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THE EFFECT OF DUCTILITY ON RESPONSE MODIFICATION FACTORS OF TADAS FRAMES

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Keywords

Hysteretic dampers

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dynamic analysis

Response modification factor

A B S T R A C T

One of the mechanism, used in order to provide positive control of structural vibration in the wake of earthquakes is hysteretic damper which dissipates the energy exerted into a structure. The TADAS (triangular-plate added damping and stiffness) device is one of them with elasto-plastic behavior. In this paper, the effect of ductility on the response modification factors of the frames equipped with TADAS dampers has been studied. For that matter, multi-story buildings were considered. The nonlinear incremental dynamic analysis and linear dynamic analysis have been performed using OpenSees software. In this research, seismic response modification factors for moderate and special moment resisting frames (MMRFs & SMRFs) with TADAS devices (T-MMRFs & T-SMRFs) and without them has been determined separately. The results showed that the response modification factors for TADAS frames were higher than the frames without TADAS devices. It was also found that the response modification factors for T-SMRFs were higher than T-MMRFs and also the ductility of structures has greater effect on the response modification factors of the frames with and without TADAS devices.

1 INTRODUCTION

Recently, One of The mechanisms has been used in order to provide positive control of structural vibration in the wake of earthquakes is hysteretic damper which dissipate the energy exerted into a structure. The TADAS (triangular-plate added damping and stiffness) device is one of the examples of the hysteretic dampers with elasto-plastic behavior (Inoue and Kuwahara, 1998). In fact, the damage to the main frame could effectively be reduced by adequately incorporating hysteretic dampers into the structure. Here, the major consideration is the selection of strength and stiffness of hysteretic dampers for maximizing the damping effect as well as minimizing the damage to the main frame. With regard to TADAS

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dampers with elastic-plastic behavior, Tsai et al. (Tsai et al, 1993) numerically examined its strength and stiffness on earthquake response and hence; obtained optimal combination of strength and stiffness. The above outcome although provided important background for the structural design combined with TADAS dampers, however, it remained empirical because the finding was only based on the numerical parametric analysis. Seismic design codes help reduce design loads, taking advantage of the fact that structures possess significant reserve strength (overstrength) and the capacity to dissipate energy (ductility). These two factors are incorporated in structural design through a force reduction or a response modification factor. If it was to remain elastic to design seismic force, the above factor must represent maximum ratio of force on a structure during specified ground motion. Consequently, to obtain design forces, the actual seismic forces are reduced by the factor ''R''. The basic flaw in code procedures is that they use the linear method while relying on the nonlinear behavior (Asgarian and Shokrgozar, 2009). The response modification factors were first proposed in ATC-3-06 (ATC-3-06, 1978). The product of three factors i.e. Overstrength, Ductility, and Redundancy were calculated in ATC-19 (ATC-19, 1995) and ATC-34 (ATC-34, 1995). The response modification factor for moderate and special moment resisting frames with TADAS devices (T-MMRFs & T-SMRFs) should be computed relatively, defining the system according to its ductility and performance in a manner consistent with factors already established for other structural systems, such as ordinary moment resisting frames (OMRFs), moderate moment resisting frames (MMRFs) and special moment resisting frames (SMRFs). The present study focuses on effect of the changes of ductility on response modification factors for the frames equipped with TADAS dampers. The frames were designed in accordance with the Iranian Earthquake Resistant Design Codes (BHRC, 2005) and the Iranian National Building Code for Structural Steel Design (MHUD, 2006). To obtain the proposed factors, nonlinear incremental dynamic analysis and linear dynamic analysis were carried out.

2 MODELING VEGETATIVE ROUGHNESS

The elastic analysis of structures exposed to earthquake could create base-shear force and stresses which noticeably are bigger than the real structural response. In a structure, overstrength means the maximum lateral strength generally exceeds its design strength. Hence, seismic codes reduce design loads, taking advantage of the fact that structures possess overstrength and ductility. In fact, the response modification factor includes inelastic performance of structure and indicates overstrength and structural ductility (Asgarian and Shokrgozar, 2009).

While computing the response modification factor, Mazzolani and Piluso (Mazzolani and Piluso, 1996) addressed several theoretical aspects such as the maximum plastic deformation, energy and low cycle fatigue approaches. As it is shown in Fig. 1, usually the real nonlinear behavior is idealized by a bilinear elasto-plastic relation (Asgarian and Shokrgozar, 2009). Here, the yield force of structure is shown by Vy and the yield displacement is Δy,. In this Fig., Ve (Vmax) corresponds to the elastic response strength of the structure. The maximum base shear in an elastic perfectly behavior is Vy (Asgarian and Shokrgozar, 2009). Consequently, the response modification factor is determined as follows :

$$R = R_{\mu}.R_S \tag{1}$$

Where, R_{μ} is a reduction factor due to ductility and Rs is the overstrength factor.

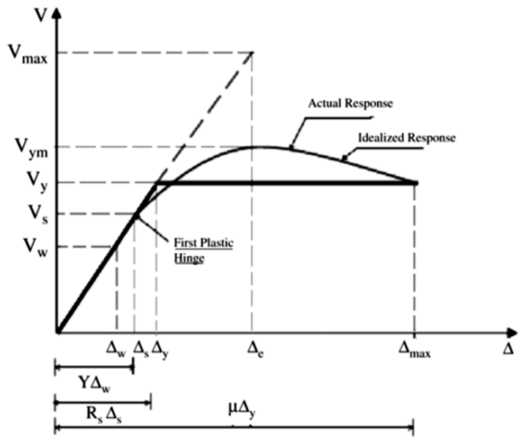


Fig. 1 Real and ideal structural response curves

2.1 Reduction factor due to ductility

As mentioned, R_μ is a parameter to measure the global nonlinear response of a structure, due to the hysteretic energy. The maximum base-shear ratio is called force reduction factor due to ductility considering the elastic behavior V_e to the yield force of structure V_y :

$$R_\mu = \frac{V_e}{V_y} \tag{2}$$

2.2 Overstrength factor

As observed during some of the intermittent quake occurrences, it seemed building structures could take force considerably larger than they designed for. The presence of significant reserve strength that was not accounted in design, explains this phenomenon (Asgarian and Shokrgozar, 2009). Overstrength could help structures stand safely not only against sever tremors but it reduces the elastic strength demand, as well. This object is performed using the force reduction factor (Mahmoudi, 2003). Here, the design overstrength factor (R_{sd}) is defined as (Tasnimi and Massumi, 2006):

$$R_{Sd} = \frac{V_y}{V_d} \tag{3}$$

Where, V_d is the design base-shear in the building and V_y is the base-shear in relevance to the first life safety (LS) performance (Fig. 1). Overstrength, redundancy and ductility, the three concepts used to scale down the earthquake forces need to be defined and expressed clearly in quantifiable terms.

Although, the overstrength factor is based on the applied nominal material properties, the actual overstrength factor should consider the help of some other effects (Asgarian and Shokrgozar, 2009):

$$R_S = R_{Sd}.F_1.F_2...F_n. \tag{4}$$

Here, parameter F_1 is used to account for difference between the actual and the nominal static yield strengths. A statistical study on structural steel shows that the value of F_1 might be 1.05 (Schmidt &

Bartlett, 2002). Parameter F_2 probably used to consider an increase in yield stress due to strain rate effect during an earthquake. To account the strain rate effect, value of 1.1 or a 10% increase could be used (Asgarian and Shokrgozar, 2009). It must be noted that the proposed research has used steel type St-37 for all structural members. Consequently, parameters F_1 and F_2 equal to 1.05 and 1.1 were considered taking into 1.155 as material overstrength factor. Other parameters such as nonstructural component contributions, variation of lateral force profile could be included once a reliable data is available (Asgarian and Shokrgozar, 2009).

3 TADAS YIELDING DAMPERS

3.1 General

Introducing energy-based seismic design and developing structural systems with hysteretic dampers, the current study mainly focuses on. As mentioned, there have been flourishing researches in recent years on hysteretic dampers that are incorporated into structures in order to achieve positive control of structural vibrations induced by wind and earthquakes. Past researches have already detailed about these dampers and their applications (Hanson et al, 1993; Aiken et al, 1993). Meanwhile, the current study gives special attention on the hysteretic dampers with TADAS (triangular-plate added damping and stiffness) that for energy dissipation rely on hysteresis materials such as structural steels.

The hysteretic behavior of a structural system with a TADAS damper can be simplified as in Fig. 2. In this Figure, the system consists of a main frame and a hysteretic damping mechanism. The hysteretic behavior of the damping system is assumed to be linear-elastic and perfectly-plastic, and hysteretic behavior of the main frame is also idealized as linear-elastic and perfectly-plastic. Thus, the hysteretic behavior of the entire structural system is given as the two linear-elastic and perfectly-plastic models linked in parallel (Inoue and Kuwahara, 1998).

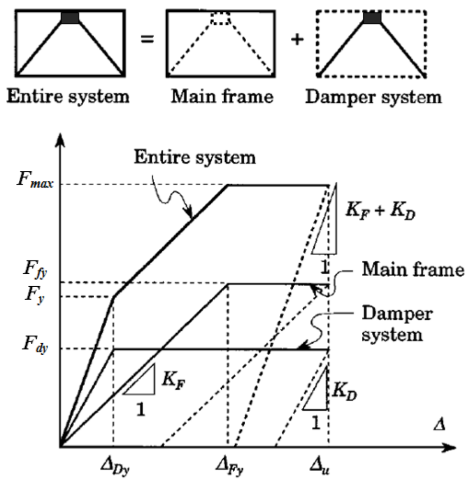


Fig. 2 Schematic hysteretic behavior of a structure with a hysteretic damper

The first yielding of the system corresponds to the yielding of the damping mechanism, while the second significant change in stiffness occurs at yielding of the main frame. The ratio of the stiffness after the first yielding (called the second stiffness) relative to the initial elastic stiffness (called the initial stiffness) is a function of the relative stiffness between the main frame and damping mechanism. According to previous designs applied for structures with hysteretic dampers, the ratio of second stiffness (Kf / (Kf + Kd)) cannot be assigned a small value, i.e., the main frame cannot be designed so flexibly as compared to the hysteretic damping mechanism because of the lateral stiffness provided to the frame in the course of design for design vertical load and others, and the ratio of second stiffness easily reaches more than 0.5 (Nakashima, Kazuhiro & Tsuji, 1996).

The fabrication details and the cyclic force versus deformation relationships of a typical TADAS device are shown in Fig. 3, where t, h, and B stand for the thickness, the height and the width of triangular plates, respectively. Past experiments have confirmed that the properly designed TADAS devices could absorb a large amount of hysteresis energy thereby reducing the structural responses during severe earthquakes (Chang, Tsai & Chen, 1998). The mechanical property of a TADAS device is highly predictable and has been documented in the reference section (Tsai K. C. et al, 1993; Tsai C. S. and Tsai K. C., 1995).

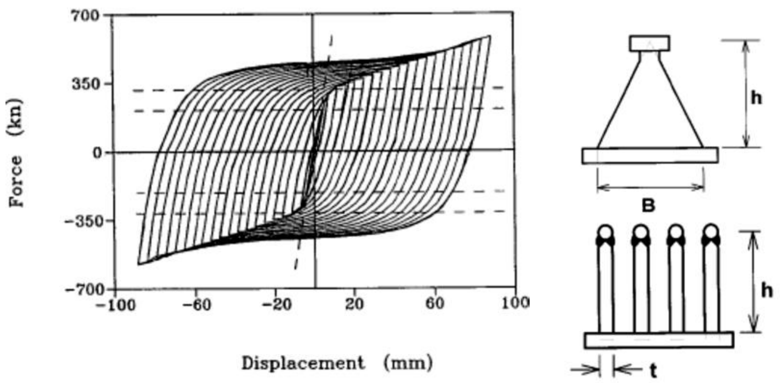


Fig. 3 A typical hysteretic loop and details for a TADAS device

3.2 Design parameters of TADAS devices

The force-deformation behavior of TADAS yielding dampers has some common characteristics. Further, the force-deformation response under arbitrary cyclic loading of hysteretic devices is often approximated by discrete multi-linear models i.e. elasto-perfectly plastic and bilinear ones. To represent more accurate constitutive behavior of these devices, some researchers have devised more comprehensive and accurate models (Moreschi and Singh, 2003). The current study has preferred a simple bi-linear hysteretic force-deformation model in order to facilitate the identification of parameters involved in designing a typical damper. Fig. 4(a) presents a structural frame bay installed with an added hysteretic damper. Herein, the combination of damper and the brace members supporting the device are called the device-brace assembly. The design parameters of such an assembly are the yield displacement and stiffness of the device as well as the brace. For given stiffness of the story of a building where device is installed, the yield force Py can be related to the device parameters as follows (Moreschi and Singh, 2003):

$$P_y = K_d \Delta_{yd} = SR.K_s (1+\frac{1}{B/D})\Delta_{yd} \quad (5)$$

Where Ks, Δyd and B/D=Kb/Kd are story stiffness, yield displacement and the ratio of the brace stiffness Kb to the device stiffness Kd, respectively. SR= Kbd/ Ks is the ratio of the assembly stiffness Kbd to story stiffness Ks. Fig. 4b & c schematically show the combined stiffness of device-brace assembly. In these Figures, the system consists of a main frame and a hysteretic damping mechanism, with both components being linked parallel and in terms of device and bracing stiffness Kd and Kb can be expressed as:

$$K_{bd} = \frac{1}{(1/K_b) + (1/K_d)} = \frac{K_d}{(1 + \frac{1}{B/D})} \tag{6}$$

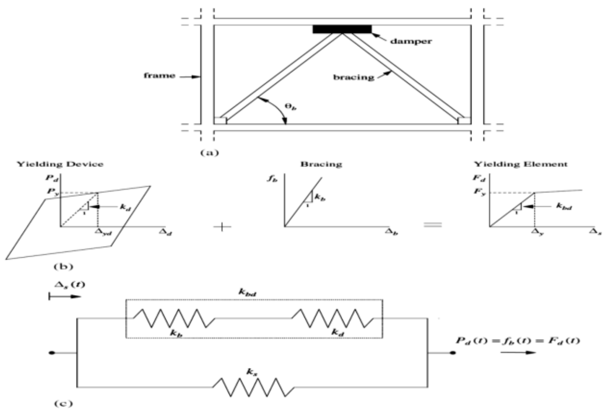


Fig. 4 Yielding metallic damper, (a) typical configuration, (b) yielding metallic device, bracing and yielding element parameters, (c) stiffness properties of device-bracing assembly

The bracing and the main structural members, in this study, are designed to remain elastic during an earthquake occurrence. And the stiffness coefficient K_d of the device used in E_q. 6 corresponds to the initial elastic values of yielding elements. E_q. 5 states the basic relationship between parameters of the proposed bilinear model. According to this equation, in a given structure (i.e. Ks), the behavior of a hysteretic element is governed by the four key parameters: yield load P_y, yield displacement of the device Δ_{yd}, stiffness ratios SR and B/D. Only three of these variables are independent as the fourth one could be determined by E_q. 5 (Moreschi and Singh, 2003).

The optimal values for damper 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. Here, the parameters SR and B/D are considered to take on any integer value varying 1 to 10. Based on experiments as well as previously proposed guidelines (Xia and Hanson, 1992; Xia, Hanson & Wight, 1990), admissible values considered for the device yield level varied between 0.0014H and 0.002H, Here, H is the height of structure.

To calculate the optimal usage of SR and B/D parameters, a Nonlinear Dynamic Analysis was performed on the models with strong ground motions. Fig. 5 shows the response spectra related to Tabas, Abhar and Elcentro earthquakes. The acceleration time histories were normalized to the maximum ground acceleration value of 0.35g. 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. For the results in Fig. 6 (that is an example for 3-story frame), only the SR parameter was considered to be an independent variable; the parameters of the yield level Δ_{yd} and B/D were fixed at and. For the results in Fig. 7 (that is an example for 3-story frame), only the B/D parameter was considered to be an independent variable; the parameters of the yield level Δ_{yd} and SR were fixed at and. As can be seen in Figures 6 & 7, increased SR and B/D parameters values in the range of 1 to 10, has had little impact in changing the roof displacement. According to these Figures, parameters B/D and SR were assumed at 2 and 3, respectively.

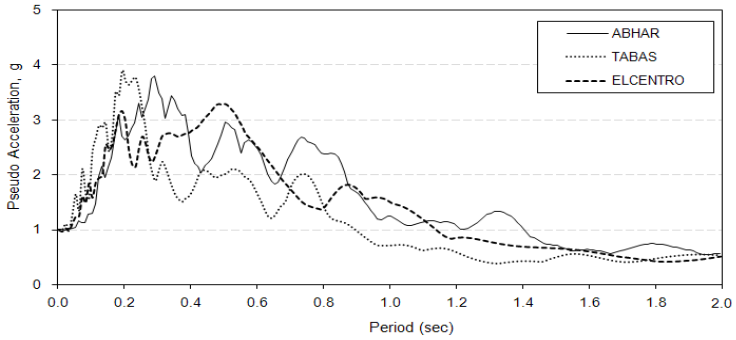


Fig. 5 Variation of response spectra with period of structure

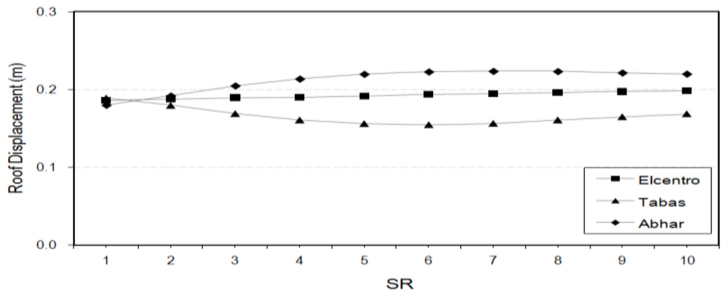


Fig. 6 Variation of roof displacement with SR parameter

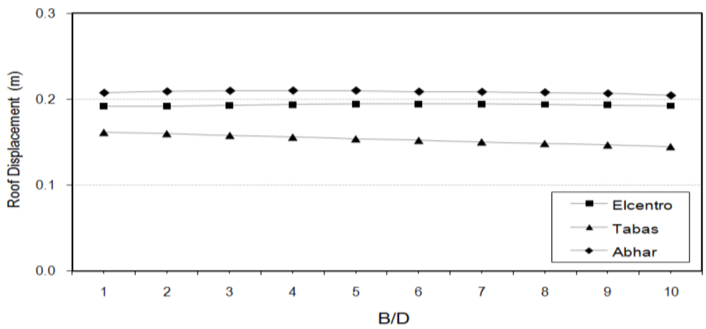
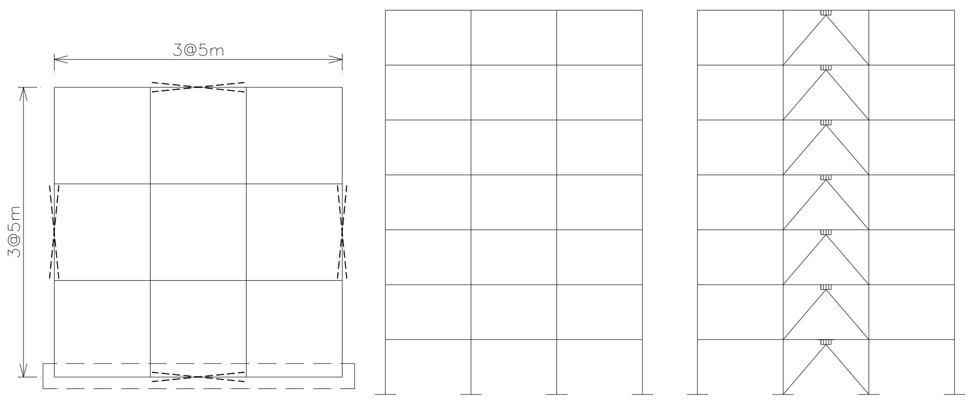


Fig. 7 Variation of roof displacement with B/D parameter

4 FRAME CHARACTERISTICS

To evaluate the performance of the TADAS devices in moderate and special moment resisting steel frames and compare their behavior to the same frames using TADAS dampers, twenty frames were used. All of the structures considered in this study have been designed according to the Iranian Earthquake Resistance Design Code (BHRC, 2005) and the Iranian National Building Code (MHUD, 2006). Fig. 8 shows the typical plan of structure and the chevron braces are equipped with TADAS dampers that located in the mid-bay of the perimeter frames. All these frames have spans of 5m with story heights of 3.2 m. The 3, 5, 7, 10 and 15 story buildings with TADAS devices and without them were used.



(a) Plan of the structures. (b) Main frames. (c) TADAS frames.

Fig. 8 Configuration of model structures

The dead and live loads of 6 and 2 kN/m2 were used for gravity load, respectively. For member design subjected to an earthquake, equivalent lateral static forces were applied on all the story levels. Further, these forces were calculated as per the provisions of the Iranian Earthquake Code (BHRC, 2005).

The base shear design was computed as:

$$V = CW \quad \rightarrow \quad C = \frac{ABI}{R} \tag{7}$$

Where, V, C and W are the base shear, the seismic coefficient and the equivalent weight of the structure, respectively. is the design spectral acceleration, expressed as the gravitational acceleration ratio, for soil type and the fundamental period of structure T. Further, I and R denote the importance factor and the response modification factor, respectively.

To design the models, importance factor , preliminary response modification factors , soil type III and seismic zone factor were considered. The beam-column joints were assumed to be moment resisting at both the ends. ST-37 steel was used for every structural member. Allowable stress design method was used to design frame members in accordance to part 10 of the Iranian National Code. To ensure that columns have enough strength to resist the earthquake force; the Iranian Standard No. 2800 (BHRC, 2005) has instructed to design vertical columns for following load combinations:

(a) Axial compression according to:

$$P_{DL} + 0.85P_{LL} + 2.8P_{E} < P_{SC} = 1.7F_{a}A \quad (8)$$

(b) Axial tension according to:

$$0.8P_{DL} + 2.8P_{E} < P_{ST} = F_{y}A \quad (9)$$

In which Fa is allowable compressive stress, Fy is the yield stress, A is column area. PDL, PLL and PE are axial load due to dead, live and earthquake loads, respectively. PST and PSC are design tensile and compression strength of the column, respectively (BHRC, 2005).

5 MODELING THE STRUCTURES

The computational model of structures was developed using the OpenSees software (Mazzoni et al, 2007). This software is finite element software which has been specifically designed in performance systems of soil and structure under earthquake. For modeling members in nonlinear range of deformation, the following assumptions were preferred:

The beam-column joints were assumed to be moment resisting at both the ends. Therefore, beams will behave as one of the parts of the lateral resisting system. For the dynamic analysis, story masses were placed at the story levels considering rigid diaphragms action. A damping coefficient of 5% was assumed. To model the St-37 steel behavior, ‘steel01’ bilinear kinematic stress-strain curve was assigned to the elements from the library of materials introduced in OpenSees. Considering the idealized elasto-plastic behavior of steel materials, compressive and tensional yield stresses were considered equal to steel yield stress. The strain hardening of 2% was considered for the member behavior in the inelastic range of deformation (Fig. 9). This was done, in part, for consistency purposes since this is the same approach taken by Asgarian and Moradi (Asgarian and Moradi, 2011). For modeling the TADAS elements, braces and nonlinear beam as well as columns were used with Steel01 material behavior. The used section for each member is uniaxial ones (Mazzoni et al, 2007).

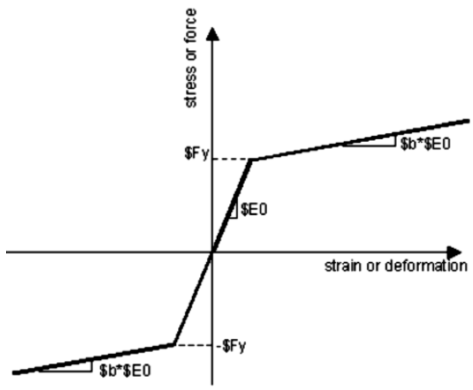


Fig. 9 Steel01 material for nonlinear elements

6 DYNAMIC ANALYSIS

The installation of TADAS hysteretic dampers in a structure would render it to behave nonlinearly even if all other structural members were designed to remain linear. Therefore, the analysis of structures installed with these devices must be done by a step-by-step time history analysis. In this paper, factors Rs and Rμ have been calculated through dynamic analysis:

6.1 Overstrength factor (Rs)

An incremental nonlinear dynamic analysis of the models subjected to strong ground motions, matched with the design spectrum was carried out to calculate Vy. Fig. 5 shows response spectrums of time history of Tabas, Abhar and Elcentro earthquakes. The details of these ground motion records can be seen in Table 1. In these analysis under above-mentioned time histories, their PGA's with several try and errors were changed in a way that the acquired time history resulted in the life safety (LS) structural performance level as well as the nonlinear behavior of elements as suggested by FEMA-356 (Fig. 10) (FEMA-356, 2000). The maximum nonlinear base shear of this time history is the inelastic base shear of the structure (Asgarian and Shokrgozar, 2009). Finally, the material overstrength factor of 1.155 was considered for the actual overstrength factor.

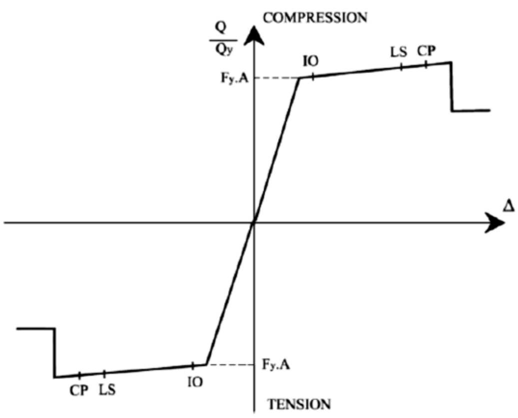


Fig. 10 Generalized force-deformation relation for steel elements (FEMA-356)

6.2 Ductility reduction factor (Rμ)

To calculate Rμ, nonlinear and linear dynamic analyses were taken into account. As mentioned, the nonlinear base shear Vy was calculated using the nonlinear dynamic analysis as well as try and error on PGA of time history of Tabas, Abhar and Elcentro earthquakes (Table 1). Further, applying the linear dynamic analysis of the structure under the same time history, the maximum linear base shear Ve was calculated and the ductility reduction factor was evaluated (Asgarian and Shokrgozar, 2009).

Table 1 Ground motion data

Earthquake	Year	PGA (g)	Duration (s)
Elcentro	1940	0.348	19.18
Tabas	1978	0.933	25.00
Abhar	1990	1.000	29.52

7 RESULTS

The incremental dynamic analysis results in terms of base shear-roof displacement for 5-story T-MMRF and T-SMRF have been shown in Figures 11 & 12. As can be seen, there is a good correlation between the responses of dynamic analysis by the use of time history of Tabas, Abhar and Elcentro earthquakes. The ultimate base shear Vy from nonlinear dynamic analysis under Elcentro, Abhar and Tabas occurrences for moderate and special ductility frames with and without TADAS dampers have been offered in Tables 2 & 3. Tables 4 & 5 show the maximum elastic base shear Ve, resulted from the linear dynamic analysis under above-mentioned time histories.

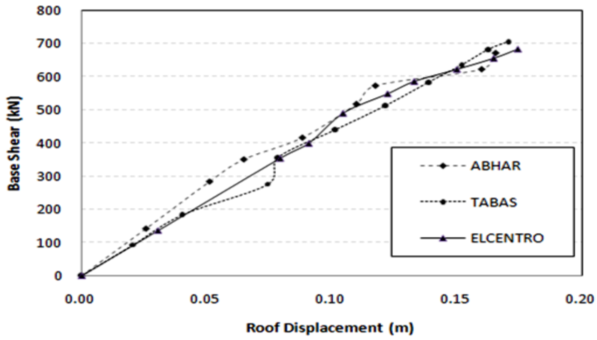


Fig. 11 Incremental dynamic roof displacement-base shear curve for 5 story T-MMRF

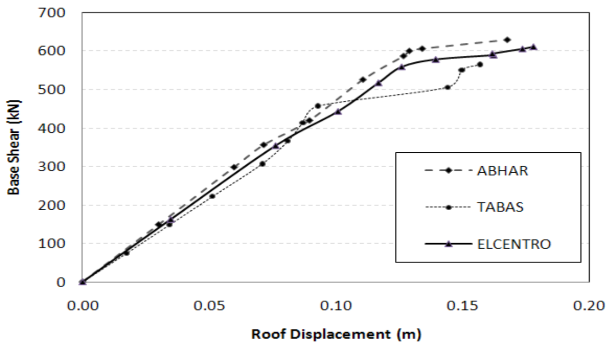


Fig. 12 Incremental dynamic roof displacement-base shear curve for 5 story T-SMRF

Table 2 Ultimate base shear Vy for MMRFs and SMRFs from nonlinear dynamic analysis

No. Story	MMRFs				SMRFs			
	V_y(avg.) (kN)	Abhar	Elcentro	Tabas	V_y(avg.) (kN)	Abhar	Elcentro	Tabas
3	311.9	300.7	274.9	360.0	327.3	300.3	326.5	355.2
5	329.4	339.1	311.6	337.6	307.0	355.4	271.0	294.7
7	340.8	277.1	326.3	419.1	317.2	278.1	316.6	356.9
10	481.6	487.6	526.5	430.7	412.8	362.4	462.4	413.6
15	550.7	567.2	494.9	589.9	444.3	466.5	403.2	463.2

Table 3 Ultimate base shear Vy for T-MMRFs and T-SMRFs from nonlinear dynamic analysis

No. Story	T-MMRFs				T-SMRFs			
	V_y(avg.) (kN)	Abhar	Elcentro	Tabas	V_y(avg.) (kN)	Abhar	Elcentro	Tabas
3	353.7	326.9	325.5	408.6	401.0	390.8	390.2	421.8
5	353.5	350.4	354.0	356.1	338.6	355.4	353.4	306.9
7	345.7	340.0	310.8	386.3	317.8	307.3	300.0	346.2
10	452.2	384.8	460.6	511.1	406.8	380.7	441.1	398.6
15	600.5	493.4	618.5	689.6	503.1	579.8	378.4	551.2

Table 4 Maximum elastic base shear Ve for MMRFs and SMRFs from linear dynamic analysis

No. Story	MMRFs				SMRFs			
	V_e(avg.) (kN)	Abhar	Elcentro	Tabas	V_e(avg.) (kN)	Abhar	Elcentro	Tabas
3	725.2	567.3	635.7	972.6	931.9	704.9	827.3	1263.5
5	1068.5	1327.2	1109.2	769.1	1018.4	1241.9	1105.6	707.6
7	1108.8	993.1	1261.0	1072.2	1173.2	1233.1	1300.7	985.9
10	1270.5	871.3	1761.6	1178.5	1307.6	1201.5	1783.6	937.7
15	2029.5	1432.9	1890.8	2764.9	1591.4	1559.1	1377.3	1837.9

Table 5 Maximum elastic base shear Ve for T-MMRFs and T-SMRFs from linear dynamic analysis

No. Story	T-MMRFs				T-SMRFs			
	V_e(avg.) (kN)	Abhar	Elcentro	Tabas	V_e(avg.) (kN)	Abhar	Elcentro	Tabas
3	1097.4	1355.4	742.1	1194.8	1273.9	1340.0	882.7	1599.1
5	1923.5	1775.2	1949.8	2045.4	2147.3	2213.2	2294.0	1934.8
7	2201.4	1850.0	2431.7	2322.5	2479.3	2397.2	2532.1	2508.6
10	2328.8	1541.4	3266.0	2178.8	2586.7	2383.9	3732.2	1644.1
15	2781.7	2580.0	3128.0	2637.0	2372.7	2659.6	2104.4	2354.0

Tables 6 & 7 show the incremental dynamic analysis results of overstrength, ductility and response modification factors for moderate and special moment resisting frames with TADAS devices and without them. It can be seen that the overstrength factors, ductility factors and response modification factors decrease as the height of building increases. Figures 13 & 14 show the variation in overstrength and ductility factors with respect to the incremental dynamic analysis results for difference type of ductility. It can be seen that the effect of the changes of ductility on ductility factor (R_{μ}) is more significantly than overstrength factor (Rs) and this effect is greater in the TADAS frames. Or in other words, the influence of TADAS dampers on the ductility factor is more than the overstrength factor. Figures 15 & 16 indicate the response modification factors of moderate and special moment resisting frames and their comparison with the TADAS frames. It could be seen that the influence of TADAS dampers on increase of response modification factor is noticeable hence; this influence in special moment resisting frames is greater than the others. Finally, it can be observed that the response modification factor decreases gradually with an increase of the height of building. This result was apparent in all type of frames with and without TADAS devices.

Table 6 Response modification factor of MMRFs and SMRFs

No. Story	MMRFs				SMRFs			
	R	R_{sd}	R_s	R_μ	R	R_{sd}	R_s	R_μ
3	7.61	2.83	3.27	2.32	13.44	4.09	4.72	2.85
5	6.57	1.76	2.03	3.24	8.77	2.29	2.64	3.32
7	5.60	1.49	1.72	3.25	7.93	1.86	2.14	3.70
10	5.26	1.73	1.99	2.64	7.75	2.12	2.45	3.17
15	6.72	1.58	1.82	3.69	7.14	1.73	1.99	3.58

Table 7 Response modification factor of T-MMRFs and T-SMRFs

No. Story	T-MMRFs				T-SMRFs			
	R	R_{sd}	R_s	R_μ	R	R_{sd}	R_s	R_μ
3	11.52	3.21	3.71	3.10	18.38	5.01	5.78	3.18
5	11.84	1.88	2.17	5.44	18.49	2.52	2.91	6.34
7	11.11	1.51	1.75	6.37	16.76	1.86	2.15	7.80
10	9.64	1.62	1.87	5.15	15.33	2.09	2.41	6.36
15	9.21	1.72	1.99	4.63	10.64	1.95	2.26	4.72

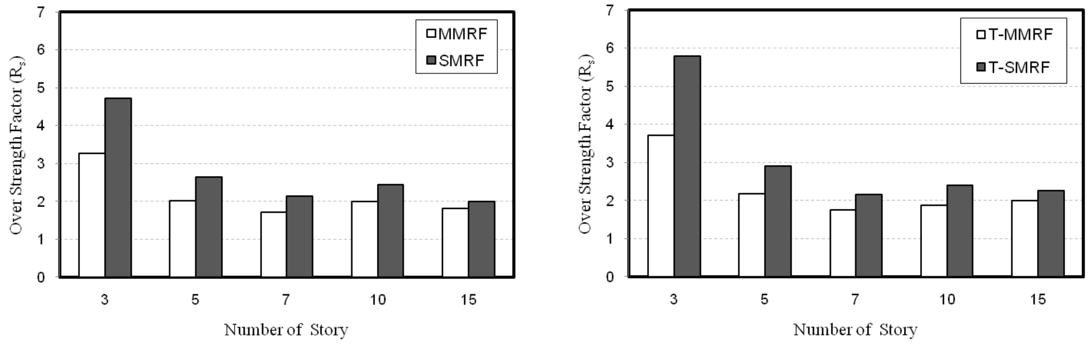


Fig. 13 Overstrength factors of structures from incremental dynamic analysis results

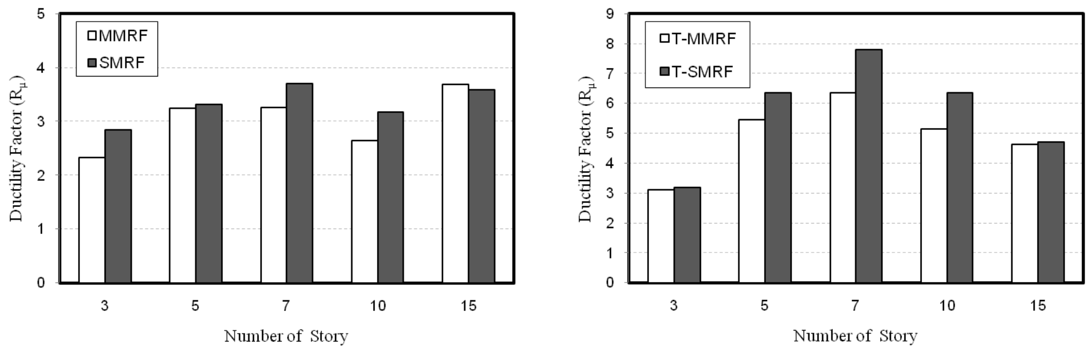


Fig. 14 Ductility factors of structures from incremental dynamic analysis results

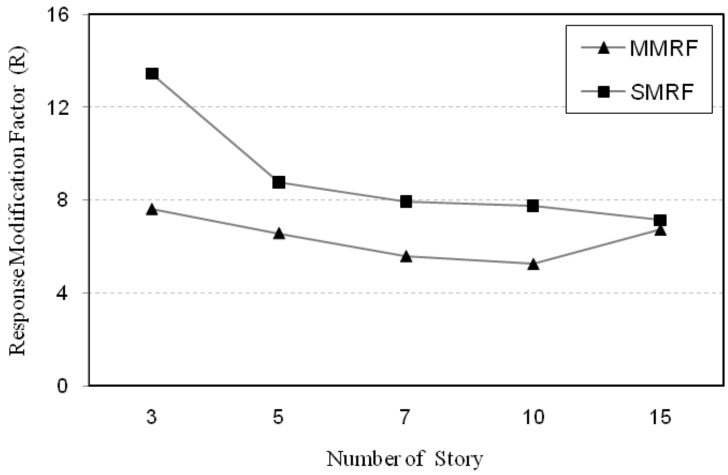


Fig. 15 Number of story-response modification factor for moderate and special moment resisting frames (MMRFs & SMRFs)

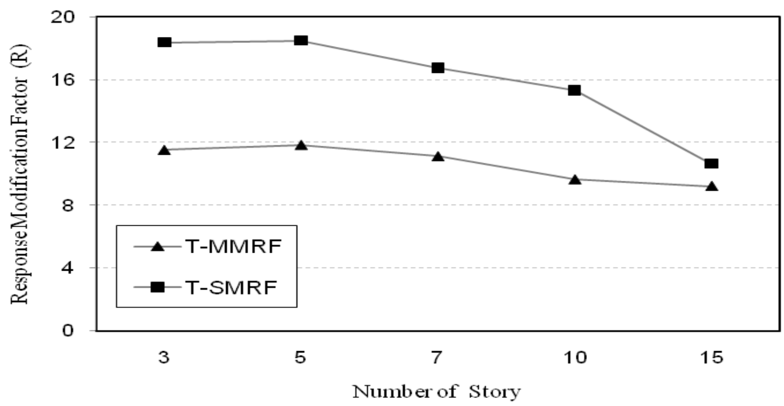


Fig. 16 Number of story-response modification factor for moderate and special moment resisting frames with TADAS devices (T-MMRFs & T-SMRFs)

8 CONCLUSION

This paper has evaluated the effect of the changes of ductility on the response modification factors for the frames are equipped with TADAS yielding dampers. The factors such as overstrength, reduction due to ductility, and the response modification of twenty frames considering life safety structural performance levels were evaluated. The overstrength, ductility and response modification factors of the moderate and special moment resisting frames with TADAS devices and without them considering life safety structural performance levels with various stories number have been studied by performing linear dynamic and incremental nonlinear dynamic analysis. The result of the proposed study can thus be summarized as follows:

1. It was observed that the response modification factors of the moderate and special moment resisting frames with TADAS devices and without them decrease with an increase in the height of buildings. However, the reduction factors due to ductility for these frames are different.

2. The type of structure ductility (moderate or special) and the height of the building have a low affect on overstrength factors of TADAS frames (Fig. 13).

3. The influence of TADAS dampers on increase of response modification factor for special moment resisting frames (SMRFs) is greater than the moderate moment resisting frames (MMRFs) noticeably (Fig. 16).

4. It is found that the most important factor determining R is the ductility, and R increase with increasing ductility and this effect is significant in the frames are equipped with TADAS yielding dampers (Figures 15 & 16 and Tables 6 & 7).

5. Codes present constant value of response modification factor for each of moderate and special moment resisting frames (MMRFs and SMRFs), however, the response modification factors evaluated in this paper have different values for the buildings with various stories.

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