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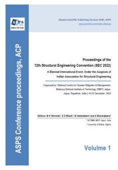

Seismic Response Control of Parallel Structures Connectd by Passive Shape Memory Alloy Damper

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Abstract

A shape Memory Alloys are a class of novel functional materials that possess unique properties, including shape memory effect, super-elasticity effect, high damping characteristics, high corrosion resistance, temperature dependent Young modulus and extra ordinary fatigue resistance. In super-elastic phase, shape memory alloys are initially austenitic, upon loading, stress-induced martensite is formed, upon unloading, the martensite reverts to austenite at a lower stress level, resulting in the hysteretic behaviour, is more suitable property for energy dissipation device in structural engineering. In this paper, the response behaviour of two parallel structures coupled by passive Shape Memory Alloy dampers under various earthquake ground motion excitations is investigated. The equation of motion for the two parallel, single-degree-of-freedom structures connected by damper is formulated. The effectiveness of damper in terms of the structural response reduction namely, relative displacement and absolute acceleration of coupled structure is investigated. A parametric study is conducted to investigate the optimum parameter of the dampers. Results show that Shape Memory Alloy dampers connecting the parallel structures of different fundamental frequencies, the earthquake induced displacement response of either structure can be reduce effectively, but acceleration response of flexible structure is increases. Thus passive shape memory alloy damper is not much effective for seismic protection of coupled structure concept.

Keywords: Connected structures, Parallel structures, Passive control, Seismic response, Shape memory alloy damper

1. Introduction

The conventional design approach for seismic protection is to design structure with sufficient strength to withstand natural forces, an adequate ductility to absorb excessive energy during natural disturbances and appropriate stiffness to maintain structural integrity and serviceability. It provides the minimum level of protection to the structure. Another alternate is implementing the energy dissipation devices and control system into the structures to reduce the excessive structural vibrations due to natural disturbances. These devices increase the safety and reliability of engineering structures against natural loads. The control strategies are able to modify dynamically the response of the structure in a desirable manner, thereby termed as protective systems for the new structures. The control system also used to strengthen or retrofits the existing structure against future natural disturbances. The energy dissipation devices dissipate the additional energy added to the structure during earthquake. The additional energy is dissipated either by transforming into heat or transforming it to any connected structures or mass dampers, prevent catastrophic structural failure.

Interactions between existing neighbouring inadequately separated structures are frequently recurrent problems during the strong earthquake. Due to different dynamic properties, adjacent structures can vibrate out of phase during earthquake excitations and crates relative displacement problems. The collapse can occur when the common vertical support share by structures and the distance between structures increases, for example falling of bridge deck from supports. When distance between vibrating structures decreases, pounding of the structures may occur. The restrainers provided to limit the separation distance between adjacent structures, are subjected to severe pulling which may result in local failure and/or undesirable inertia forces transfers from one segment to the other one of the bridge [1]. Connecting the parallel structures is one of the alternate techniques for seismic protection of the structures. The basic concept is to exert control forces upon one another of dynamically dissimilar coupled structure to reduce the overall response of the system. The available free space between parallels structures can be effectively utilize (if possible), thus, additional space does not require for installation of damping devices. It alters the dynamic characteristics of the uncoupled structures and in case of asymmetric geometry, torsional response and base shear of stiffer structure are increases. The coupling parallel structures also helpful to overcome the problem of the pounding, a more dangerous condition during the

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earthquake, observed during past earthquake events such as 1985 Maxico City earthquake [2], 1989 Loma Prieta earthquake and many others. Matsagar and Jangid [3] presented the seismic response analysis of base-isolated adjacent structures connected by viscous and viscoelastic dampers. The analysis results show that for the existing under-designed fixed base structure, the base isolation of both or one of the adjacent building found advantageous in the retrofitting works. Bhaskararao and Jangid [4] presented the optimum viscous damper for connecting adjacent single-degree-of-freedom (SDOF) structures for harmonic and stationary white-noise random excitations. Patel and Jangid [5] analyzed the adjacent multi-degree-of-freedom (MDOF) structures connected with Maxwell dampers; results show that using Maxwell dampers of appropriate parameter to connect the adjacent structures of different fundamental frequencies can effectively reduce earthquake-induced responses of either structure. The lesser dampers of proper parameter and at appropriate location can significantly reduce the earthquake response of connected system. Patel and Jangid [6] presented inter-building damper coupling approach for the dynamically similar adjacent structures with three different configurations of inter-connecting damper distributions. The main purpose is how to differently amplify the relative response of the two ends of the damper. Abdeddaim et al. presented the pounding hazard reducation using a coupling strategy for adjacent buildings [7]. Abdeddaim et al. also presented the retrofitting of a weaker building by coupling it ot an adjacent stronger building using MR dampers [8]. Patel [9] presented the response behaviour of two parallel single-degree-of-freedom (SDOF) structures connected with Maxwell damper subjected to non-stationary as well as stationary white-noise random excitation. The influence of parameters such as relaxation time of damper, frequency and mass ratio of connected structures on the performance of damper have presented. The closed-form expression for optimum damper damping for the minimum value of mean square relative displacement and absolute acceleration of either of coupled structures also presented.

2. Shape Memory Alloy Damper

Shape memory alloy (SMA) possess unique properties like super-elasticity, shape memory effect, high corrosion resistance, high fatigue resistance, temperature-dependent Young's modulus and high damping characteristics. SMAs application mainly in mechanical, aerospace, electrical, electronics engineering and medical science is well known. As far as energy dissipation device application for structural response control in Civil and Earthquake engineering ins concern, the high damping characteristics is prime important requirement. SMA exhibits shape memory effect at low temperature and by heating the material above the austenite finish temperature, the residual deformation can be recover effectively. SMA exhibits super-elasticity effect at high temperature. The SMA at higher temperature in super-elastic phase are initially in austenitic. The stress-induced martensite is formed when material is subjected to loading. Upon unloading at a lower stress level the martensite return to austenite. This loading-unloading cycle results in hysteretic behaviour which is the suitable characteristics for energy dissipation device application for seismic response control of the structures. The shape memory effect and pseudo elasticity of SMAs associated with stress-induced or thermo-induced reversible hysteretic phase transformation between austenite and martensite are the two main properties make the SMAs more suitable for application as passive energy dissipaters and actuators for structural control. Thus, SMA based devices are used for active, semi-active and passive control of civil structures. Graesser and Cozzarelli [10] presented the seismic isolation using SMA. A state-of-the–art review of SMA based passive control devices has been presented by Dolce et al. [11]. A study on the high damping capacity of shape memory alloys by Humbeeck and Liu [12] reported that the significant part of mechanical energy due to either by vibration or shock waves or impact loading converts into heat by passive damping. It also reported that damping capacity is nearly independent of frequency of applied vibration and is increases with increasing in amplitude of vibration. SMA devices has been found its applications in the form of connection elements for columns, in the forms of damper for simply supported and cable-stayed bridges and braces for framed structures. A review of application of the SMA materials for active, semi-active and passive control of civil structures is presented by Song et al. [13].

Parulekar et al. [14] presented the shape-memory alloy damper device made up of austenite wires. The damper device was designed, fabricated and tested. The performance of the structure with SMA dampers compared with that of the same structure with yielding dampers. Qian et al. [15] tested the recentering shape memory damper for tensile-compressive test with various pre-strain under different displacement amplitudes and loading frequencies. The damper performance found nearly insensitive for excitation frequency more than 0.5 Hz.

The above studies confirm that the SMA damper is effective for seismic protection of the structures. The present study is aim to investigate the performance of the two parallel SDOF structures connected with SMA damper subjected to base excitations. The specific objectives of the present study are (i) to formulate the equations of the motion for the parallel structures connected with SMA damper; (ii) to investigate the optimum parameter of SMA damper; and (iii) to evaluate the performance of the SMA damper connecting parallel structure under various real earthquake ground motion excitations.

3. Structural Model

Let us consider two parallel SDOF structures connected with the SMA damper as shown in Fig. 1, referred as Structure 1 and 2, respectively. The connected structures are symmetric with their symmetric planes in the alignment, ground motion is to occur in the one direction in the symmetric planes and both structures subjected to the same ground excitation. The considered parallel structures connected with the energy dissipation device, the two structures are assumed to be remain linearly elastic. The damper is connecting two parallel structure at their floor level. The effect of soil-structure interaction is been neglected.

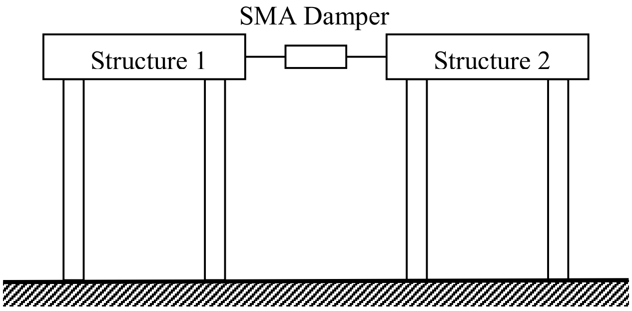

Fig. 1 Structural model of two SDOF parallel structures connected with SMA damper

Let m_1, c_1, k_1 and m_2, c_2, k_2 be the mass damping coefficient and stiffness of the Structure 1 and 2, respectively. Let $\omega_1 = \sqrt{k_1/m_1}$ and $\omega_2 = \sqrt{k_2/m_2}$ be the natural circular frequencies and $\xi_1 = c_1 / 2m_1\omega_1$ and $\xi_2 = c_2 / 2m_2\omega_2$ be the damping ratio of Structure 1 and 2, respectively. Let us consider two parameters frequency ratio (β) and mass ratio (η) of the connected structure define as

$$\beta = \frac{\omega_2}{\omega_1} \qquad (1)$$

$$\eta = \frac{m_1}{m_2} \qquad (2)$$

To represent the different behaviour of SMA materials, researchers have utilized different theories and approaches, varying from theoretical model using micromechanics and/or thermodynamics formulation to experimental based model. The governing equation of motion for the damper-connected system can be written as

$$m_1\ddot{y}_1 + c_1\dot{y}_1 + k_1 y_1 + f_d = -m_1\ddot{y}_g \qquad (3)$$

$$m_2\ddot{y}_2 + c_2\dot{y}_2 + k_2 y_2 - f_d = -m_2\ddot{y}_g \qquad (4)$$

The over dot represents time derivative; $\ddot{y}_g$ is the ground acceleration; and f_d represent the SMA damper force. The structural control criteria depend on the nature of dynamic loads and the response quantities of interest. In case of stiff structure, acceleration is of more concern, generating higher inertia force in the structure, should be mitigated. Whereas, in case of flexible structure, displacement is predominant that needs to be controlled. Thus, minimizing relative displacement and/or absolute acceleration of the system has always considered as the control objective. In view of this, the study aims to evaluate the performance of SMA damper for minimizing exclusively displacement as well as acceleration response of the coupled structure.

4. Numerical Study

The performance of SMA damper on response of two parallel SDOF structures under four real earthquake ground motion excitations is been investigated. The earthquake ground motion namely Imperial Valley, 1940 with peak ground acceleration (PGA) 0.32 g; Loma Prieta, 1989 with PGA 0.57 g; Northridge 1994 with PGA 0.84 g and Kobe, 1995 with PGA 0.63 g, (g is acceleration due to gravity) are applied to a model of parallel connected structure with parameters $\xi_1 = \xi_2 = 0.05$, mass ratio $\eta = 1$ and frequency ratio $\beta = 2$ (i.e. $\omega_1 = \pi$ read/sec and $\omega_2 = 2\pi$ rad/sec). The coupled structures are dynamically well separated structures. To predict the cyclic behaviour of the super-elastic SMA device, the thermo-mechanical model proposed by Motahari and Ghassemieh represent the multi-linear stress strain ($\sigma - \varepsilon$) relationship, is used. On removal of the external load, the SMA recovers its initial shape is termed as pseudo-elastic effect. The damper force is been calculated from Pseudo-elastic hysteresis loop. On loading, the SMA wires undergoes the elastic loading of austenite phase till it reaches the yield stress (i.e. reaches the transformation level stress σ_{MS}) as shown in Figure 2. At yielding the transformation from austenite to martensite will be initiated and will continue till all the austenite transformed to martensite (reaches to σ_{MF}). Upon release of stress, reverse transformation take place and martensite unloads elastically up to it transform back to austenite (reaches σ_{AS}). The reverse transformation from martensite to austenite completed up to stresses reaches σ_{AF} , and the final austenite phase elastic unloading complete the loading cycle. Some experimental investigations shows that at the end of each load cycle there is residual martensite remains. For the present study the zero residual strain is considered. The critical stresses (σ_{MS} , σ_{MF} , σ_{AS} and σ_{AF}) and strains (ε_{MS} , ε_{MF} , ε_{AS} and ε_{AF}) considered are obtained from uniaxial test on SMA wire. The various properties of SMA damper wire considered are stress σ_{MS} = 551 MPa, σ_{MF} = 667 Mpa, σ_{AS} = 402 Mpa and σ_{AF} = 288.1 MPa; strain ε_{MS} = 0.013, ε_{MF} = 0.064, ε_{AS} = 0.054 and ε_{AF} = 0.0068, and Young's moduli during elastic austenite phase E_A =42308 MPa and during elastic martensite phase E_M =28571 MPa [14] and zero residue strain. The stress-strain relation on various path are given as

Elastic fully austenite (loading) path O-A and Elastic fully austenite(unloading) path E-O

$$\sigma = E_A \varepsilon \qquad (5)$$

Austenite to martensite transformation (forward transformation) path A-B

$$\sigma = \sigma_{MS} + \frac{\sigma_{MF} - \sigma_{MS}}{\varepsilon_{MF} - \varepsilon_{MS}} \left(\varepsilon - \varepsilon_{MS} \right) \qquad (6)$$

Elastic fully martensite (loading) path B-C and elastic fully martensite (unloading) path C-D

$$\sigma = \sigma_{MF} + E_M \left(\varepsilon - \varepsilon_{MF} \right) \qquad (7)$$

Martensite to austenite transformation (reverse transformation) path D-E

$$\sigma = \sigma_{AS} + \frac{\sigma_{AF} - \sigma_{AS}}{\varepsilon_{Af} - \varepsilon_{AS}} \left(\varepsilon - \varepsilon_{AS} \right) \qquad (8)$$

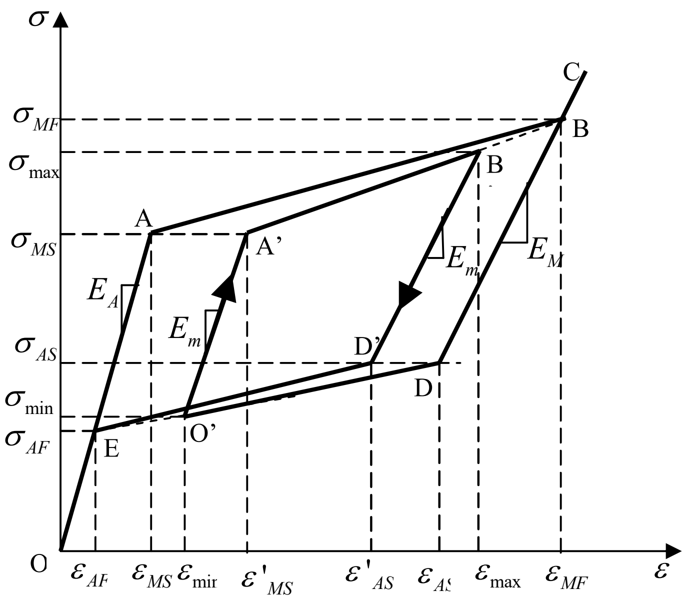

Fig.2 Stress-strain relationship diagram, pseudo-elasticity loop and sub-loop

The elastic stiffness would be different from austenite as well as martensite phases, if reloading occurs before the completion of the reverse transformation or unloading begins before the completion of the forward transformation. Considering L_m the compliance of mixture constituents, have the linear relationship with strain (Motahari and Ghassemieh)

$$L_m = xL_M + (1-x)L_A \tag{9}$$

where x is $\varepsilon_{\min} - \varepsilon_{AF} / \varepsilon_{AS} - \varepsilon_{AF}$ for reloading and x is $\varepsilon_{\max} - \varepsilon_{MS} / \varepsilon_{MF} - \varepsilon_{MS}$ for unloading. The $\varepsilon_{\min}$ is minimum strain before reloading and $\varepsilon_{\max}$ is maximum strain before unloading as shown in Figure 7. The elastic stiffness value can become

$$E_m = \frac{E_M E_A}{x(E_A - E_M) + E_M} \tag{10}$$

The new value for critical martansite strain is been obtained as

$$\varepsilon'_{MS} = \varepsilon_{\min} + \frac{\sigma_{MS} - \sigma_{\min}}{E_m} \tag{11}$$

and the new value for critical austenite strain is been obtained as

$$\varepsilon'_{AS} = \varepsilon_{\max} + \frac{\sigma_{AS} - \sigma_{\max}}{E_m} \tag{12}$$

Now for sub-loop path the stress-strain relationship is given as

Path O'-A'

$$\sigma = \sigma_{\min} + E_m(\varepsilon - \varepsilon_{\min}) \tag{13}$$

Path A'-B'

$$\sigma = \sigma_{MS} + \frac{\sigma_{MF} - \sigma_{MS}}{\varepsilon_{MF} - \varepsilon'_{MS}}(\varepsilon - \varepsilon'_{MS}) \tag{14}$$

Path B'-D'

$$\sigma = \sigma_{\max} + E_m(\varepsilon - \varepsilon_{\max}) \tag{15}$$

Path D'-E

$$\sigma = \sigma_{AS} + \frac{\sigma_{AF} - \sigma_{AS}}{\varepsilon_{AF} - \varepsilon_{AS}}(\varepsilon - \varepsilon'_{AS}) \tag{16}$$

The strain value in the SMA damper wire is been calculated from the relative deformation of damper ends. Using Eqs. 5 to 16, the corresponding stress value is been obtained. Finally, the damper force f_d is obtained by stress value and cross-sectional area of the SMA wire/s. The response of the system is obtained by solving the coupled Eq. (3) and (4) in the time domain using Newmarks step-by-step method.

The SMA damper device having 4 wires of 150 mm length is considered. The variation of displacement and acceleration response of both the structures against various wire diameter is shown in Fig. 3 and 4, respectively, for all four considered earthquakes. It is difficult to identify the optimum wire diameter for which peak value of all responses reduces to minimum. With increase in wire diameter value, the displacement response of Structure 1 is decreases, whereas, for Structure 2, it is increases.

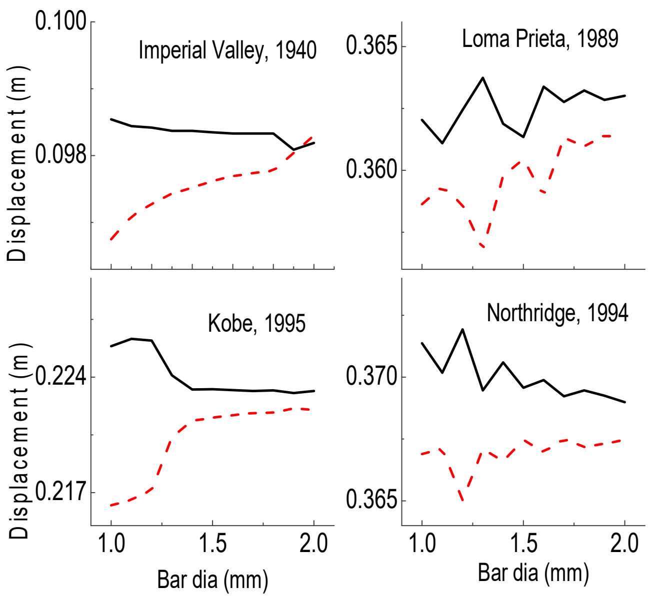

Fig.3 Variation of peak displacement response of parallel SDOF connected structures against wire diameter

Table 1. Seismic response of two parallel SDOF structures connected with SMA damper ($\beta = 2, \eta = 1, \xi_1 = \xi_2 = 0.05$)

Earthquake		Displacement		Acceleration	
		y_1 (m)	y_2 (m)	$\ddot{y}_{a1}$ (m/s^2)	$\ddot{y}_{a2}$ (m/s^2)
Imperial Valley, 1940	UC	0.1366	0.1123	1.3558	4.4711
	C	0.0954 (30.13)#	0.0725 (35.54)	2.3585 (-73.95)	2.2725 (49.17)
Kobe, 1995	UC	0.2506	0.3023	2.4952	11.9950
	C	0.2154 (14.07)	0.1763 (41.68)	5.5222 (-121.3)	5.6048 (53.27)
Loma Prieta, 1989	UC	0.6377	0.2601	6.3376	10.3323
	C	0.3568 (44.05)	0.3039 (-16.84)	9.4549 (-49.19)	9.3879 (9.14)
Northridge, 1994	UC	0.6120	0.2155	6.0697	8.5284
	C	0.3680 (39.88)	0.3631 (-38.50)	9.3379 (-53.84)	9.1642 (-7.45)

Quantity within parenthesis denotes percentage reduction with respect to unconnected system

UC= unconnected, C = Connected

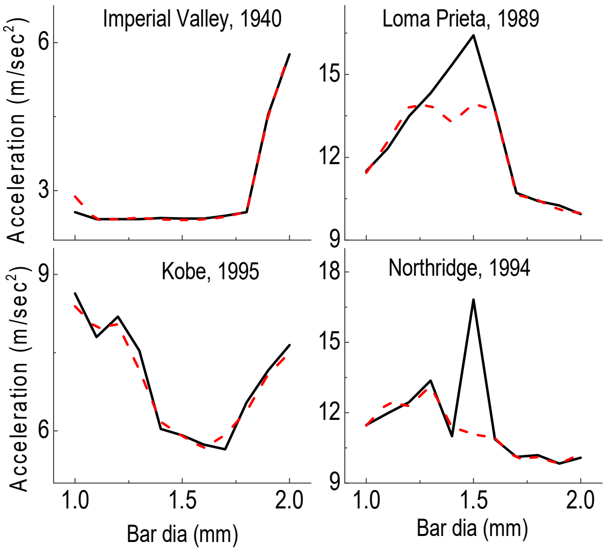

Fig.4 Variation of peak acceleration response of parallel SDOF connected structures against wire diameter

Considering SMA damper device having 3 wires of 1mm diameter and length 150 mm, the displacement response for Structure 1 and 2 for all four considered earthquake are shown in Fig. 5 and 6, respectively, and the acceleration response for Structure 1 and 2 are shown in Fig. 7 and 8, respectively. These figures show how effective the SMA damper is to mitigating the displacement and acceleration responses of connected structures. The peak responses of Structure 1 and 2 for unconnected and connected with SMA damper for all four considered earthquake excitations is as given in Table 1. It is observed that displacement response of Structure 1 is reduces for all four earthquakes.

The displacement response of Structure 2 is reduces for Imperial Valley 1940 and Kobe, 1995 earthquakes, whereas it increases for Loma Prieta, 1989 and Northridge, 1994 earthquakes. The acceleration response of Structure 1 is increases for all four earthquakes. The acceleration response of Structure 2 is reduces for Imperial Valley, 1940, Kobe, 1995 and Loma Prieta, 1989 earthquakes, whereas, it is increases for Northridge, 1994 earthquake.

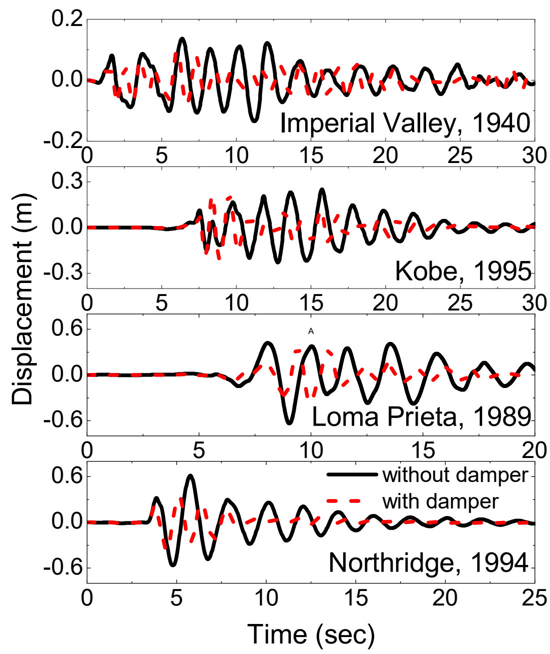

Fig.5 Time history for displacement response of Structure 1 ($\beta = 2, \eta = 1, \xi_1 = \xi_2 = 0.05$)

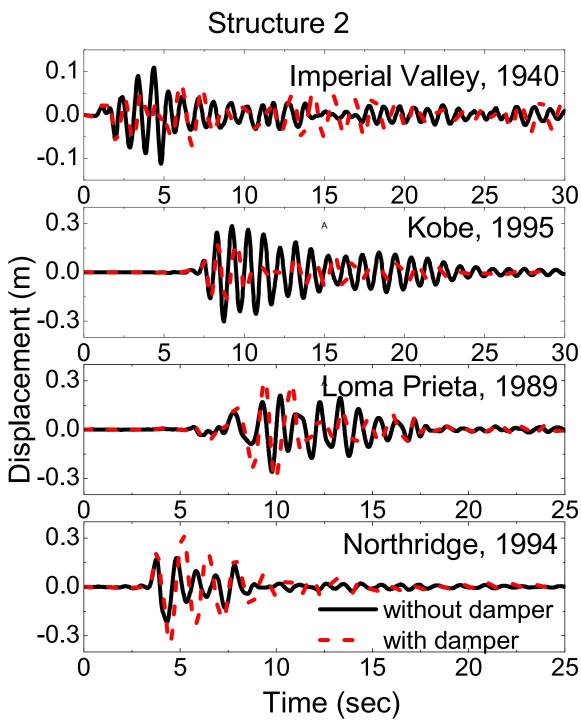

Fig.6 Time history for displacement response of Structure 2 ($\beta = 2, \eta = 1, \xi_1 = \xi_2 = 0.05$)

Thus, SMA damper is less effective for earthquake response control of parallel connected structure concept. The nature of earthquake ground motion excitation also affects the performance of the SMA damper. This concludes that the SMA damper is less effective for seismic protection of connected structure concept.

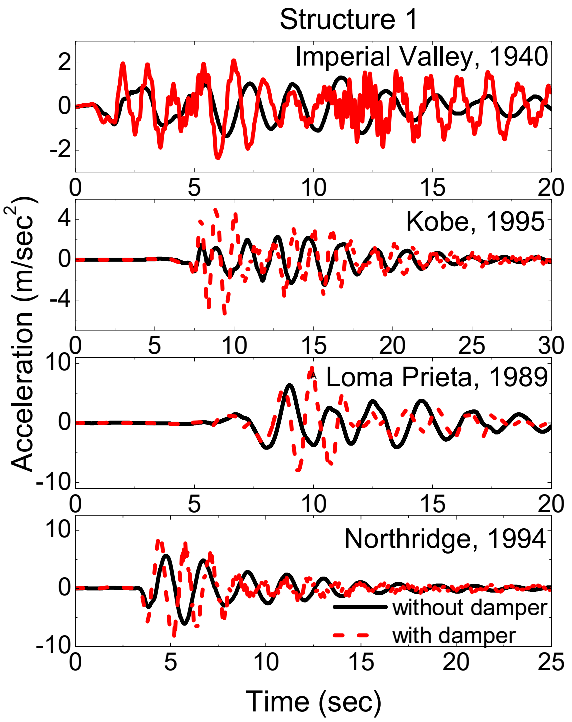

Fig.7 Time history for acceleration response of Structure 1 ($\beta = 2, \eta = 1,\ \xi_1 = \xi_2 = 0.05$)

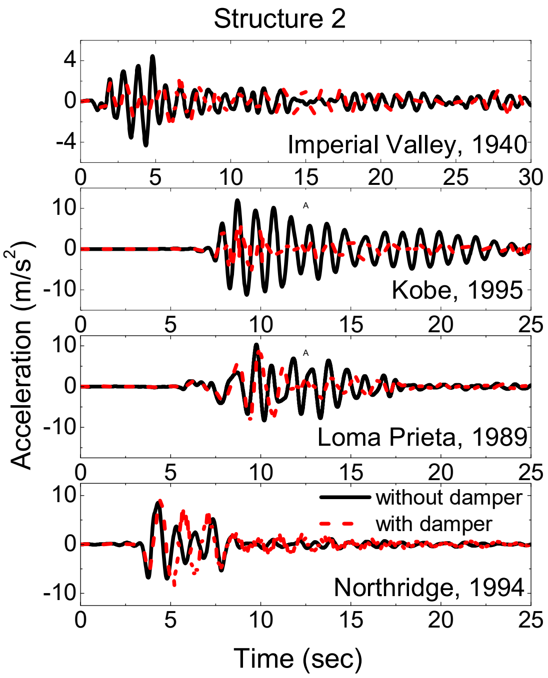

Fig.8 Time history for acceleration response of Structure 2 ($\beta = 2, \eta = 1,\ \xi_1 = \xi_2 = 0.05$)

5. Conclusions

The seismic response behaviour of two adjacent SDOF structures connected with SMA damper is investigated under various earthquake ground motion excitation considering a linear elastic behaviour of the structure. The governing equations of motion are formulated for the coupled structures. From the results of the study the conclusions drawn are summarized below.

1. The displacement response of flexible structure (Structure 1) is reduces for all considered earthquakes
2. The displacement response of stiff structure (Structure 2) is reduces for Imperial Valley, 1940 and Kobe, 1995 earthquakes, whereas, it increases for Loma Prieta, 1989 and Northridge, 1994 earthquakes.
3. Acceleration response of flexible structure (i.e. Structure 1) is increases for all earthquakes.
4. Acceleration response of stiff structure (i.e. Structure 2) is reduces for Imperial Valley, 1940; Kobe, 1995 and Loma Prieta, 1989 earthquake, whereas, increases for Northridge, 1994 earthquake.
5. The nature of ground motion excitation affects the performance of SMA damper.

Disclosures

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