

Seismic assessment of steel frames with triangular-plate added damping and stiffness devices



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ABSTRACT

The current paper tries to investigate the seismic behavior and to determine the global damage parameter (GDP) of the moment resisting frames equipped with triangular-plate added damping and stiffness (TADAS) devices. TADAS devices are the most popular metallic dampers in order to dissipate energy in structures and are used in most projects as a passive structural controlling system. Four frame types with 3, 6, 9 and 12-story and three bays are modeled and nonlinear analyses results such as target displacement, effective and elastic lateral stiffness and fundamental period, pushover curve and the response modification factor are estimated and compared for two case of moment resisting frames (MRFs) and moment resisting frames equipped with TADAS devices (TMRFs). The results showed that the response modification factors for TMRFs are higher than the MRFs ones and decrease by 40 percent gradually with an increase in the height of the frames. It was also found that influence of TADAS devices on the increase of the global damage parameter are not so significant for low-rise frames but it is significant for high-rise frames. By using TADAS devices, GDP decreases up to 55 percent averagely.

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1. Introduction

Using the inelastic deformation of metals is a useful strategy to dissipate input energy in structures due to the earthquake loads. In traditional structures this inelastic deformation is generally concentrated in beam-column joints and thus is associated with damage to the main structural elements [1]. In order to achieve economical earthquake-resistant buildings, structures must be constructed to absorb and dissipate a large amount of seismic energy. In recent years, a number of researchers have investigated different techniques increasing the building energy absorbing capacity through the use of steel-plate added damping and stiffness (ADAS) device [2–5]. The use of mechanical dampers provides the means for consuming the kinetic energy of a vibrating building without sacrificing the integrity of primary structure elements, and also represents an effective method reducing the possibility of the resonant response [6]. The effectiveness of dampers is now well recognized for consuming much of earthquake-induced energy in disposable elements which are not part of the gravity framing system [7]. During the past years, it had been confirmed that the

earthquake-induced energy in a building structure can be effectively dissipated by the use of bolted X-shaped steel plate added damping and stiffness devices. Recent experimental results obtained in National Taiwan University also indicate that properly welded steel triangular-plate added damping and stiffness (TADAS) devices can sustain a large number of yielding reversals without any sign of stiffness or strength degradation. The performance of buildings can be increased employing additional dampers since these devices can absorb and damp some of the earthquake input energy [8]. These devices exhibit stable hysteretic behavior; they are insensitive to thermal effects, and extremely reliable. Bergman et al. [2] evaluated the cyclic testing of steel-plate devices for added damping and stiffness. Whittaker et al. [9] worked on the seismic testing of steel plate energy dissipation devices. Tsai et al. [10] studied on the design of steel triangular plate energy absorbers for seismic-resistant construction. Kobori et al. [11] developed some techniques on the application of hysteresis steel dampers. Two important aspects in the use of energy dissipating devices in earthquake engineering applications are: (i) to have a stable and sufficiently large dissipation capacity capable of controlling the earthquake response of the structure, and (ii) to have a representative model of its cyclic behavior [12]. Earthquake energy dissipation is concentrated in locations which have been designed for this purpose in TADAS device. Energy dissipation demands on other structural members can be substantially reduced, and because the devices are part of the lateral load resisting system only, yielding of

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Fig. 1. TADAS device-to-brace connection details [17].

TADAS devices will not affect the gravity load service capacity of the structural system. The TADAS devices can be easily replaced after an earthquake, if necessary [13]. Previously elastic analysis was the main method in the seismic design of structures. However, behavior of structures during recent earthquakes indicates that relying on just elastic analysis is not sufficient. On the other hand, nonlinear dynamic analysis, although yields accurate results, is time consuming and more complex. Such analysis must be repeated for each time step in the acceleration time histories, not to mention the need for delicate interpretation of its results. Researchers have long been interested in developing fast and efficient methods to simulate nonlinear behavior of structures under earthquake loads. The idea of inelastic static pushover analysis was first introduced by Freeman for single degree of freedom (SDOF) systems in 1975. Then other researchers extended this method for multi-degree of freedom systems [14]. To evaluate the nonlinear

behavior of the structures, the nonlinear static (pushover) analysis was performed by subjecting a structure to monotonically increasing lateral forces representing inertia forces in an earthquake until a target displacement is exceeded.

The aim of the research is to investigate the nonlinear behavior of TADAS frames by using the nonlinear static analyses. This study mainly focuses on the effects of using TADAS devices and evaluating the impact of nonlinear factors on the responses of structural frames such as target displacement, effective and elastic lateral stiffness and fundamental period, pushover curve and the response modification factor. The results are estimated and compared for two case of moment resisting frames (MRFs) and moment resisting frames equipped with TADAS devices (TMRFs).

2. Triangular-plate added damping and stiffness damper (TADAS)

2.1. General

One of the most effective mechanisms for dissipating input energy to the structure during an earthquake is an inelastic deformation of metals [15]. Fig. 1 depicts the typical configuration of TADAS damper and connection of the device to the brace. Mentioned damper consists of several triangular plates welded to a common base plate as shown in Fig. 1. Each triangular plate is inserted into the slotted base plate before fillet welds are applied to attach the triangular plate. During earthquakes, the inter-story drifts cause movement of the upper end of TADAS damper relative to the lower end. This causes yielding of metallic plates of the damper and as a result, the energy is dissipated. Fig. 2 shows the behavior of TADAS damper during earthquake. TADAS is a variation of ADAS consisting of triangular plate elements that are made to deform as cantilever beams [16]. TADAS dampers are usually made from steel. If mild steels are designed to use, TADAS deforms so much when structure vibrates during an earthquake, then there is a permanent deformation.

Because of triangular shapes of TADAS dampers, the metal plates experience uniform flexural strains along their length. Thus, when the strain reaches the yield level, yielding occurs over their entire volume. During cyclic deformations, the metal plates are subjected to hysteretic mechanism and the plastification of these plates consumes a substantial portion of the earthquake energy. The behavior of the TADAS damper under loading shown in Fig. 3.

2.2. Hysteretic damping mechanism

Energy dissipation in different dampers relies on hysteresis materials such as structural steels. Different hysteretic loops for passive control systems and a typical hysteretic loop for a TADAS device, respectively shown in Fig. 4 and Fig. 5. Past experiments have confirmed that the properly designed TADAS devices could absorb a large amount of hysteresis energy, thereby reducing the structural response during severe earthquakes [19]. Generally the device will yield before the frame undergoes inelastic deformations, it has been found that the

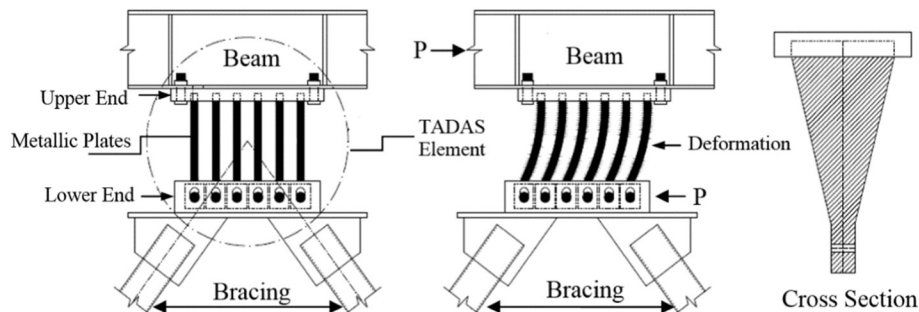


Fig. 2. Deformation of TADAS damper during earthquake [18].

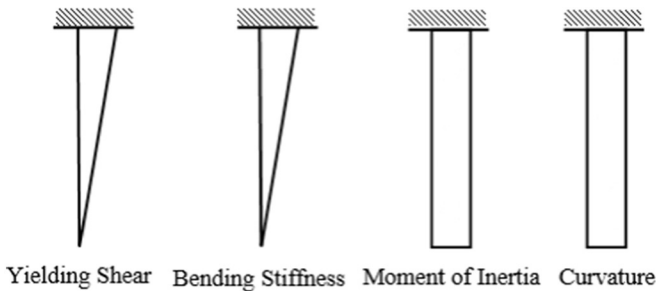
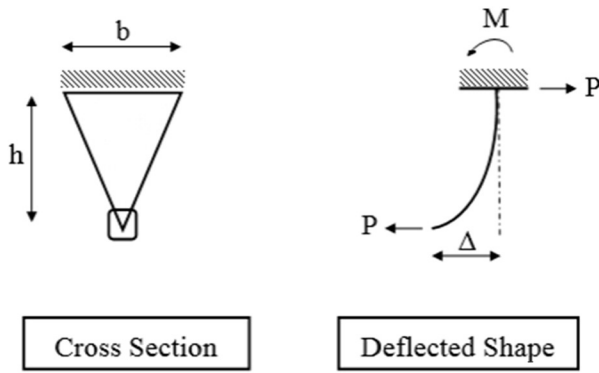


Fig. 3. Behavior of the TADAS under loading [17].

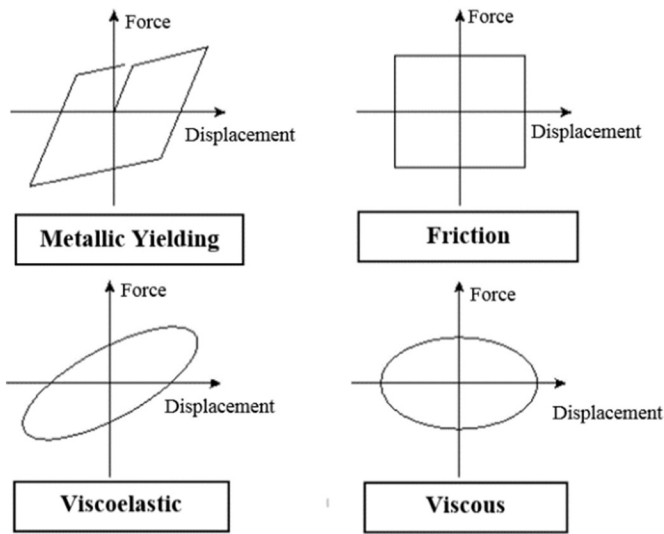


Fig. 4. Hysteretic loop for passive control systems.

force-deformation relationships of a steel frame with the TADAS device can be adequately characterized by a tri-linear model as shown in Fig. 6 [10].

2.3. Mechanics-based modeling

There is no direct relationship between model parameters and damper geometry. Alternatively, a force-displacement relationship can be provided from a constitutive model of the metal, along with a geometric description of the device, by using the laws of mechanics. TADAS, which consists of N identical triangular plates positioned in parallel, is typically installed within a frame bay between a chevron brace and the overlying beam. The base of each plate is welded into a rigid

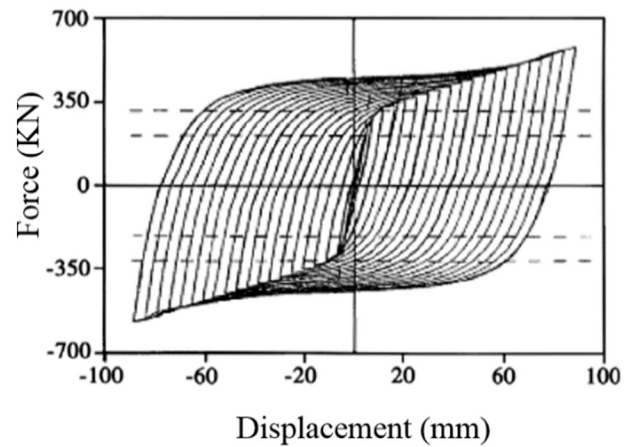


Fig. 5. A typical hysteretic loop for a TADAS device [17].

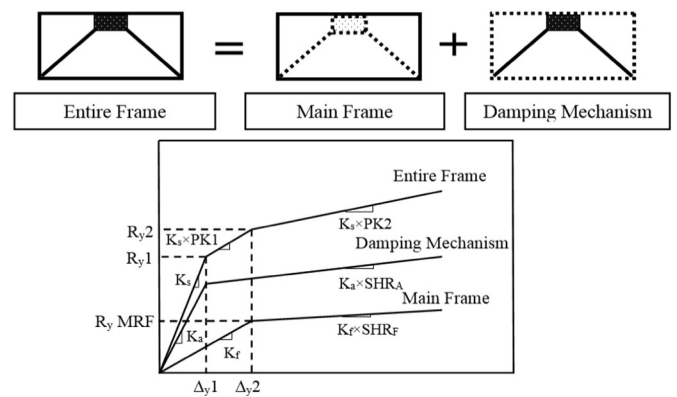


Fig. 6. Tri-linear force versus deformation relationships of the TADAS frame system [17].

base plate to approximate a fixed end condition, while a slotted pin connection is used at the tip to ensure relatively free movement in the vertical direction. As a result of this configuration, the damper primarily resists lateral forces P , along with an interstory drift D , via uniform flexural deformation of the individual plates. Thus, it is proper to assume a single cantilevered plate of thickness h , length L , and base width w_0 , subjected to a load P/N inserted at its free end as detailed in Fig. 7 and Fig. 8. Coordinate axes x , y and z are defined on the undeformed midsurface of the plate.

The force-displacement relationship for the damper can be readily established for infinitesimal, elastic response. For that case, the classical Euler-Bernoulli beam theory is valid. A quasistatic formulation is

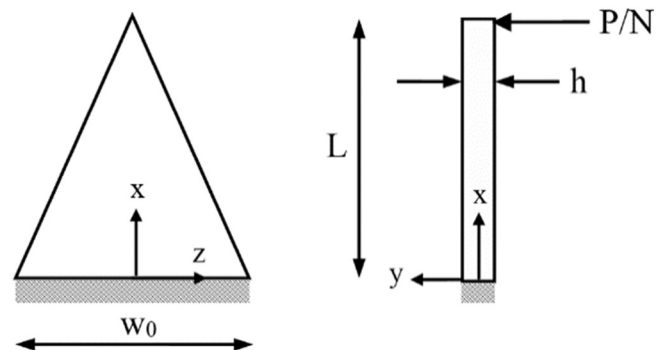


Fig. 7. Geometric definition for triangular plate device.

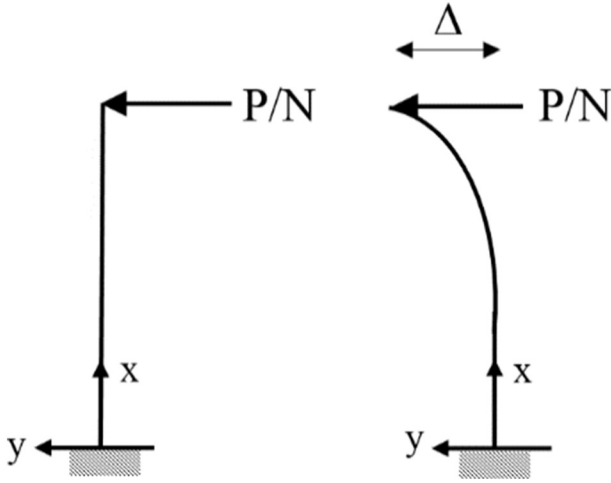


Fig. 8. Beam idealization for triangular plate device.

adopted by ignoring inertia of the plate. Then, at any cross-section, the moment equilibrium equation can be written [20]:

$$\frac{P}{N}(L-x) = \frac{w_0(L-x)}{L} \int_{-\frac{h}{2}}^{+\frac{h}{2}} y \sigma dy \quad (1)$$

where h and σ are the thickness of triangular plate and the stress respectively. With the cancellation of the term $(L-x)$ from both sides of Eq. (1), it is evident that the stress is independent of position along the beam axis. After applying the unidirectional elastic constitutive relationship:

$$S = Ee \quad (2)$$

in which S is stress, E is the elastic module and e shows the strain. According to the kinematic condition:

$$e = ky \quad (3)$$

in which k is the curvature and is constant along the entire length of the plate, with:

$$k = \frac{2\Delta}{L^2} \quad (4)$$

in Eq. (4), Δ is the displacement.

Substituting Eqs. (2), (3), and (4) in Eq. (1), it is possible to find relation between lateral load P and deformation Δ with expression:

$$P = \frac{NEw_0h^3}{6L^3} \Delta \quad (5)$$

2.4. Design of TADAS frames

A practical design procedure, incorporating the seismic-resistant design concept adopted in the Japanese Building Standard Law, is devised for the construction of seismic-resistant TADAS frame systems as follows [10]:

- 1- Establish the site-dependent service level design earthquake, an event corresponding to an effective peak ground acceleration of 80 cm/s^2 for buildings in regions of high seismic risk is generally adopted.
- 2- Select a suitable stiffness ratio (SR) value based on the fundamental period estimated for the frame. The horizontal stiffness of the TADAS

element K_a , the elastic stiffness K_s and the stiffness ratio SR are defined as follows:

$$K_a = \frac{K_b K_d}{K_b + K_d} \quad (6)$$

$$K_s = K_a + K_f \quad (7)$$

$$SR = \frac{K_a}{K_f} \quad (8)$$

where K_b and K_d are the lateral stiffness of the braces and the device stiffness respectively. K_f is the structural story stiffness without the TADAS device and braces in place.

- 3- Proportion the frame without the TADAS element in place. A moment resisting frame (MRF) capable of resisting at least 25% of the prescribed seismic force is recommended. Compute the lateral stiffness K_f of each floor. Compute the yield displacement of the bare frame, Δ_y2 .
- 4- Compute the TADAS element stiffness, K_a , from Eq. (8) and the yield displacement of the device, Δ_y1 for selected stiffness ratio (SR), ratio of the TADAS element (with braces) stiffness (SHR_A) and strength ratio (U) from Eq. (9).

$$\frac{\Delta_y2}{\Delta_y1} = \frac{UK_a + UK_f + K_a SHR_A - K_a}{K_f + K_a SHR_A} \quad (9)$$

- 5- Perform lateral force analyses for the frame with the TADAS element in place using the design earthquake given in Step 1. When necessary, repeat steps 3 and 4 to meet member strength and frame drift requirement for serviceability limit state.
- 6- Perform capacity design checks for the braces, columns and beams assuming the ultimate force generated in the device is $1.5 P_p$, where P_p is plastic force.

3. Design of model frames

3.1. Specifications of steel frames

To evaluate the nonlinear behavior of moment resistant frames (MRFs) and moment steel frames equipped with triangular-plate added damping and stiffness (TADAS) devices (TMRFs), four cases with 3, 6, 9 and 12-story 2-dimensional regular generic frames that have three bays are considered in the present study. The bay length of 4 m and the story height of 3 m were designed as per the requirement of the Iranian Earthquake Resistance Design Code [21] and Iranian National Building Code (INBC) section 10 [22], which are almost identical to AISC ASD design code [23] for steel structures. The moment resistant frames with different stories, including 3-MRF, 6-MRF, 9-MRF and 12-MRF are underdesigned in such a way that they can resist adequately one half of the codified base shear. Therefore, frames are retrofitted with TADAS devices. New frames with TADAS devices are defined as 3-TMRF, 6-TMRF, 9-TMRF and 12-TMRF. Typical geometry and section properties of the MRF and the TMRF frames with six stories and three bays, along with their plan view are depicted in Fig. 9. The considered soil is type II of Iranian Standard No. 2800 in the present study which is the same soil type C of Standard ASCE/SEI 41-06. For evaluating nonlinearity in nonlinear analysis, assumed frames are placed on an area with a high earthquake hazard, according to Iranian Earthquake Resistance Design Code, equal to $PGA = 0.35 \text{ g}$. The dead and live loads of 600 and 200 kg/m^2 were used in the current study, respectively. The base shear design was computed as:

$$V = CW \quad (10)$$

in which W , C and V are weight of the frame, the seismic coefficient and finally the seismic shear force respectively.

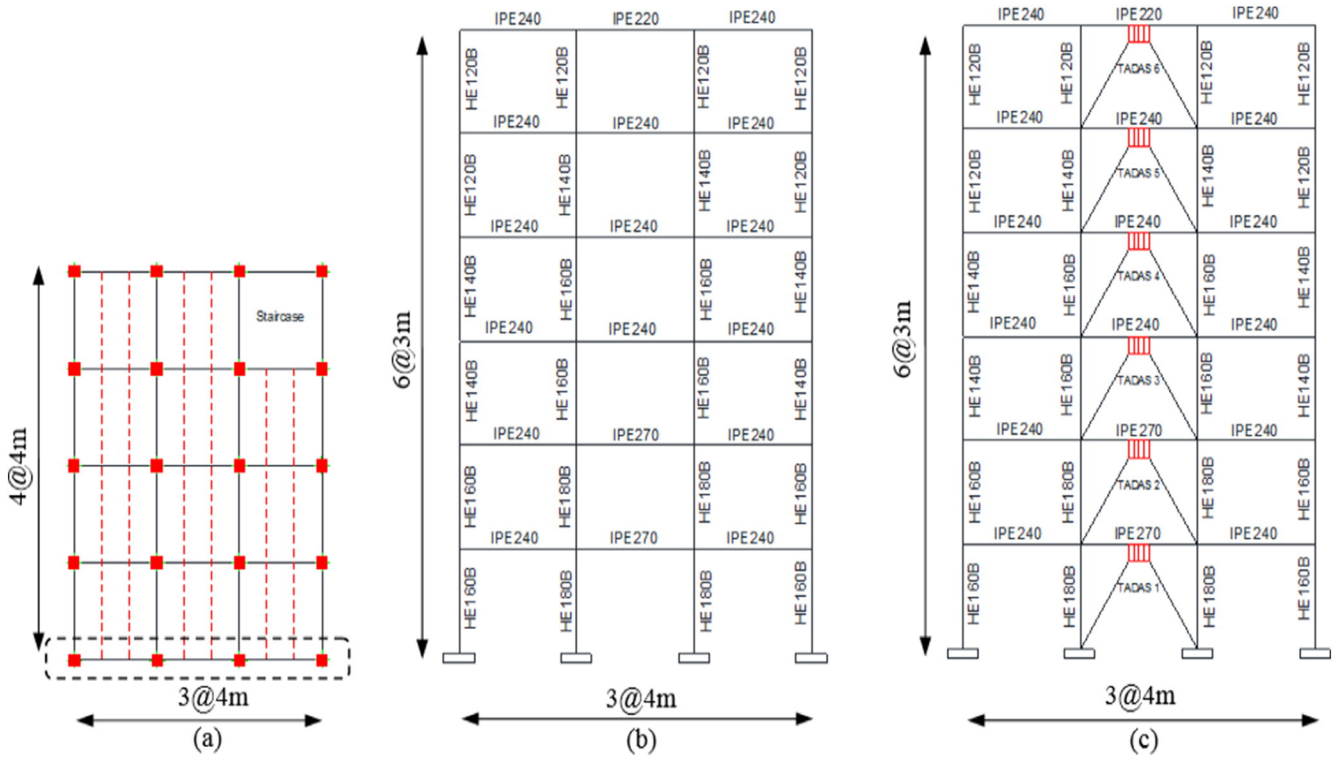


Fig. 9. Typical configuration of 6-story frames. (a) Plan view, (b) 6-MRF, (c) 6-TMRF.

To design all of frames, the required parameters such as importance factor I and preliminary response modification factor R , are considered 1 and 7 respectively. The beam-column connections are assumed to be moment resisting at both the ends.

3.2. Specifications of frames equipped with TADAS devices

Before designing of TADAS devices for vulnerable frames, to evaluate the accuracy of the basic TADAS model in the present study, the pseudodynamic response results for a two-story steel frame under the north-south component of the 1940 El Centro earthquake scaled to a peak ground acceleration (PGA) of 50 cm/s^2 are calculated and compared with the results given in Tsai et al. in reference 10. Fig. 10 shows the dimensions and member sizes of the two-story moment frame equipped with TADAS and investigated by Tsai et al. [10].

The parameters of TADAS devices such as thickness of triangular plate (t), height of triangular plate (h), base width of triangular plate (B), number of triangular plate (N), distance of triangular plate from each other (Gap), yield displacement (Δ_y), yield force (P_y) and plastic force (P_p) are given in Table 1.

The pseudodynamic response results given by Tsai et al. and the response result of same two-story frame modeled in the present research are presented in Fig. 11 and Fig. 12 respectively.

As mentioned, there are five steps that needed to be computed for designing of TADAS devices. The optimal values for TADAS parameters will depend upon the desired objectives which may be as simple as just to reduce a single response quantity like roof displacement or floor acceleration. Based on the recent studies [10], it is found that a strength ratio U of about 2 is optimal, SR value less than 4 and 2 are appropriate for systems of short and medium to long vibration periods, respectively. In the present study, strength ratio U and stiffness ratio SR of 2 and 3 are considered respectively. The damping ratio of 5% in each mode was assumed to define the inherent energy dissipation of structures. This ratio was used to construct the damping matrix for the

structure. It is assumed that there could be one device in each story. The ratio of the TADAS element (with braces) stiffness SHR_A are assumed 0.05 for each device. The Device stiffness (K_d) can be related to device parameters such as the number of triangular plate (N), elastic modulus (E), the base width of triangular plate (b), the thickness of triangular plate (t) and the height of triangular plate (h) as follows:

$$K_d = \frac{NEbt^3}{6h^3} \tag{11}$$

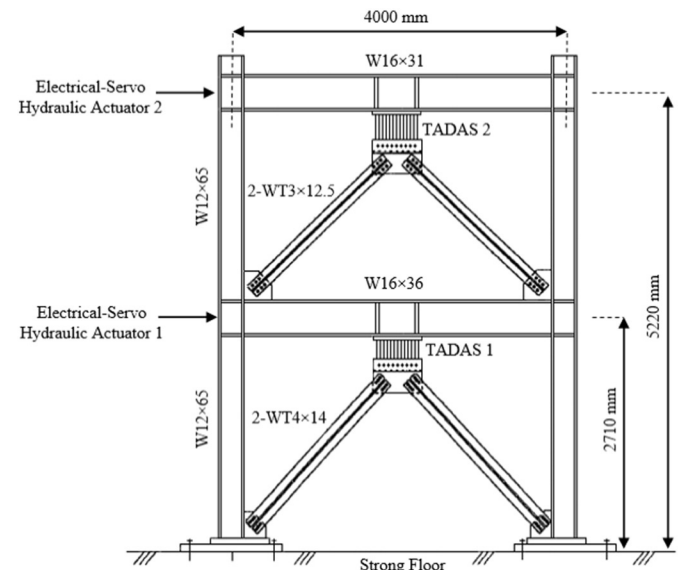


Fig. 10. Experimental set-up of pseudodynamic testing of a two-story TADAS frame [10].

Table 1
TADAS specification for two-story frame [10].

TADAS ID	t (mm)	h (mm)	B (mm)	N	Gap (mm)	Δ_y (mm)	P_y (KN)	P_p (KN)
TADAS 1	36	325	17,706	8	13	3.6	260.2	390.3
TADAS 2	36	325	178.5	5	13	3.6	163.5	245.3

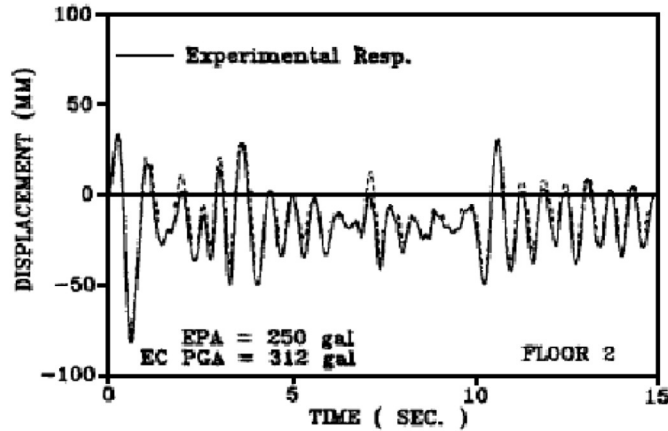


Fig. 11. Displacement response results by Tsai et al. [10].

also, the device parameters have the following relationships with the yield force (P_y), the plastic force (P_p) and the yield displacement (Δ_y).

$$P_y = \frac{F_y N b t^2}{6h} \quad (12)$$

$$P_p = \frac{F_y N b t^2}{4h} \quad (13)$$

$$\Delta_y = \frac{F_y h^2}{E t} \quad (14)$$

where F_y is yield stress in all these equations.

Specifications of 30 TADAS devices were defined in the current research for different frames. Typical TADAS specification for 6-TMRF is shown in Table 2.

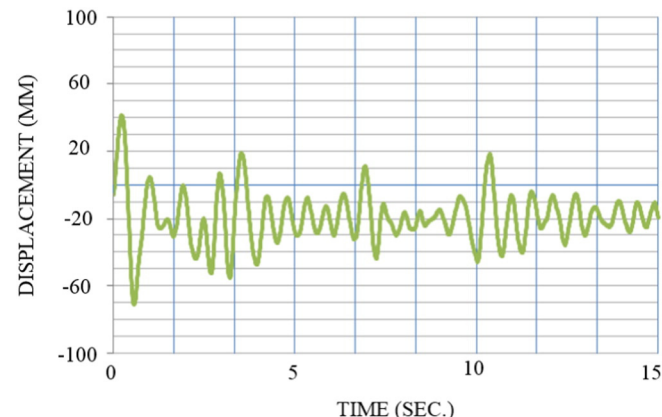


Fig. 12. Displacement response results of the present study.

4. Nonlinear static analysis

The Nonlinear Static Procedure (NSP), often called nonlinear pushover analysis, uses simplified procedure to evaluate and estimate the nonlinear behavior of structures. Pushover analysis has recently gained considerable popularity in the seismic assessment of structures and is appropriate for analyzing structures, especially for structures with short-period [24]. An appropriate engineering judgment can be obtained from the global pushover curve [25]. In the nonlinear static procedure, a mathematical model participating the load-deformation characteristics of individual components and elements of the structures will be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target point in displacement is exceeded. The target displacement is intended to represent the maximum displacement probable to be experienced during the design earthquake.

4.1. Target displacement

As mentioned, the target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The target displacement δ_t can be computed in accordance with Eq. (15) [26].

$$\delta_t = C_0 C_1 C_2 C_3 C_a \frac{T_e^2}{4\pi^2} g \quad (15)$$

in which C_0 is the modification factor to relate spectral displacement of an equivalent single degree of freedom (SDOF) system to the roof displacement of the building multi degree of freedom (MDOF) system, C_1 shows the modification factor to relate expected maximum inelastic displacements to displacements, C_2 indicates the modification factor to represent the effect of pinched hysteretic shape and stiffness degradation and also strength deterioration on maximum displacement response, C_3 is the modification factor to represent increased displacements due to dynamic P- Δ effects, S_a is the response spectrum acceleration, T_e shows the effective fundamental period of the building in the direction under consideration and finally g is the gravity acceleration.

4.2. Pushover curve and fundamental period

Pushover curve represents the relationship between a total strength (base shear) and a total deformation (roof displacement) obtained by subjecting the structure to a lateral load pattern. FEMA 440 has present that applying multiple load patterns does not improve the accuracy of pushover analysis considerably [27]. So in the current study a single pattern based on the first mode shape is used for extracting pushover curve.

The nonlinear force-displacement relationship (pushover curve) between base shear and displacement of the control node (center of mass at the roof of a structure) can be replaced with an idealized relationship to compute the effective lateral stiffness K_e and effective yield strength V_y of the structure as shown in Fig. 13. This relationship shall be bilinear, with initial slope K_e and post yield slope α . Line sections on the idealized force-displacement curve shall be located using an iterative graphical procedure that approximately balances the area above and below the curve. The effective lateral stiffness K_e will be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure. The post-yield slope α , can be determined by a line segment that crosses through the actual curve at the computed target displacement. Additionally, initial stiffness of the curve defined as initial or elastic stiffness K_i [26]. Fig. 13 shows a typical idealized force-displacement curve.

Table 2
Typical TADAS specification for 6-TMRF.

Story no.	Δ_y2 (cm)	Δ_y1 (cm)	K_r (Kgf/cm)	K_a (Kgf/cm)	$(K_d)_{min}$ (Kgf/cm)	$(K_b)_{min}$ (Kgf/cm)	P_p (Kgf)
1F	2.55	0.57	8350.730	25,052.191	37,578.287	75,156.575	21,419.623
2F	4.52	1.01	4573.519	13,720.557	20,580.836	41,161.673	20,786.645
3F	5.10	1.14	3579.098	10,737.294	16,105.941	32,211.882	18,360.772
4F	5.64	1.26	2712.452	8137.358	12,206.037	24,412.075	15,379.607
5F	6.54	1.46	1714.824	5144.473	7716.710	15,433.421	11,266.397
6F	4.57	1.02	1321.999	3965.997	5948.996	11,897.993	6067.976

Story no.	SR	SHR _A	U	h (mm)	b (mm)	t (mm)	N
1F	3	0.05	2	300	150	30	7
2F	3	0.05	2	300	150	25	7
3F	3	0.05	2	300	150	25	5
4F	3	0.05	2	300	150	20	8
5F	3	0.05	2	300	150	20	5
6F	3	0.05	2	300	150	20	4

The effective fundamental period T_e can be related to the idealized force-displacement curve shown in Fig. 13, by the following expression:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{16}$$

where T_i in the above formulation is the elastic fundamental period in the direction under investigation.

5. Response modification factor

The elastic analysis of structures under earthquake usually gives the base shear force and stress, bigger than the real response. The structure usually absorbs a lot of earthquake energy due to the inelastic deformation. Overstrength in structures is related to the fact that the maximum lateral strength of a structure generally exceeds its design strength. Hence, seismic codes reduce design loads, taking advantage of the fact that structures having overstrength and ductility. Therefore, the response modification factor includes inelastic performance of the structure and indicates overstrength and ductility of structure in inelastic stage [28]. In fact, the response modification factor is the ratio of strength required to maintain the structural elasticity [29].

There are several theoretical aspects such as the maximum plastic deformation, energy and low cycle fatigue approaches to computing the response modification factor [30]. As mentioned and shown in Fig. 13, the real nonlinear behavior of the structure is idealized by a bilinear elasto-plastic relationship. As shown in Fig. 14, usually the real

nonlinear behavior is idealized by a bilinear elasto-plastic relationship. In Fig. 14, the maximum base shear considering elastic behavior (V_e), the ultimate base shear (V_u), the yield force of the structure (V_y), maximum seismic demand for elastic response (V_d), the yield displacement of the structure (Δ_y), the displacement of a corresponding elastic structure (Δ_e) and the maximum displacement of a structure (Δ_{max}) are determined clearly.

Consequently, the response modification factor (R) is defined as follows:

$$R = R_{\mu} \cdot R_s \tag{17}$$

in which R_{μ} and R_s are the reduction factor due to ductility and the overstrength factor of structure respectively.

5.1. Reduction factor due to ductility

Reduction factor due to the ductility (R_{μ}) is defined as the ratio of the maximum base shear considering elastic behavior (V_e) to the yield force of structure (V_y) [29]:

$$R_{\mu} = \frac{V_e}{V_y} \tag{18}$$

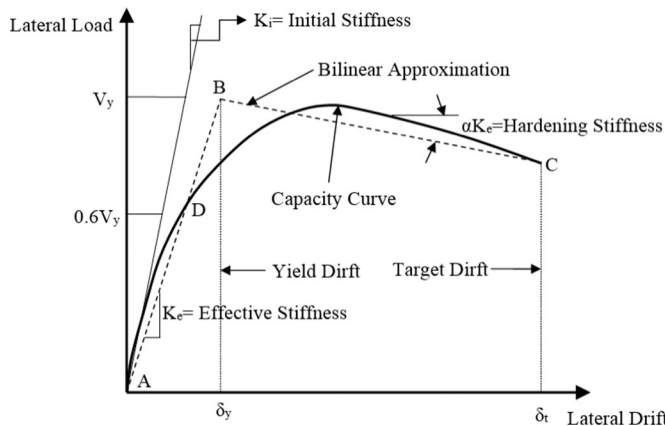


Fig. 13. Typical idealized force-displacement curve.

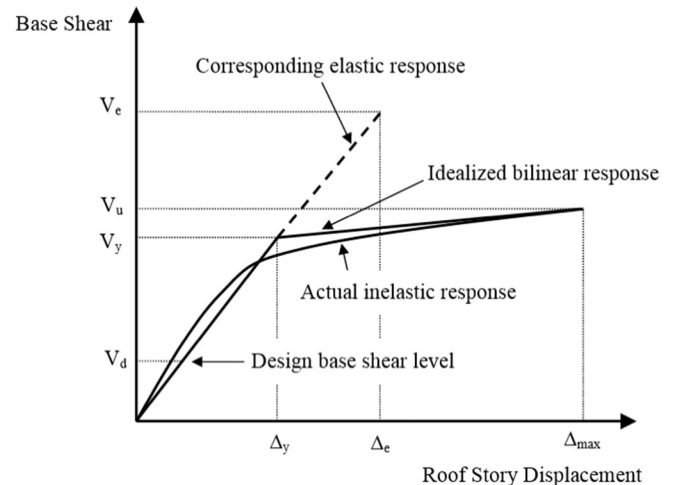


Fig. 14. General structure response.

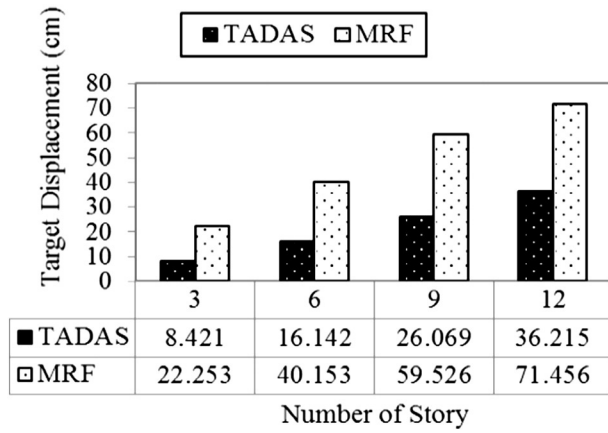


Fig. 15. Target displacement of MRFs and TMRFs.

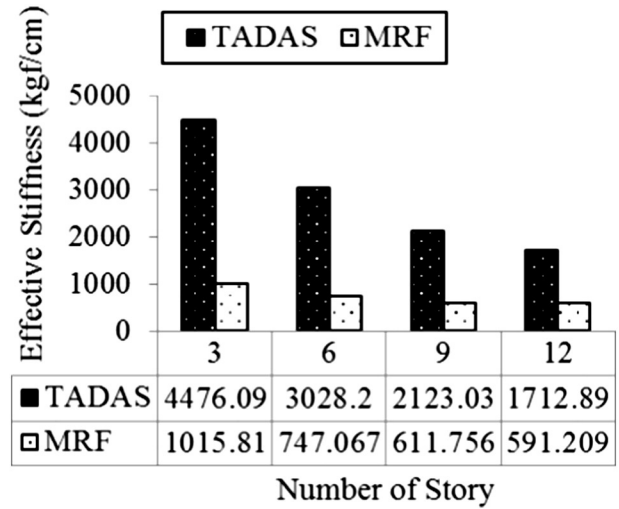


Fig. 17. Effective stiffness of MRFs and TMRFs.

5.2. Overstrength factor

As observed during some of the intermittent earthquake occurrences, it seemed building structures could take force significantly larger than they provided [29]. The presence of significant reserve strength that was not considered in design, explains this phenomenon. Overstrength could help structures resist safely not only against sever vibration, but it decreases the elastic strength demand, as well. The overstrength factor (R_s) can be written as below:

$$R_s = \frac{V_y}{V_d} \tag{19}$$

in which V_y and V_d are the yield force of structure and the maximum seismic demand for elastic response respectively.

Finally, with combining Eq. (18) and Eq. (19), the response modification factor (R) is obtained by dividing of elastic shear (V_e) to design shear (V_d).

$$R = \frac{V_e}{V_d} \tag{20}$$

6. Global damage parameter

It is obvious that the earthquake resistant design of structures usually allows for a building to experience repairable damage during moderate and large earthquakes and vibrations. Sometimes, the damage of a structure is probably irreparable and may even gradually lead to collapse the structure. Therefore, it is important to measure the damage quantity in order to seismic evaluation of structures [31]. Seismic vulnerability is known as a quantity of damage that applying to the series of factors under the earthquake and the amount of seismic vulnerability varies from zero to one. According to the recent research, there are several different methods for seismic evaluating of the present structures. Additional to controlling of the performance of structures, recognizing the quantity of damage due to earthquake can be significant in making decision about that structure [32]. Also assessment of the existing building stock regarding the effects of deterioration during the time is very important. A closely related subject is the safety evaluation of structures that have been damaged, either by accidental loadings such as blast or by extreme environmental events like storms or earthquakes. One of the significant damage indexes was proposed by Roufaiel and Meyer in 1987. The global damage parameter (GDP) has been presented to

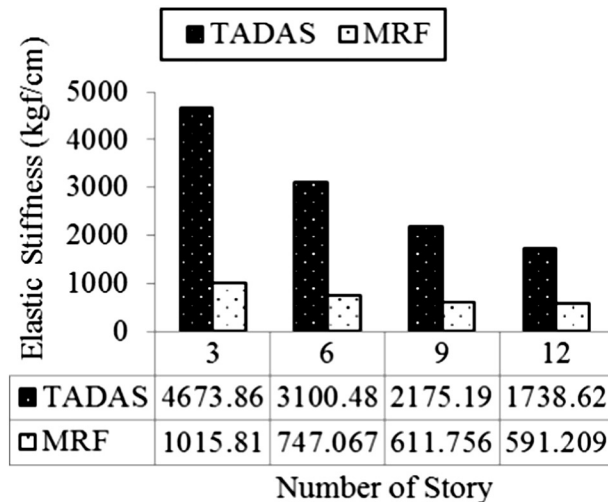


Fig. 16. Elastic stiffness of MRFs and TMRFs.

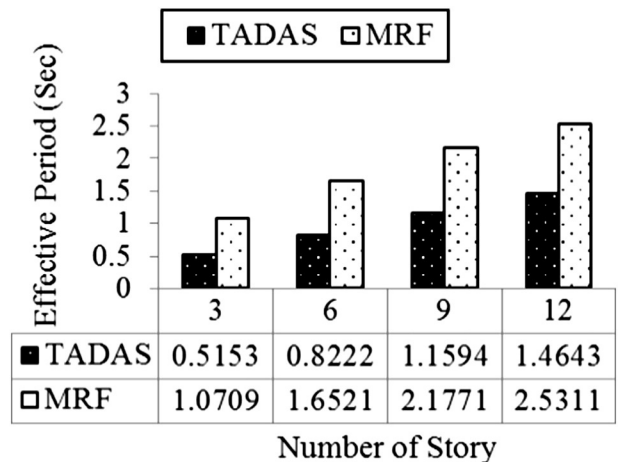


Fig. 18. Elastic period of MRFs and TMRFs.



Fig. 19. Effective period of MRFs and TMRFs.

define the overall degree of damage in a frame building by the following expression [33]:

$$GDP = \frac{d_R - d_Y}{d_F - d_Y} = \frac{14.2\delta_y \left(\sqrt{\frac{\omega_e}{\omega} - 1} \right)}{\delta_f - \delta_y} \quad (21)$$

where d_R , d_Y and d_F are known as the maximum roof displacement, the roof displacement at which the first member in the frame reaches its yield moment capacity assuming the frame displaces in its first mode only and the roof displacement at which the frame is assumed to fail respectively. Also ω_e , ω , δ_y and δ_f are the frequency of the undamaged or elastic frame, the current fundamental frequency of the frame, displacement of the roof corresponding to structural collapse of the first hinge on one of the members of the story and displacement of the roof corresponding to structural collapse, respectively.

There are various failure modes possible, but at this point the definition will be restricted to excessive roof displacements. The global damage parameter is set equal to 0 if $d_R < d_Y$. In this case, no member has yielded; therefore no damage observe. At the other extreme $d_R \geq d_F$, the frame has failed.

7. Results

The target displacements for 3-MRF, 3-TMRF, 6-MRF, 6-TMRF, 9-MRF, 9-TMRF, 12-MRF and 12-TMRF are calculated and shown in

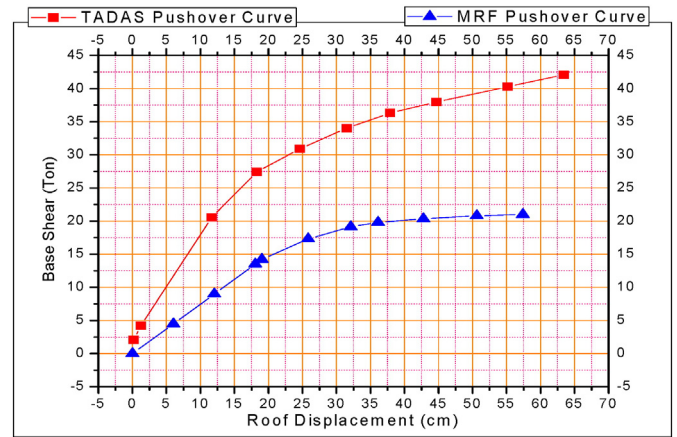


Fig. 21. Pushover curve for 6-MRF and 6-TMRF.

Fig. 15. As clearly observe, using TADAS devices decrease more than 55% of the target displacement due to significant damping and stiffness effects in comparison with MRFs.

In Fig. 16 and Fig. 17, elastic stiffness and effective stiffness of MRFs and TMRFs are calculated and presented respectively and two Figs. 18 and 19 give elastic fundamental period and effective fundamental period for MRFs and TMRFs. Elastic and effective stiffness of MRFs and TMRFs due to increasing of the story levels, decrease to approximately 41% and 62% respectively.

The nonlinear static analysis results in terms of base shear–roof displacement for MRFs and TMRFs have been shown in Figs. 20–23. As indicated, frames equipped TADAS devices contain high capacity compared with MRFs because of the area under the TMRFs pushover curves.

Table 3 highlights the response modification factor of MRFs and TMRFs estimated from nonlinear static analysis. In addition, elastic shear and design shear are presented. Variation of design shear is almost uniform between MRFs and TMRFs for each frame with the same story level. It is found that the influence of TADAS dampers on the elastic shear is more than the design shear hence; this influence in middle stories is greater than the others. Another important point is that the response modification factor decreases gradually with an increase in the height of structures.

Finally, Table 4 gives the global damage parameter (GDP) or damage index for 3-MRF, 3-TMRF, 6-MRF, 6-TMRF, 9-MRF, 9-TMRF, 12-MRF and 12-TMRF. Also, three parameters of d_Y , d_F and d_R are listed in Table 4. It

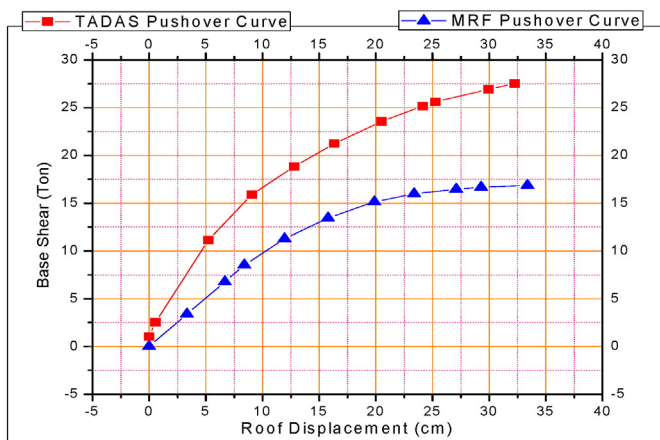


Fig. 20. Pushover curve for 3-MRF and 3-TMRF.

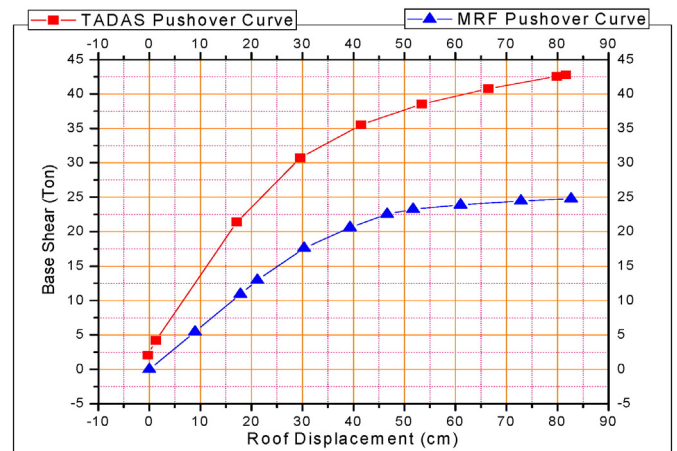


Fig. 22. Pushover curve for 9-MRF and 9-TMRF.

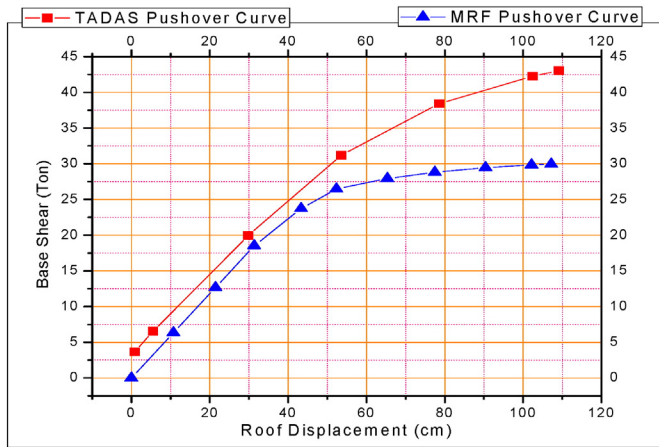


Fig. 23. Pushover curve for 12-MRF and 12-TMRF.

Table 3
Response modification factor of MRFs and TMRFs.

Frame ID	Elastic Shear (V_e) (ton)	Design Shear (V_d) (ton)	Response modification factor (R)
3-MRF	27.96	5.98	4.67
3-TMRF	50.28	5.99	8.39
6-MRF	37.05	9.63	3.85
6-TMRF	72.87	9.67	7.54
9-MRF	44.51	11.87	3.75
9-TMRF	68.11	11.92	5.71
12-MRF	52.56	13.78	3.81
12-TMRF	77.28	13.85	5.58

Table 4
Global damage parameter of MRFs and TMRFs.

Frame ID	dY parameter (cm)	dF parameter (cm)	dR parameter (cm)	Global damage parameter (GDP)
3-MRF	9.29	20.88	16.67	0.64
3-TMRF	10.76	16.07	11.92	0.22
6-MRF	24.95	28.71	27.39	0.65
6-TMRF	22.25	24.64	22.84	0.25
9-MRF	26.52	32.78	30.61	0.65
9-TMRF	28.58	31.26	29.39	0.30
12-MRF	37.18	50.70	46.22	0.67
12-TMRF	35.33	41.83	37.93	0.40

can be seen that these three factors ascend by increase in the story height of the frame. The results also show the influence of TADAS dampers when the GDP decreases. These influences are not so high for low-rise frames but is so significant for high-rise frames. For 3-story frames, decrease in GDP is 65% but for 6-story frame is 61%. For 9-story framed the GDP decrease 46% comparing the frame equipped with TADAS and Frame without any dissipate energy devices.

8. Conclusion

The present research investigated the seismic nonlinear behavior and the global damage parameter of MRFs and TMRFs with different story levels. Depending on the severity of the earthquake, the structures may undergo different nonlinear behavior. In the present study, nonlinear static analysis was performed to evaluate the nonlinear behavior of

structures. The result of the proposed models is estimated and study can thus be summarized as follows:

- 1) The estimated target displacement decreased from 48.35 cm for MRFs to 21.71 cm for TMRFs due to the damping and stiffness in TMRFs.
- 2) Elastic lateral stiffness of MRFs and TMRFs are estimated which are 741.46 kgf/cm² and 2922.04 kgf/cm² respectively. Furthermore, effective lateral stiffness of MRFs and TMRFs are 741.46 kgf/cm² and 2835.05 kgf/cm² respectively. Clearly, the effect of TADAS device is more significant on the elastic lateral stiffness.
- 3) Elastic fundamental period of MRFs and TMRFs are 1.86 s and 0.98 s in order. Also, effective fundamental period of MRFs and TMRFs are 1.86 s and 0.99 s. Performance of TADAS device is obvious in decreasing of elastic and effective fundamental periods.
- 4) TADAS devices increase the capacity of MRFs according to pushover curves of TMRFs.
- 5) The response modification factor by using of TADAS devices, increasing between 45% to 75%. The enhancement of the response modification factor is related to the story level. By increasing the story level, the growth rate of the enhancement shall be low.
- 6) To controlling of the efficiency of the buildings, determination of the quantity of the damage by earthquake can be significant to making decision about desired structure. Using of TADAS devices are recommended to increasing of the global damage parameter. By using TADAS devices, GDP decreases up to 55%.

Nomenclature

Notations

ADAS	added damping and stiffness
TADAS	triangular plate added damping and stiffness
K_a	horizontal stiffness of the TADAS element
K_f	the structural story stiffness without the TADAS device and braces in place
K_s	elastic stiffness
Δ_{y1}	yield displacements of the TADAS element (with braces)
Δ_{y2}	yield displacements of the frame without the TADAS element (device and braces)
R_{y1}	total restoring forces developed in the system when Δ_{y1} is reached
R_{y2}	total restoring forces developed in the system when Δ_{y2} is reached
SHR_A	ratio of the TADAS element (with braces) stiffness
U	strength ratio
MRF	moment resistant frame
P	the damper primarily resists horizontal force
N	number of identical triangular plates
L	length
x	horizontal coordinate axe
w_0 or B	base width
h or t	thickness of triangular plate
y	distance of stress axis from neutral axis
σ or S	stress
E	elastic modulus
e	strain
k	curvature
Δ	displacement
SR	stiffness ratio
K_b	lateral stiffness of the braces
K_d	device stiffness
TMRF	moment resistant frame with TADAS device
INBC	Iranian National Building Code

Δ_y or δ_y	yield displacement
P_y	yield force
P_p	plastic force
PGA	peak ground acceleration
F_y	yield stress
NSP	nonlinear static procedure
SDOF	single degree of freedom
MDOF	multi degree of freedom
δ_t	target displacement
C_0	modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system
C_1	modification factor to relate expected maximum inelastic displacements to displacements
C_2	modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response
C_3	modification factor to represent increased displacements due to dynamic P- Δ effects
S_a	response spectrum acceleration
T_e	effective fundamental period of the building in the direction under consideration
g	acceleration of gravity
K_e	effective lateral stiffness
V_y	effective yield strength
α	post yield slope
K_i	initial or elastic stiffness
T_e	effective fundamental period in the direction under consideration
T_i	elastic fundamental period in the direction under consideration
V_e	maximum base shear considering elastic behavior
V_u	ultimate base shear
V_d	maximum seismic demand for elastic response
Δ_e	displacement of a corresponding elastic structure
Δ_{max}	maximum displacement of a structure
R	response modification factor
R_s	overstrength factor of structure
R_{μ}	reduction factor due to ductility
GDP	global damage parameter
d_R	maximum roof displacement
d_Y	roof displacement at which the first member in the frame reaches its yield moment capacity assuming the frame displaces in its first mode only
d_F	roof displacement at which the frame is assumed to fail
ω_e	frequency of the undamaged or elastic frame
ω	current fundamental frequency of the frame
δ_y	displacement of the roof corresponding to structural collapse of the first hinge in one of the member of the story
δ_f	displacement of the roof corresponding to structural collapse

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