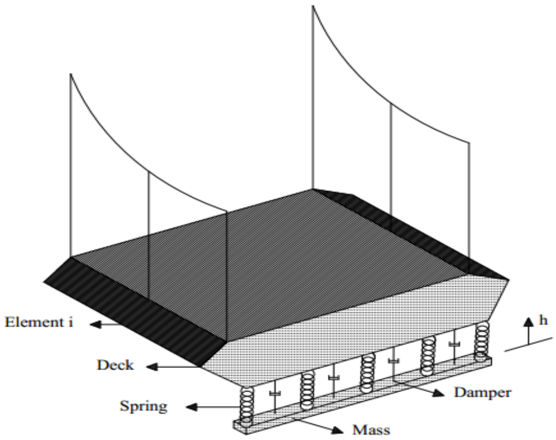


Fig. 2 Details of TMD's attachment to the suspended structure

equation of motion can be written as follows (Abdel-Ghaffar, 1980)

$$[\mathrm{M}]\{\ddot{u}(t)\} + [C]\{\dot{u}(t)\} + [K]\{u(t)\} = -[M]\{r\}\ddot{u}_g(t) \tag{3}$$

M. C and K represent the mass, damping and stiffness matrices. $u(t)$ represents the displacement response. By the way, number of dots signify the order of the derivation in account to time. Also, the stiffness matrix of the symmetric mode is as follows

$$[K] = [K_{CG}] + [K_{CE}] + [K_{GE}] \tag{4}$$

And for antisymmetric mode mentioned matrix is as follows

$$[K] = [K_{CG}] + [K_{GE}] \tag{5}$$

The motion equation can be comfortably solved in the state-space:

$$\{\dot{Z}\}_{2n*1} = \begin{bmatrix} 0_{n*n} & I_{n*n} \\ -M^{-1}K & -M^{-1}C \end{bmatrix}_{2n*2n}\{Z\}_{2n*1} + \begin{Bmatrix} 0_{n*1} \\ 1_{n*1} \end{Bmatrix}_{2n*1}(r)\ddot{u}_g(t). \quad \{Z\} = \begin{Bmatrix} u_T \\ \dot{u}_T \end{Bmatrix} \tag{6}$$

In which, Z ans I are the state vector and unit matrix. Also, the ultimate response can be stated as follows

$$\{d\} = [C1]\{Z\}. \quad [C1] = \begin{bmatrix} 1 & \dots & 1 & 0 & \dots & 0 \\ 0 & \dots & 0 & 0 & \dots & 0 \end{bmatrix}_{2*2n} \tag{7}$$

3. Equation of motion with TMD

A tuned mass damper is a passive control system containing a mass, spring and damper in the simplest form as seen in Fig. 2. It is attached to the structures to decrease the response of them under dynamic loads like ground motions, and wind. The natural frequency of TMD should be adjusted to the fundamental mode of the structures to occur resonance in order to dissipate the enormous part of dynamic loading's energies by dampers (Amini and Doroudi 2010, Debbarma and Das 2016).

In Fig. 2, element i compresses nodes i and $i+1$ and TMD can be attached to one of them while is distributed along the cross section. By the way, h denotes the vertical displacement of TMD signifying the vertical degree of freedom. In the pure vertical mode of suspension bridges, TMD contains three main parameters called mass ratio (m_T), damping ratio (ξ_T) and tuning frequency (ω_N).

$$K_T = m_T(\omega_N)^2 \tag{8}$$

$$C_T = 2\xi_T m_T \omega_N \tag{9}$$

N represents the specific mode that TMD should be adjusted to it. Adding of per TMD to the bridge increase one more degree of freedom. The stiffness, damping and mass matrices with the presence of one TMD have a little changes given by following equations

$$[K_S] = \begin{bmatrix} PU & PU & PU & \{0\} \\ PU & (K_{PU}+K_T)_{m\times m} & PU & (-K_T)_{m\times(n+1)} \\ PU & PU & PU & \{0\} \\ \{0\} & (-K_T)_{(n+1)\times m} & \{0\} & K_{T\,(n+1)\times(n+1)} \end{bmatrix}_{(n+1)\times(n+1)} \tag{10}$$

$$[C_S] = \begin{bmatrix} PU & PU & PU & \{0\} \\ PU & (C_{PU}+C_T)_{m\times m} & PU & (-C_T)_{m\times(n+1)} \\ PU & PU & PU & \{0\} \\ \{0\} & (-C_T)_{(n+1)\times m} & \{0\} & C_{T\,(n+1)\times(n+1)} \end{bmatrix}_{(n+1)\times(n+1)} \tag{11}$$

$$[M_S] = \begin{bmatrix} [M]_{n\times n} & \{0\}_{n\times 1} \\ \{0\}_{1\times n} & m_{T\,(n+1)\times(n+1)} \end{bmatrix}_{(n+1)\times(n+1)} \tag{12}$$

PU denotes the pure component of bridge's structural properties and sub-index s shows combined system of bridge and TMD. Also, the configuration can be simply generalized for using more TMDs.

4. OTLBO algorithm

Meta-heuristic optimization algorithm starts with an initial population of solutions and continuously moves to optimum solution from one generation to another (Vallada and Ruiz 2011). TLBO is a population based algorithm inspired from the scientific progress of the learners improved by teacher and interaction of the learners. A teacher is a person with high level of science trying to promote the scientific level of students and as far as he can bring them to his level. In each iteration, the best solution is selected as the teacher. The scientific level of learner is improved by training of teacher and also interaction between all members of the class. So, TLBO has two independent phase namely teacher and learners.

4.1 Teacher phase

In each iteration a solution causing the best solution is selected as the teacher responsible to upgrade the mean scientific level of the leaners making to generation the novel learners. Let X represents the learners (Rao *et al.*)

$$X_i^{new} = X_i^{old} + r_i(X_t - T_F X_{mean}^{old}) \tag{13}$$

T_F is the teaching factor and is either 1 or 2 and r is a random variable placing between [0,1]. Also, Eq. (13) expresses that all the learners promote up to difference of teacher level from the mean level of the learner. Of course, r and T_f specify how of the success of the teacher in upgrading of the learners.

4.2 Learner phase

Learner or learning phase is the second part of TBLO in which learners try to enhance their level by interplay with together. If X_i and X_j be randomly selected through the population such that $i \neq j$ the learner phase will be completed by flowing loop (Rao *et al.* 2011):

$$\begin{gathered} for \quad i = 1{:}N_p \\ if \quad f(x_i) < f(x_j) \\ X_i^{new} = X_i^{old} + r(X_i - X_j) \\ else \\ X_i^{new} = X_i^{old} + r(X_j - X_i) \\ end \\ end \end{gathered}$$

In order to prevent from trap of local extremums, one independent phase called observer is added to the algorithm. The new improved algorithm is called OTLBO.

4.3 Observer phase

The observer is produced by combination of all learners through following loop (Shahrouzi *et al.* 2017)

$for \quad j = 1{:}N_V$

$l =$ is a random number between 1 up to N_P

$X_l^{exp} = X_l^j$

end

Now a new solution is defined that can be accepted if the following condition occur:

$$if \quad f(x_i) < f(X^{new}) \rightarrow \quad x_i = X^{new}$$

5. Problem definition

Four independent parameters called mass ratio, damping ratio, tuning frequency and position of TMD should be optimized by OTLBO algorithm. Mentioned parameters are optimized in account to each span's features vice versa to tradition procedure that first three parameters are optimized considering to the whole structure. Due to symmetricity between left and right spans all parameters are optimized according to the one side and center spans. Three

Table 1 information of selected ground motions

Type	No.	Earthquake	Station	Year	Distance to Fault (m)	magnitude	PGA(g)	PGV(cm)	PGV/ PGA
Near-Field ground motions	1	Bam	Bam	2003	0.05	6.6	0.97	39.21	0.04
	2	Northridge	Rinaldi	1994	7.1	6.7	0.852	51	0.059
	3	Landers	Lucern	1992	2.19	7.28	0.823	41.07	0.049
	4	Tabas	Tabas	1978	1.79	7.35	0.688	44.4	0.064
	5	Imperial valley	El Centro #7	1979	0.56	6.53	0.544	56.28	0.103
	6	Kobe	Kobe University	1995	0.9	6.9	0.452	18.47	0.04
	7	Chi-Chi	TCU129	1999	1.83	7.62	0.342	39	0.114
	8	Northridge	Newhall	1978	1.79	6.7	0.29	37.21	0.128
	9	Erzincan	Erzincan	1992	2	6.7	0.248	18.33	0.073
	10	Kocaeli	Izmit	1999	3.62	7.51	0.145	12	0.082
Far-Field ground motions	11	Chi-Chi	TCU045	1999	26	7.62	0.356	21	0.058
	12	Montenegro	Herceg Novi	1979	23.59	7.1	0.21	4.87	0.023
	13	Duzci	Bolu	1999	17.6	7.1	0.203	17.36	0.085
	14	Irpinia	Brienza	1980	22.54	6.9	0.203	12	0.059
	15	Kobe	Kakogawa	1995	22.5	6.9	0.171	10.94	0.063
	16	Tottori	OKY004	2000	19.72	6.61	0.173	6.93	0.04
	17	Landers	Coolwater	1992	19.74	7.28	0.17	10	0.058
	18	San Fernando	Castic-Old Ridge	1971	19.33	6.61	0.167	4	0.023
	19	Kocaeli	Faith	1999	53.34	7.51	0.133	9	0.067
	20	Parkfield	Temblo Pre		15.96	6	0.13	2	0.015

independent condition are defined to optimize the TMDs.

• one TMD in the left, center and right spans.

• one TMD in the left and right spans and two TMDs in the center span.

• one TMD in the left and right spans and three TMDs in the center span.

Suspension bridges are famous to have more spaced modes and this matter causes difficulty to choose dominant mode for tuning the frequency of TMD. On the other hand, an earthquake spreads its energy along a relatively wide frequency range. By the way, due to fortuitous inherent of earthquake, predicting the dominant frequency is not possible. Hence, rather than to use tuning frequency parameter, an innovative parameter namely frequency ratio is defined to specify the optimum ratio of mode one. Also, mass ratio is defined as dividing of TMD mass to mass of each span, so two discrete mass ratio should be informed.

Here, minimizing of root mean square of displacement response of all vertical degrees of freedom is considered as the objective function.

$$F=\sqrt{\sum_{i=2}^{n} D_i^2} \quad .i=2.4.6.\ldots.100 \tag{14}$$

D is the vertical displacement of the suspended structure and sub-index i denotes the number of degree of freedom.

In this regard, twenty ground motions records divided to two various groups namely near and far-field ones individually including ten records are gathered and are provided in Table 1. The records cover wide range of peak ground accelerations and frequency content.

Two characteristics which are fault distance and velocity parameters are used to distinguish the near-field records from the far ones. By increasing the fault distance the PGV reduces which is the sign of the decreasing of the kinetic energy of recor

6. Numerical analysis

The Vincent Thomas, second Tacoma Narrows and Golden Gate suspension bridges placed in Los Angeles, Washington and San Francisco, respectively as a short, mean and long span bridges, are selected for case studies. All of them have three spans i.e. one central and two symmetric side spans, vertical hangers, steel pylons and externally-anchored type system. Also, the suspended structure of them compresses two-hinged stiffening girders truss type. The structural parameters of all bridges are summarized in Table 2 (Rubin *et al.* 1983).

In all bridges the side and center spans are divided to 11 and 28 elements, respectively. It is worth noting that the shear effect is neglected through the computation.

Table 3 provides the periods of bridges and Fig. 3 shows first two symmetric and antisymmetric mode shapes of them.

6.1 Optimized parameters

In this study, the mentioned set of ground motions are subjected to the suspended structure of bridges through the piers, and response of it is evaluated. It is worth noting that records are continuously imposed and also, optimizing is done considering the set and no individual records. Table 4 represents the outputs.

It is found during computation that by increasing the mass ratio the performance of TMD will be improved. But, too heavy TMDs can change the structural properties of

Table 2 structural and geometrical specification of bridges

Parameter		Vincent Thomas Bridge	Tacoma Narrows Bridge	Golden Gate Bridge
$L\ (m)$	S*	154.38	335.28	342.9
	C	457.2	853.44	1280.16
$W_d\ (\frac{kg}{m})$	S	5341.03	6443.75	17188.33
	C			17039.5
$E_g\ (\frac{N}{m})$	S C	200016662533.32	204154938309.87	200016662533.32
$I\ (m^4)$	S	0.3746	2.8482	1.6782
	C	0.3626		2.5893
$d\ (m)$	S C	4.572	10.0584	7.62
$E_c\ (\frac{N}{m})$	S C	186222409944.81	182773846797.69	200016662533.32
$A_c\ (cm^2)$	S C	783.8694	1625.80	5367.08
$L_e\ (m)$	S C	1054.61	1853.18	2346.35
$H_w\ (kN)$	S C	30035.8125	67435.96125	237914.78325

*S: side span; C: center span

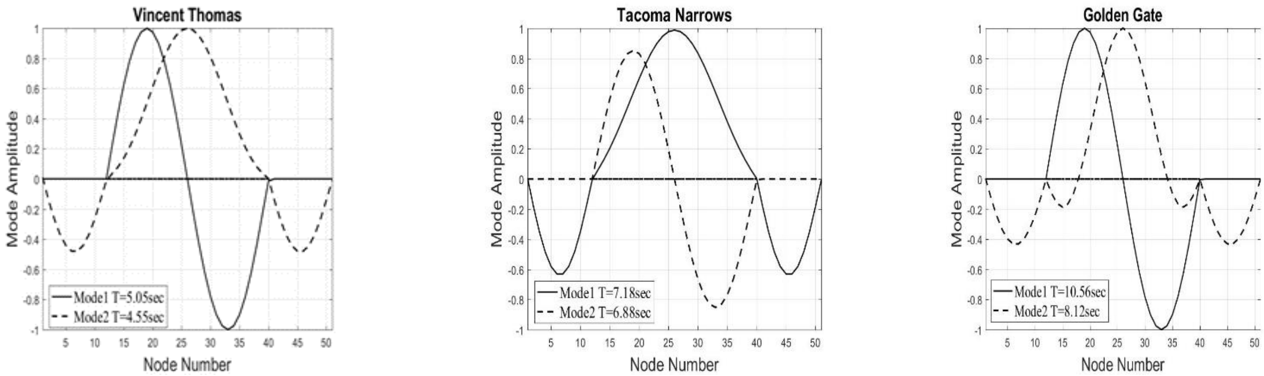


Fig. 3 Mode shapes of bridges

Table 3 periods of bridges (second)

Mode No.	1	2	3	4	5	6	7	8
Vincent Thomas	5.07	4.55	2.93	2.87	2.16	1.81	1.24	0.93
Tacoma Narrows	7.18	6.88	4.94	4	3.13	2.46	1.68	1.63
Golden Gate	10.56	8.12	6.45	5.51	4.92	3.93	3.49	2.97

bridges like natural frequencies and this is not recommended. So certain limitation bound seems to be necessary and here, mentioned bound is determined according to each span and are compared with whole structure. In all bridges, mass ratio is taken the most allowable value.

It is recognizable from table that frequency ratio for center span of short bridges is less than corresponding values to the mean and long bridges, and for side spans any difference is not seen. Damping ratio like to mass ratio adopts the most value permitted. Just damping ratios of TMDs in the center span of short bridge are different numbers. At 3TMD case, placing a TMD at the middle point of each span is considered as the optimum condition.

The optimum location of TMDs for side spans in cases 4TMD and 5TMD are similar to case 3TMD. But for center span, it is recognizable that in case 4TMD by increasing the span's length two TMDs recede each other and come close to pylons. Adverse result is expressible for case 5TMD.

6.2 Controlled responses

In order to investigate the performance of each optimized case, the results of bridge's responses to the fourteen (seven out of ten near and far-field) ground motions records from the available set are summarized in Table 5 in appendix. Table 6 resulted from Table 5 provides the responses reduction of bridges. In addition to three different cases and length parameter due to different suspension bridges, three various conditions specified based on the proximity to the fault and velocity parameters. So, all the controlled responses of all bridges should be addressed under near-field, far-field, and combined records.

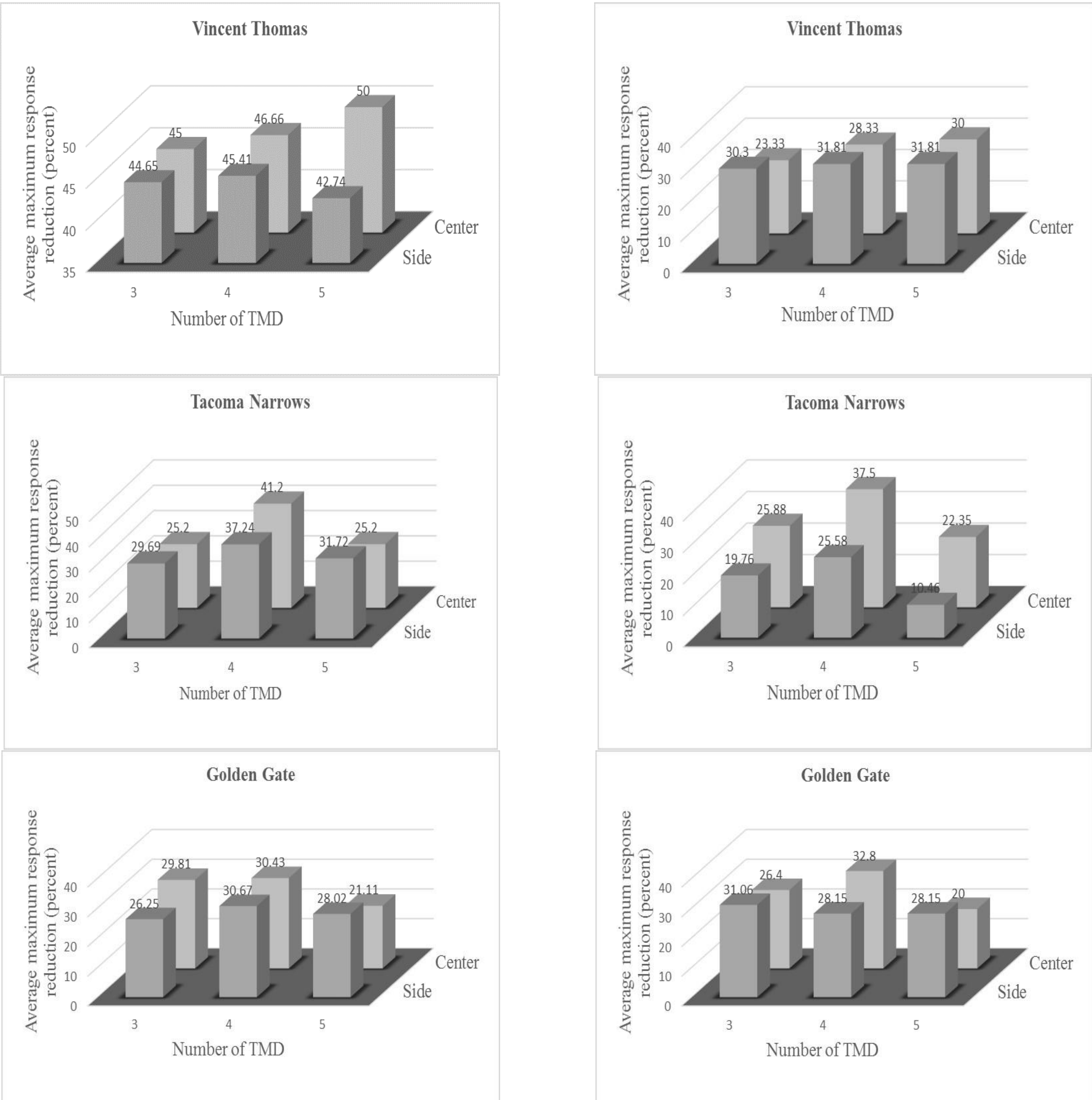
Fig. 4 shows the average of maximum respons reduction of each bridge in account to the number of used TMDs.

For the center span of short span bridges located in the

Table 4 optimized parameters

No.	Case	Bridge	Parameter									
			Mass ratio account to (%)				Frequency ratio		Damping ratio		Number of node	
			Side		Center							
			Itself	Total	Itself	Total	Side	Center	Side	Center	side	center
1	3TMD	V*	15	3	0.08	4.7	1.5	0.88	0.3	0.13	5,45	25
		T	15	3.3	0.08	4.4	1.5	1.5	0.3	0.3	5,45	25
		G	15	2.6	0.08	5.1	1.5	1.5	0.3	0.3	5,45	25
2	4TMD	V	15	3	0.08	4.7	1.5	0.7	0.3	0.15	5,45	27,27
		T	15	3.3	0.08	4.4	1.5	1.5	0.3	0.3	5,45	20,34
		G	15	2.6	0.08	5.1	1.5	1.5	0.3	0.3	5,45	17,37
3	5TMD	V	15	3	0.08	4.7	1.5	0.73	0.3	0.14	5,45	21,27,33
		T	15	3.3	0.08	4.4	1.5	1.5	0.3	0.3	5,45	22,27,32
		G	15	2.6	0.08	5.1	1.5	1.5	0.3	0.3	5,45	23,27,31

*V: Vincent Thomas; T: Tacoma Narrows; G: Golden Gate



(a) near-field (b) far-field

Fig. 4 Average maximum response reduction of bridges

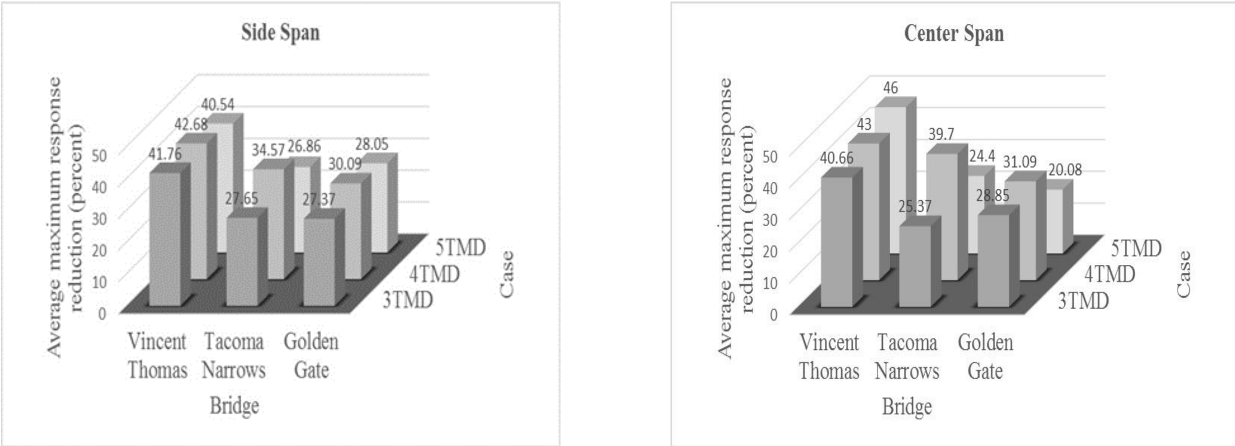
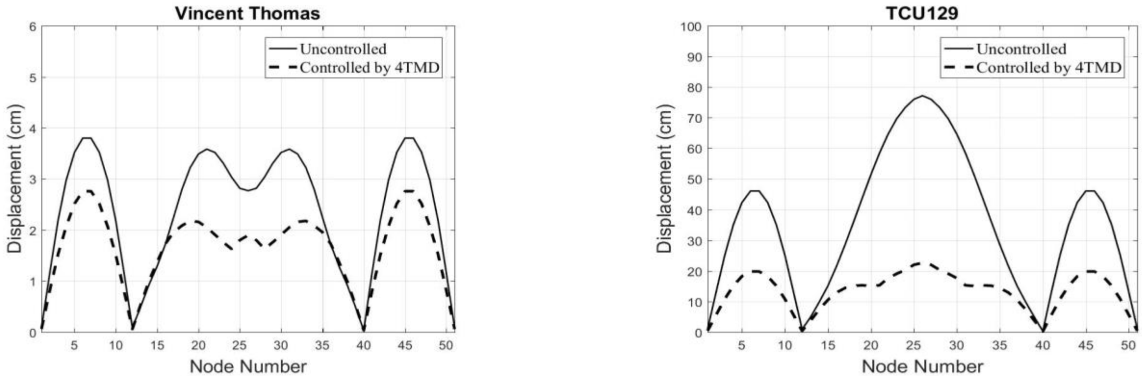
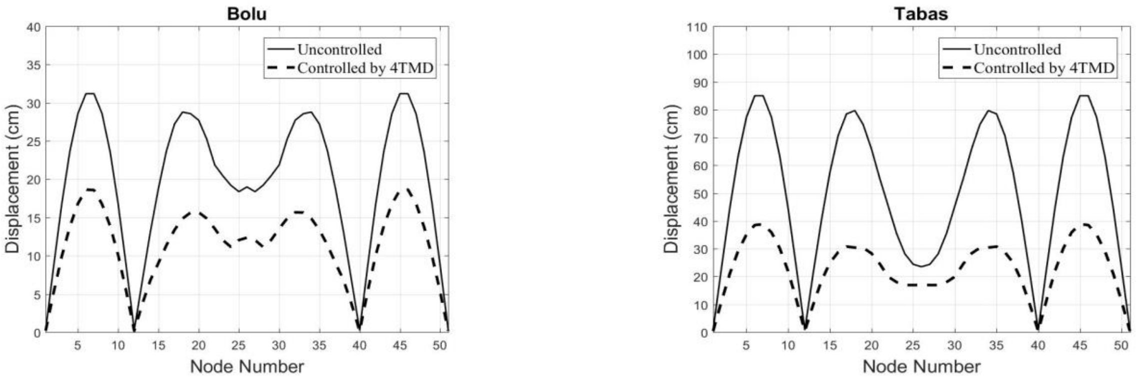


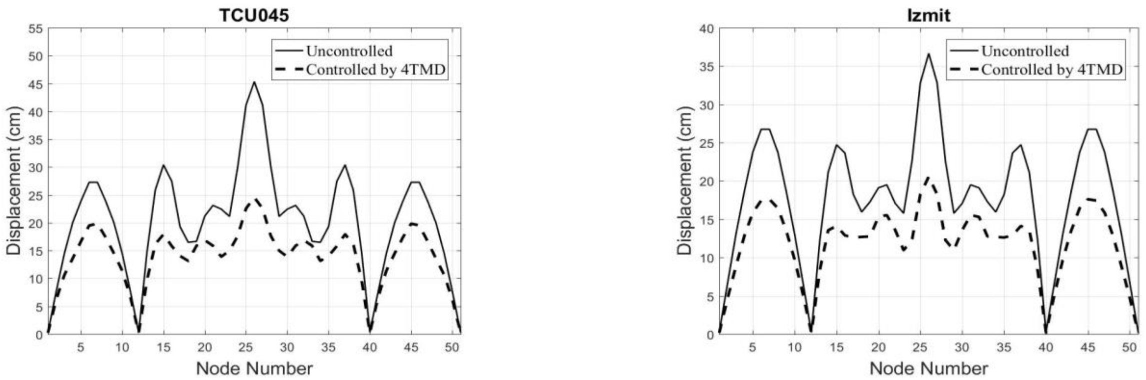
Fig. 5 performance of TMD on average of maximum response reduction



(a) Vincent Thomas



(b) Tacoma Narrows



(c) Golden Gate

Fig. 6 Controlled and uncontrolled responses of bridges

Table 5 response of bridges

parameter	Case	Bridge		Ground motions record													
				Near-Field							Far-Field						
				Bam	Rinaldi	Lucern	Tabas	TCU129	Erzincan	Izmit	TCU045	Bolu	Kakogava	OKY004	coolwater	Fatih	Brienza
Maximum displacement response (cm)	Uncontrolled	V	s	31	34	40	68	46	25	18	19	15	8	5	9	5	4
Maximum displacement response (cm)			c	18	30	27	47	77	27	14	16	13	6	5	7	9	4
Maximum displacement response (cm)		T	s	25	28	46	85	47	31	28	22	31	7	5	8	9	4
Maximum displacement response (cm)			c	21	22	40	80	44	26	26	18	29	5	5	6	9	4
Maximum displacement response (cm)		G	s	24	41	48	79	74	46	27	27	31	8	6	11	16	4
Maximum displacement response (cm)			c	20	24	42	85	66	48	37	45	43	7	4	8	15	3
Maximum displacement response (cm)	3TMD	V	s	22	21	29	31	17	14	11	15	9	6	4	6	3	3
Maximum displacement response (cm)			c	18	16	23	29	21	15	10	12	12	5	3	6	5	3
Maximum displacement response (cm)		T	s	19	20	41	55	34	20	14	20	23	6	3	7	7	3
Maximum displacement response (cm)			c	20	20	27	50	35	21	14	19	18	5	4	6	8	3
Maximum displacement response (cm)		G	s	20	29	34	67	48	33	19	22	20	5	6	6	10	2
Maximum displacement response (cm)			c	17	23	42	55	46	24	19	29	28	7	4	10	11	3
Maximum displacement response (cm)	4TMD	V	s	22	20	29	31	17	14	10	16	9	6	2	6	3	3
Maximum displacement response (cm)			c	18	17	22	27	21	14	9	11	13	5	2	5	5	2
Maximum displacement response (cm)		T	s	18	20	37	39	34	19	15	18	19	7	3	7	8	2
Maximum displacement response (cm)			c	17	18	27	30	31	13	11	19	16	4	2	6	6	2
Maximum displacement response (cm)		G	s	19	28	37	60	44	30	17	19	22	7	5	8	11	3
Maximum displacement response (cm)			c	15	19	48	48	48	26	20	25	28	6	3	8	12	2
Maximum displacement response (cm)	5TMD	V	s	21	22	31	29	21	16	10	15	9	5	3	6	4	3
Maximum displacement response (cm)			c	16	15	23	23	22	13	8	11	12	5	2	5	5	2
Maximum displacement response (cm)		T	s	18	20	45	44	37	18	16	23	25	6	4	8	9	2
Maximum displacement response (cm)			c	18	18	30	43	45	18	15	23	16	5	4	6	9	3
Maximum displacement response (cm)		G	s	23	25	37	67	44	30	18	25	21	5	5	5	10	3
Maximum displacement response (cm)			c	18	23	55	54	58	23	23	37	29	6	4	9	13	2
Maximum response reduction (%)	3TMD	V	s	29	38	27	54	63	44	39	21	40	25	20	33	40	40
Maximum response reduction (%)			c	0	47	15	38	73	44	29	25	8	17	40	14	44	25
Maximum response reduction (%)		T	s	24	29	11	35	28	35	50	9	26	14	40	12	22	25
Maximum response reduction (%)			c	5	9	32	37	20	19	46	-6	38	0	20	0	11	25
Maximum response reduction (%)		G	s	17	29	29	15	35	28	30	19	35	37	0	45	37	50
Maximum response reduction (%)			c	15	4	0	35	30	50	49	36	35	0	0	-25	27	0
Maximum response reduction (%)	4TMD	V	s	29	41	27	54	63	44	44	16	40	25	60	33	40	40
Maximum response reduction (%)			c	0	43	19	43	73	48	36	31	0	17	60	29	44	50
Maximum response reduction (%)		T	s	28	29	20	54	28	39	46	18	39	0	40	12	11	50
Maximum response reduction (%)			c	19	18	32	62	30	50	58	-6	45	20	60	0	33	50
Maximum response reduction (%)		G	s	21	32	22	24	41	35	37	33	29	12	17	27	31	25
Maximum response reduction (%)			c	25	21	-14	44	27	46	46	44	35	14	25	0	20	33
Maximum response reduction (%)	5TMD	V	s	32	35	22	57	54	36	44	21	40	37	40	33	20	40
Maximum response reduction (%)			c	11	50	15	51	71	52	43	31	8	17	60	29	44	50
Maximum response reduction (%)		T	s	28	29	2	48	21	42	43	-5	19	14	20	0	0	50
Maximum response reduction (%)			c	14	18	25	46	-2	31	42	-28	45	0	20	0	0	25
Maximum response reduction (%)		G	s	4	39	22	15	41	35	33	7	32	37	16	54	37	25
Maximum response reduction (%)			c	10	4	-31	36	12	52	38	18	33	14	0	-12	13	33

near-field region, by increasing the number of used TMDs average response reduction is increased. But about side spans, increasing of TMDs provide the reverse result, and case 4TMD is the optimum solution. Also, about far-field region, by increasing of the number of TMDs more average response reduction is occurred. About mean suspension bridges by increasing the number of TMDs, average response reduction is firstly raised and then is decreased such that case 4TMD is the best choice. Mentioned expression is acceptable for both side and center spans placed whether in the near-field or far-field region. For the side span of long span suspension bridges located in the near-field region similar result of mean span bridges is authentic. For the center span mentioned result is credible with a few connivances. As seen case 4TMD provides the most appropriate average response reduction for all bridges in near-field, far-field, and combined condition.

Fig. 5 demonstrates the affectivity and performance of three, four and five TMDs on all bridges for both side and center spans in near-field, far-field and combined conditions.

About bridges containing both near and far-field conditions, for both side and center span of bridges with any length, case 4TMD results in the average maximum response reduction. Also, by increasing length of the bridge average maximum response reduction decreases indicating the decrement of the efficiency of TMD.

Table 6 maximum response reduction percent

No.	Type	Bridge		Condition		
				3TMD	4TMD	5TMD
1	Combined records	Vincent Thomas	s	41.76	42.68	40.54
			c	40.66	43	46
		Tacoma Narrows	s	27.65	34.57	26.86
			c	25.37	39.70	24.4
		Golden Gate	s	27.37	30.09	28.05
			c	28.85	31.09	20.08
2	Near-Field records	Vincent Thomas	s	44.65	45.41	42.74
			c	45	46.66	50
		Tacoma Narrows	s	29.69	37.24	31.72
			c	25.2	41.2	25.2
		Golden Gate	s	26.25	30.67	28.02
			c	29.81	30.43	21.11
3	Far-Field records	Vincent Thomas	s	30.3	31.81	31.81
			c	23.33	28.33	30
		Tacoma Narrows	s	19.76	25.58	10.46
			c	25.88	37.5	22.35
		Golden Gate	s	31.06	28.15	28.15
			c	26.4	32.8	20

Fig. 6 shows the response of the Vincent Thomas, Tacoma Narrows and Golden Gate suspension bridges controlled by suitable cases stated in the former parts.

As seen in Fig. 6, case 4TMD remarkably reduce the ultimate response of bridges which is more uniform in short and mean span bridges in compare to long ones.

7. Conclusions

In this study, the performance of TMD device for controlling of vertical vibration of suspension bridges was optimized under set of near and far-field ground motions. In this regard, OTLBO algorithm was chosen as the meta-heuristic optimization algorithm. Also, the Vincent Thomas, Tacoma Narrows, and Golden gate suspension bridges were selected as the short, mean, and long span bridges, respectively, for case studies. After fulfilling the numerical analysis, the most important results are listed as follows:

• All cases of used TMDs resulted in response reduction and placing one TMD at the middle point of side spans is essential and is independent from the number of used TMDs. When case 4TMD is used as the control strategy, by increasing the length of bridge, optimum host nodes of two TMDs located at the center span recede each other and accede to the towers. But when case 5TMD is used, while one of TMDs is placed at the middle point of center span, other two optimum host nodes accede each other by increasing the length.

• In suspension bridges with any length, the performance of TMD will improve by increasing the mass ratio, and the upper bound which does not disturb the efficiency of TMD should be chosen as the optimum value. By decreasing the length of bridges more close modes will reduce, and optimum frequency ratio of center span of short suspension bridges takes lower value than corresponded values of mean and long ones. Similar result is expressible about the damping ratio.

• Considering the average response reduction as the basis of decision case 4TMD can provide the best performance in the response reduction for the bridges with any length of span placed in the near-field, far-field, and combined region.

References

Abdel-Ghaffar, A.M. (1980), “Vertical vibration analysis of suspension bridges”, *J. Struct. Div.*, **106**(10), 2053-2075.

Abdel-Ghaffar, A.M. and Rubin, L.I. (1983), “Vertical seismic behavior of suspension bridge”, *Earthq. Eng. Struct. Dyn.*, **11**(1), 1-19. https://doi.org/10.1002/eqe.4290110103.

Alizadeh, H., Lavasani, H.H. and Pourzeynali, S. (2018), “Flutter instability control in suspension bridge by TMD”, *The Proceedings of the 11th International Congress on Civil Engineering*., Tehran, Iran, May.

Amini, F. and Doroudi, R. (2010), “Control of a building complex with Magneto-Rheological Dampers and Tuned Mass Damper”, *Struct. Eng. Mech.*, **36**(2), 181-195. https://doi.org/10.12989/sem.2010.36.2.181.

Casciati, F. and Giuliano, F. (2009), “Performance of Multi-TMD in the towers of suspension bridges”, *J. Vib. Control.*, **15**(6), 821-847. https://doi.org/10.1177/1077546308091455.

Chen, S.R. and Wu, J. (2008), “Performance enhancement of bridge infrastructure systems: Long-span bridge, moving trucks and wind with tuned mass dampers”, *Eng. Struct.*, **30**(11), 3316-3324. https://doi.org/10.1016/j.engstruct.2008.04.035.

Chen, Y.H. and Huang, Y.H. (2004), “Timoshenko beam with tuned mass dampers and its design curves”, *J. Sound Vib.*, **278**(4-5), 873-888. https://doi.org/10.1016/j.jsv.2003.10.013.

Colliera, C.J. and Elnashai, A.S. (2010), “A procedure for combining vertical and horizontal seismic action effects”, *Earthq. Eng.*, **5**(4), 521-539. https://doi.org/10.1080/13632460109350404.

Debbarma, R. and Das, D. (2016), “Vibration control of building using multiple tuned mass dampers considering real earthquake time history", *Int. J. Civil Environ. Eng*., **10**(6), 694-704. https://doi.org/10.5281/zenodo.1124591.

Elias, S. and Matsagar, V. (2017), "Research developments in vibration control of structures using passive tuned mass dampers", *Annu. Rev. Control*, **44**, 1-28. https://doi.org/10.1016/j.arcontrol.2017.09.015.

Elnashai, A.S. and Papazoglou, A.J. (2007), "Procedure and spectra for analysis of RC structures subjected to strong vertical earthquake load", *Earthq. Eng*., **1**(1), 121-127. https://doi.org/10.1080/13632469708962364.

Grimaz, S. and MaliSan, P. (2014), "Near field domain effects and their consideration in the international and Italian seismic codes", *Bollettino di Geofsica Teorica ed Applicata*, **55**(4), 717-738. https://doi.org/10.4430/bgta0130.

Hosseini Lavassani, S.H., Alizadeh, H. and Homami, P. (2020). "Optimizing tuned mass damper parameters to mitigate the torsional vibration of a suspension bridge under pulse-type ground motion: A sensitivity analysis", *J. Vib. Control*, **26**(11-12), 1054-1067. https://doi.org/10.1177/1077546319891591.

Li, Q., Fan, J., Nie, J., Li, Q. and Chen, Y. (2010), "Crowd-induced random vibration of footbridge and vibration control using multiple tuned mass dampers", *J. Sound Vib*., **329**(19), 4068-4092. https://doi.org/10.1016/j.jsv.2010.04.013.

Maniatakis, C.H.A., Taflampas, I.M. and Spyrakos, C.C. (2008), "Identification of near-fault earthquake record characteristics", *In the Proceedings of the14th World Conference on Earthquake Engineering*, Beijing, China, October.

Memarpour, M.M., Ghodrati, G.A., Razeghi, H., Akbarzadeh, M. and Tajik, A.D. (2016), "Characteristics of horizontal and vertical near-field ground motions and investigation of their effects on the dynamic response of bridges", *J. Rehab. Civil Eng*., **4**(2), 1-24. https://10.22075/jrce.2016.498.

Miguel, L.F.F., Lopez, R.H., Miguel, L.F.F. and Torii, A.J. (2016), "A novel approach to the optimum design of MTMDs under seismic excitations", *Struct. Cont. Health Monit*., **23**(16), 1290-1313. https://doi.org/10.1002/stc.1845.

Miguel, L.F.F., Lopez, R.H., Torii, A.J., Miguel, L.F.F. and Beck, A.T. (2016), "Robust design optimization of TMDs in vehicle-bridge coupled vibration problems", *Eng. Struct*., **126**, 703-711. https://doi.org/10.1155/2019/9273714.

Pisal, A.Y. and Jangid, R.S. (2016), "Vibration control of bridge subjected to multiaxle vehicle using multiple tuned mass friction dampers", *Int. J. Advan. Struct. Eng*, **8**(2), 1-15. https://doi.org/10.1007/s40091-016-0124-y.

Pourzeynali, S. and Datta, T.K. (2002a), "Control of flutter of suspension bridge deck using TMD", *Wind Struct*., **5**(5), 407-422. https://doi.org/10.12989/was.2002.5.5.407.

Pourzeynali, S. and Datta, T.K. (2002b), "Reliability analysis of suspension bridges against flutter", *J. Sound Vib*., **254**(1), 143-162. https://doi.org/10.1006/jsvi.2002.4090.

Pourzeynali, S. and Esteki, S. (2009), "Optimization of the TMD parameters to suppress the vertical vibration of the suspension bridges subjected to earthquake excitation", *IJE Transactions B: Applic*., **22**(1), 23-34.

Pourzeynali, S., Lavasani, H.H. and Modarayi, A.H., (2007), "Active control of high rise building structures using fuzzy logic and genetic algorithms", *Eng. Struct*., **29**(3), 346-357. https://doi.org/10.1016/j.engstruct.2006.04.015.

Rao, R.V., Savsani, V.J. and Vakharia, D.P. (2011), "Teaching-learning-based optimization: A novel method for constrained mechanical design optimization problems", *Comput. Aided Des*., **43**(3), 303-315. https://doi.org/10.1016/j.cad.2010.12.015.

Rubin, L.I., Abdel-Ghaffar, A.M. and Scanlan, R.H. (1983), Earthquake response of long-span suspension bridges", Report No. 83-SM-13; Department of Civil Engineering, Princeton University, Princeton, U.S.A.

Shahrouzi, M., Aghabagloua, M. and Rafiee, F. (2017), "Observer-teacher-learner-based optimization: An enhanced metaheuristic for structural sizing design", Struct. Eng. Mech., **62**(5), 537-550. https://doi.org/10.12989/sem.2017.62.5.537.

Tao, T., Wang, H., Yao, C. and He, X. (2017), "Parametric sensitivity analysis on the buffeting control of a long-span triple-tower suspension bridge with MTMD", *Appl. Sci*., **7**(4), 395. https://doi.org/10.3390/app7040395.

Tubino, F., Carassale, L. and Piccardo, G. (2016), "Human-induced vibrations on two lively footbridges in Milan", *J. Bridge Eng*., **21**(8) Article Number C4015002. https://doi.org/10.1061/(ASCE)BE.1943-5592.0000816.

Ubertini, F., Comanducci, G. and Laflamme, S. (2015), "A parametric study on reliability-based tuned-mass damper design against bridge flutter", *J. Vib. Control*., **23**(9), 1518-1534. https://doi.org/10.1177/1077546315595304.

Vallada, E. and Ruiz, R. (2011), "Genetic algorithms with path re linking for the minimum tardiness permutation flow shop problem", *Omega*, **38**, 57-67. https://doi.org/10.1016/j.omega.2009.04.002.

Wang, J.F., Lin, C.C. and Chen, B.L. (2003), "Vibration suppression for high-speed railway bridges using tuned mass dampers", *Int. J. Solids Struct*., **40**(2), 465-491. https://doi.org/10.1016/S0020-7683(02)00589-9.

Wong, K.K.F. and Chee, Y.L. (2004), "Energy dissipation of tuned mass dampers during earthquake excitations", *Struct. Des. Tall Spec. Build*., **13**(2), 105-121. https://doi.org/10.1002/tal.244.

Yau, J.D. and Yang, Y.B. (2004), "A wideband MTMD system for reducing the dynamic response of continuous truss bridges to moving train loads", *Eng. Struct*., **26**(12), 1795-1807. https://doi.org/10.1016/j.engstruct.2004.06.015.

AT