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Evaluating response modification factors of TADAS frames

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ABSTRACT

The current paper tries to evaluate overstrength, ductility and response modification factors in special moment resisting frames with TADAS (triangular-plate added damping and stiffness) devices. For that matter, multi-story buildings were considered during the course of study. Further, OpenSees Software was applied to perform the static pushover analysis, the nonlinear incremental dynamic analysis as well as the linear dynamic analysis. In this research, seismic response modification factor for special moment resisting frames (SMRFs) with TADAS devices (T-SMRFs) and without them has been determined separately. The results showed that the response modification factors for T-SMRFs were higher than the SMRFs ones. It was also found that the number of stories of buildings has had greater effect on the response modification factors.

1. Introduction

The moment resisting frames are one of the most commonly used methods to resist lateral loads especially during an earthquake [1]. Recently, much emphasis has been put on developing various damping mechanisms in order to provide positive control of structural vibration in the wake of earthquakes. One of these mechanisms is hysteretic dampers which through their hysteresis 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 [2].

In fact, the energy input during an earthquake is relatively independent of restoring force characteristics of the structural system. This suggests that 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 dampers with elasto-plastic behavior, Tsai et al. [3] 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 consider a reduction in design loads, taking advantage of the fact that the structures possess significant reserve strength and capacity to dissipate energy which called overstrength and ductility respectively. These two factors are incorporated in structural design through a force reduction or a response modification factor. This factor represents ratio of maximum seismic force on a structure during specified ground motion if it was to remain elastic to the design seismic force. 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 [1].

The response modification factors were first proposed in ATC3-06 [4]. The product of three factors i.e. Overstrength, Ductility, and Redundancy were calculated in ATC-19 [5] and ATC-34 [6]. The response modification factor for special moment resisting frames with TADAS devices (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) and special moment resisting frames (SMRFs). The present study focuses on overstrength evaluation, force reduction due to ductility and response modification factors of SMRFs and T-SMRFs. These were designed in accordance with the Iranian Earthquake Resistant Design Codes [7] (BHRC, 2005) and the Iranian National Building Code (part 10) for Structural Steel Design [8].

To obtain the proposed factors, nonlinear static analyses, nonlinear incremental dynamic analysis and linear dynamic analysis were carried out.

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2. TADAS hysteretic dampers

2.1. 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 [9,10]. 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.

Fig. 1 highlights the resistance behavior of a structural system with a TADAS damper. As shown, the system consists of a main frame (serving primarily as gravity force) and a hysteretic damping mechanism, with both components being linked parallel [11]. Again, Fig. 2 shows the cyclic force versus deformation relationship of a typical TADAS device, details of which are given in Fig. 3. 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 [12]. The mechanical property of a TADAS device is highly predictable and has been documented in the reference section [3,13].

2.2. 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 [14]. 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

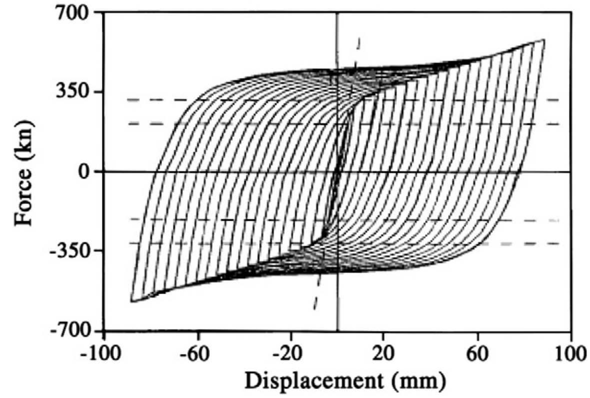

Fig. 2. A typical hysteretic loop for a TADAS device [12].

where device is installed, the yield force P_y can be related to the device parameters as follows [14]:

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

where $K_{s,}$ Δ_{yd} and $B/D = K_b/K_d$ are story stiffness, yield displacement and the ratio of the brace stiffness K_b to the device stiffness K_d, respectively. $SR = K_{bd}/K_s$ is the ratio of the assembly stiffness K_{bd} to story stiffness K_s. Fig. 4(b) schematically shows the combined stiffness of device-brace assembly and in-terms of device and bracing stiffness K_d and K_b can be expressed as:

$$K_{bd} = \frac{1}{(1/K_b) + (1/K_d)} = \frac{K_d}{\left(1 + \frac{1}{B/D}\right)} \tag{2}$$

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 Eq. (2) corresponds to the initial elastic values of yielding elements. Eq. (1) states the basic relationship between parameters of the proposed bilinear model. According to this equation, in a given structure (i.e. K_s), the behavior of a hysteretic element is governed by the four key parameters: yield load P_y, yield displacement of the device Δ_{yd}, and stiffness ratios SR and B/D. Only three of these variables are independent as the fourth one could be determined by Eq. (1) [14].

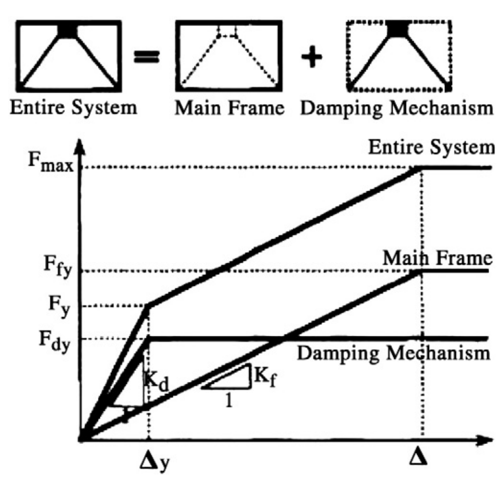

Fig. 1. Schematic hysteretic behavior of structure with a hysteretic damper [11].

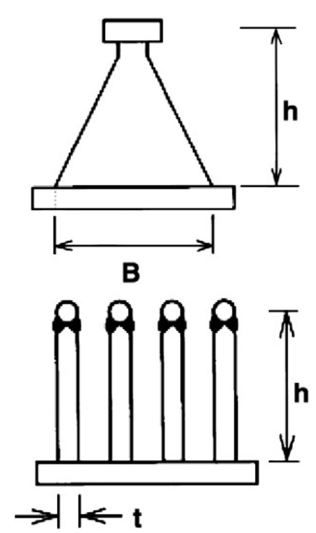

Fig. 3. The details for a TADAS device [12].

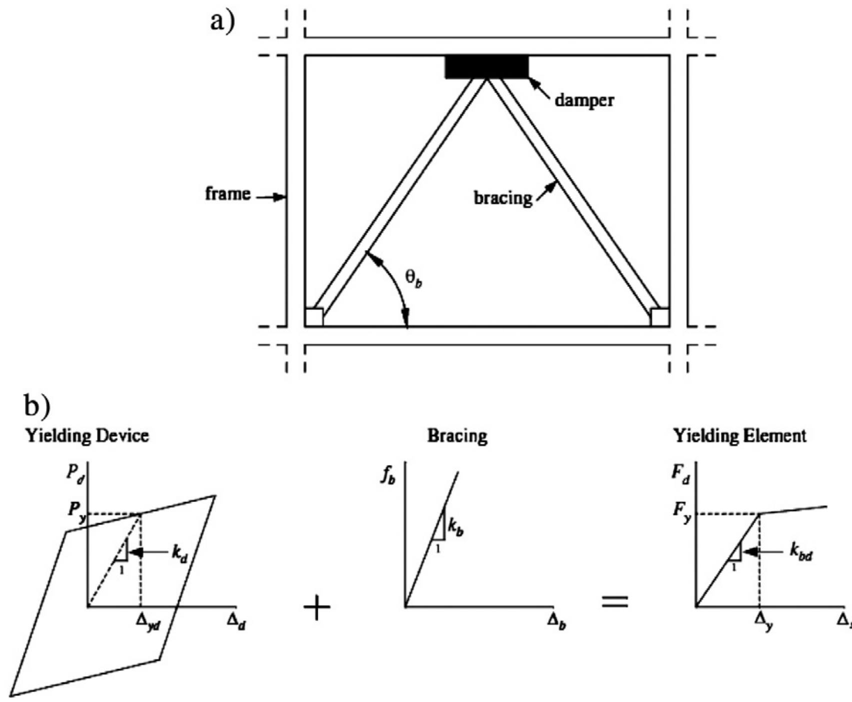

Fig. 4. Yielding metallic damper, (a) typical configuration, and (b) yielding metallic device, bracing and yielding element parameters [14].

3. Response modification factor

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 [1].

While computing the response modification factor, Mazzolani and Piluso [15] addressed several theoretical aspects such as the maximum plastic deformation, energy and low cycle fatigue approaches. As shown in Fig. 5, usually the real nonlinear behavior is idealized by a bilinear elasto-plastic relation [16]. Here, the yield force and the yield displacement of structure are shown by V_y and Δ_y, respectively. In this figure, V_e (V_{max}) corresponds to the elastic response strength of the structure [1]. Consequently, the response modification factor is determined as follows [17]:

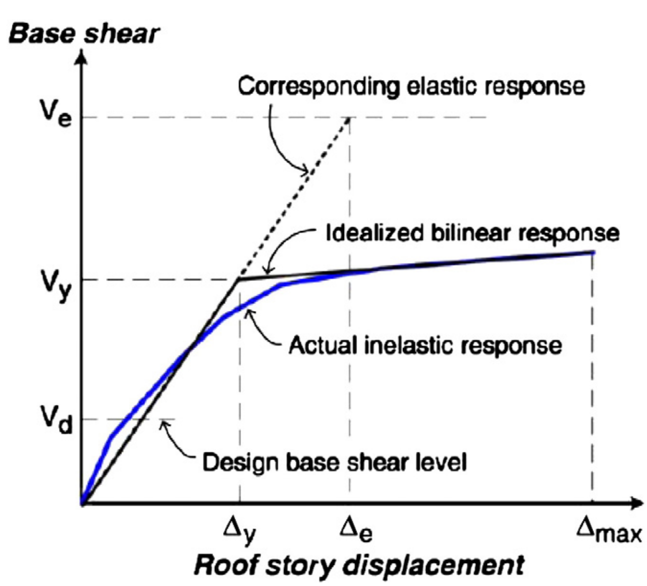

Fig. 5. Lateral load-roof displacement relationship of structure [16].

$$R = R_\mu . R_S \tag{3}$$

where, R_μ is a reduction factor due to ductility and R_s is the overstrength factor.

3.1. 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 :

$$R_\mu = \frac{V_e}{V_y} \tag{4}$$

Several proposals have been put forward for R_μ. In a simple version proposed by Fajfar [18], the reduction factor is written as:

$$\begin{aligned} R_\mu &= (\mu - 1)\frac{T}{T_C} + 1 \; (T < T_C) \\ R_\mu &= \mu \; (T \geq T_C) \end{aligned} \tag{5}$$

Where, T is the fundamental period, T_C is the characteristic of ground motion equal to 0.7 for the soil type III as considered in the proposed study based on the Iranian Earthquake Resistance Design Code (Standard No. 2800) [7] and μ is the structural ductility factor defined as:

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{6}$$

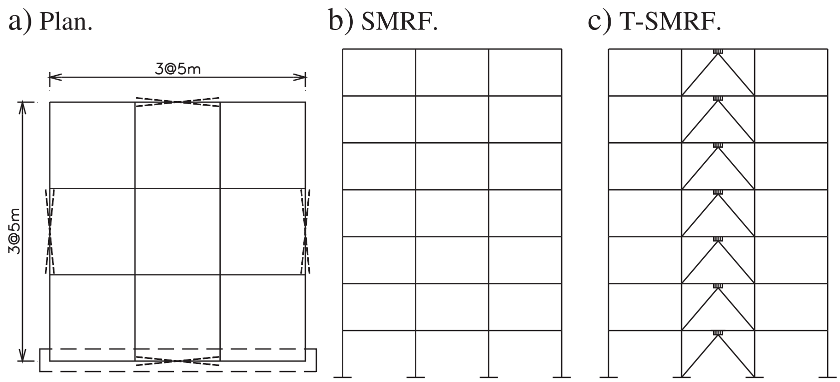

Fig. 6. Configuration of model structures.

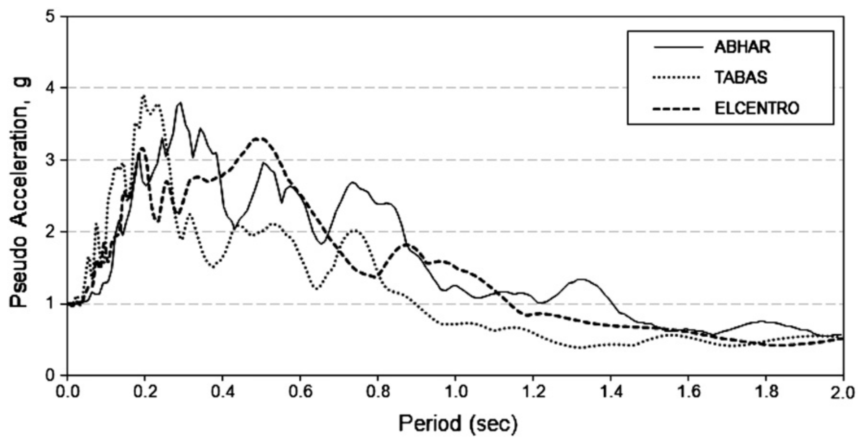

Fig. 7. Variation of response spectra with period of structure.

where, Δ_{max} is the maximum displacement for the first life safety performance in structure and Δ_y is the yield displacement observed there.

3.2. 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 [17]. 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 [19]. Here, the design overstrength factor (R_{sd}) is defined [17] as:

$$R_{Sd} = \frac{V_y}{V_d} \tag{7}$$

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 performance (Fig. 5). Overstrength, redundancy and ductility, the three concepts used to scale down the earthquake forces need to be defined and expressed clearly in quantifiable terms.

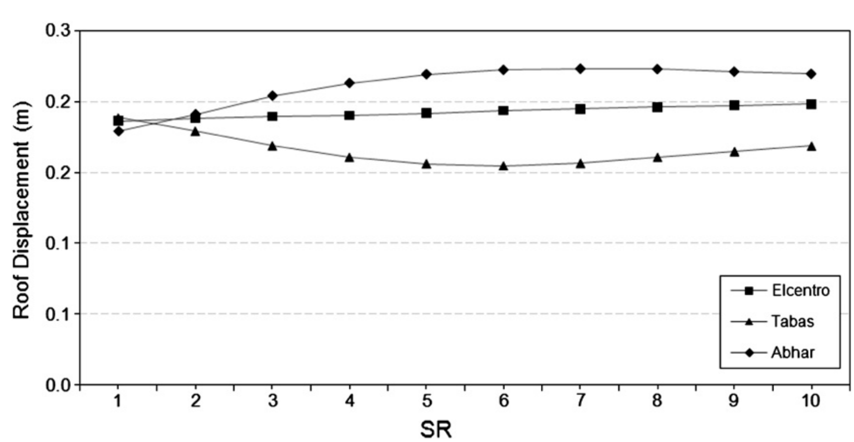

Fig. 8. Variation of roof displacement with SR parameter.

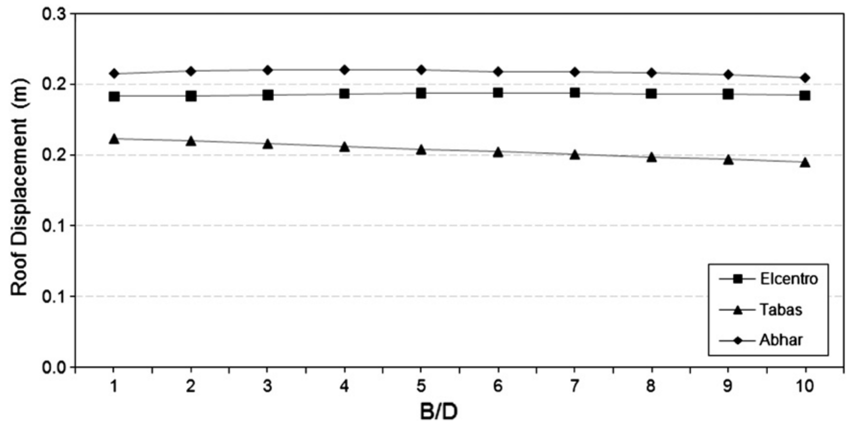

Fig. 9. Variation of roof displacement with B/D parameter.

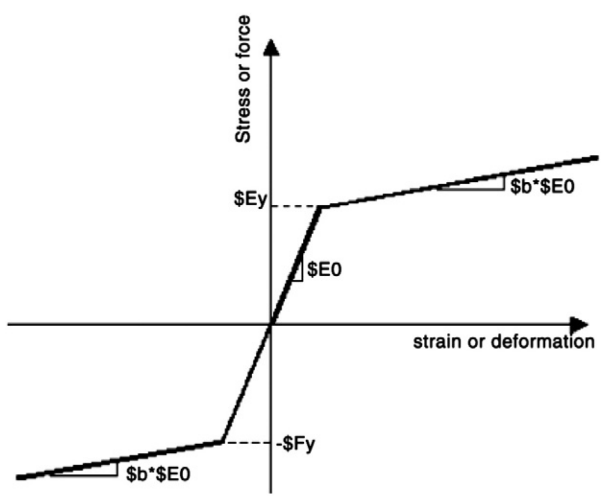

Fig. 10. Steel01 material for nonlinear elements [23].

Although, the overstrength factor is based on the applied nominal material properties, the actual overstrength factor should consider the help of some other effects [1]:

$R_S = R_{Sd}.F_1.F_2 \ldots F_n$. (8) 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 [20]. 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 [1].

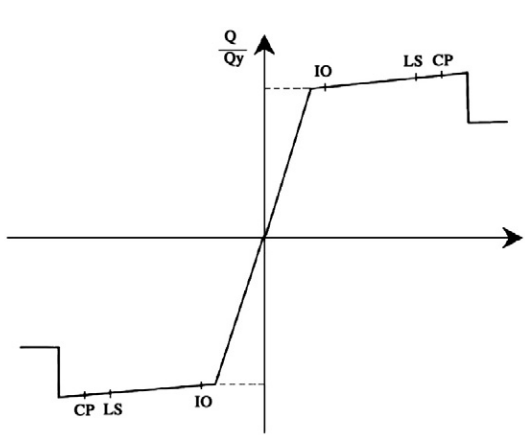

Fig. 11. Generalized force-deformation relation for steel elements (FEMA-356) [24].

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 [1].

4. Design of model structures

4.1. Structural models

To evaluate the overstrength, ductility, and response modification factors of special moment resisting frames with TADAS devices (T-SMRFs) 3, 5, 7, 10 and 15-story buildings with the bay length of 5 m were designed as per the requirement of the Iranian Earthquake Resistance Design Code [7] and the Iranian National Building Code [8]. Fig. 6a and b shows the typical configuration of models used during the present study. The story height of the models was considered as 3.2 m. 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 (Standard No. 2800) [7]. The dead and live loads of 6 and 2 kN/m^2 were used, respectively.

The base shear design was computed as:

$$V = CW \rightarrow C = \frac{ABI}{R} \tag{9}$$

where, V, C and W are the base shear, the seismic coefficient and the equivalent weight of the structure, respectively. $A \times B$is the design spectral acceleration, expressed as the gravitational acceleration ratio,

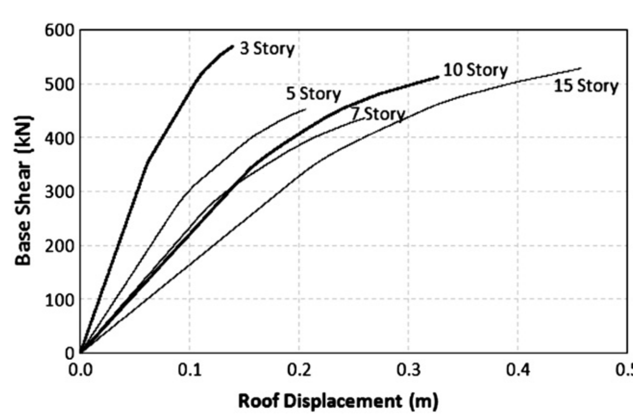

Fig. 12. Roof displacement-base shear curve for SMRFs.

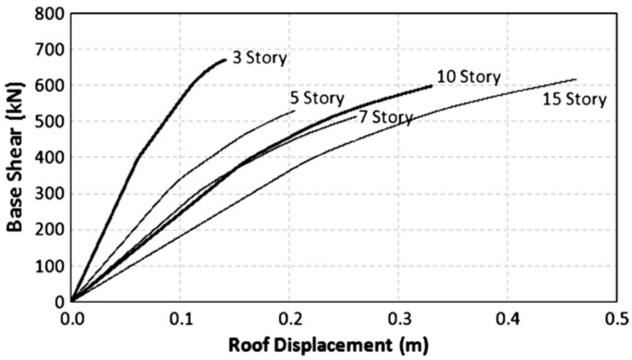

Fig. 13. Roof displacement-base shear curve for T-SMRFs.

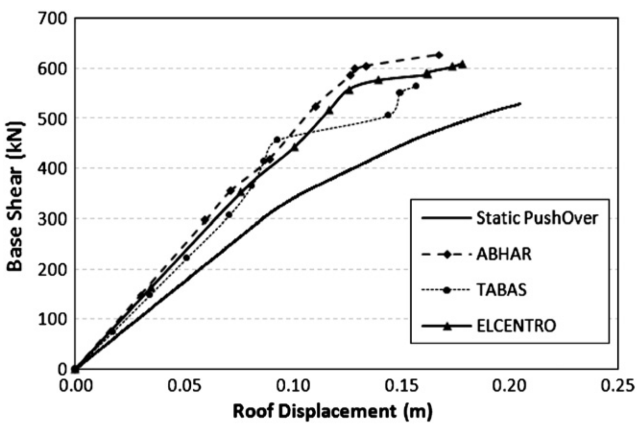

Fig. 14. Comparison of incremental dynamic and static pushover roof displacement-base shear curve, 5 story T-SMRFs.

Table 1
Ultimate base shear V_y from nonlinear dynamic analysis under Elcentro, Abhar and Tabas ground motion.

`				SMRFs				No. Story
V_y (avg.) (kN)	Tabas	Elcentro	Abhar	V_y (avg.) (kN)	Tabas	Elcentro	Abhar	
401.0	421.8	390.2	390.8	327.3	355.2	326.5	300.3	3
338.6	306.9	353.4	355.4	307.0	294.7	271.0	355.4	5
317.8	346.2	300.0	307.3	317.2	356.9	316.6	278.1	7
406.8	398.6	441.1	380.7	412.8	413.6	462.4	362.4	10
503.1	551.2	378.4	579.8	444.3	463.2	403.2	466.5	15

for soil type and the fundamental period of structure T. Further, *I* and R denote the importance factor and the response modification factor, respectively.

Table 2
Maximum elastic base shear V_e from linear dynamic analysis under Elcentro, Abhar and Tabas ground motion.

T-SMRFs				SMRFs				No. Story
V_e (avg.) (kN)	Tabas	Elcentro	Abhar	V_e (avg.) (kN)	Tabas	Elcentro	Abhar	
1273.9	1599.1	882.7	1340.0	931.9	1263.5	827.3	704.9	3
2147.3	1934.8	2294.0	2213.2	1018.4	707.6	1105.6	1241.9	5
2479.3	2508.6	2532.1	2397.2	1173.2	985.9	1300.7	1233.1	7
2586.7	1644.1	3732.2	2383.9	1307.6	937.7	1783.6	1201.5	10
2372.7	2354.0	2104.4	2659.6	1591.4	1837.9	1377.3	1559.1	15

Table 3
Response modification factor of SMRFs from incremental dynamic analysis.

No. Story	R_{sd}	R_s	R_μ	R
3	4.09	4.72	2.85	13.44
5	2.29	2.64	3.32	8.77
7	1.86	2.14	3.70	7.93
10	2.12	2.45	3.17	7.75
15	1.73	1.99	3.58	7.14

Table 4
Response modification factor of T-SMRFs from incremental dynamic analysis.

No. Story	R_{sd}	R_s	R_μ	R
3	5.01	5.78	3.18	18.38
5	2.52	2.91	6.34	18.49
7	1.86	2.15	7.80	16.76
10	2.09	2.41	6.36	15.33
15	1.95	2.26	4.72	10.64

To design the models, importance factor$I=1$, preliminary response modification factors$R=10$, soil type III and seismic zone factor $A=0.35$ were considered. The beam-column joints were assumed to be moment resisting at both the ends. 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 [7] has instructed to design vertical columns for following load combinations:

(a) Axial compression according to:

$$P_{DL}+0.85P_{LL}+2.8P_E<P_{SC}=1.7F_aA \tag{10}$$

(b) Axial tension according to:

$$0.8P_{DL}+2.8P_E<P_{ST}=F_yA \tag{11}$$

In which F_a is allowable compressive stress, F_y is the yield stress, A is column area. P_{DL}, P_{LL} and P_E are axial load due to dead, live and earthquake loads, respectively. P_{ST} and P_{SC} are design tensile and compression strength of the column, respectively [7].

4.2. TADAS design parameters

As mentioned, there are three independent parameters that need to be calculated for the optimal usage of the TADAS dampers. Herein,

Table 5
Response modification factor of SMRFs nonlinear static analysis.

No. Story	R_{sd}	R_s	R_μ	R
3	5.16	5.96	1.61	9.57
5	2.52	2.91	1.85	5.39
7	1.89	2.18	2.13	4.65
10	2.05	2.37	2.19	5.19
15	1.59	1.83	2.69	4.94

Table 6
Response modification factor of T-SMRFs from nonlinear static analysis.

No. Story	R_{sd}	R_s	R_μ	R
3	5.92	6.84	1.59	10.84
5	2.72	3.15	1.92	6.05
7	2.10	2.43	2.08	5.05
10	2.35	2.72	2.24	6.08
15	1.79	2.06	2.78	5.73

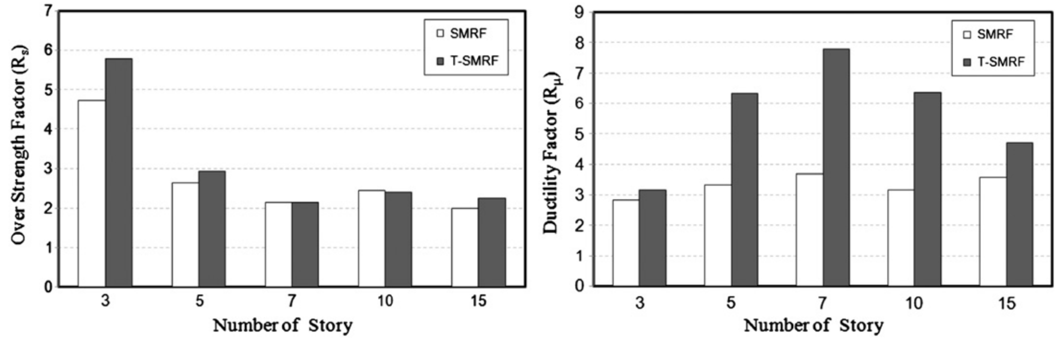

Fig. 15. Overstrength factor and ductility factor of structures from incremental dynamic analysis.

stiffness ratio SR, yield level Δ_{yd}, and stiffness ratio B/D are chosen as the design parameters [14].

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 [21,22], 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. 7 shows the response spectra related to Tabas, Abhar and Elcentro earthquakes. The acceleration time histories were normalized to the maximum ground acceleration value of 0.35 g. 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. 8 (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 $\Delta_{yd}=0.05m$ and $B/D=2$. For the results in Fig. 9, the SR parameter was fixed at $SR=3$. According to Figs. 8 and 9, parameters B/D and SR were assumed at 2 and 3, respectively.

5. Modeling of structure with OpenSees software

The computational model of structures was developed using the OpenSees software [23]. This software has finite element specifically designed for soil and structures exposed to earthquake. For modeling members in nonlinear range of deformation, following assumptions were preferred:

The beam-column joints were assumed to be moment resisting at both the ends. For the dynamic analysis, story masses were placed at the story levels considering rigid diaphragms action. For modeling the TADAS elements, braces and nonlinear beam as well as columns were used with Steel01 material behavior. Considering the idealized elasto-plastic behavior of steel material, compressive and tensional yield stresses were considered equal to the steel yield stress. The used section for each member is uniaxial ones. The strain hardening of 2% was considered for the member behavior in inelastic range of deformation (Fig. 10) [23].

6. Pushover analysis

To evaluate the behavior factors of the structures, the nonlinear static (pushover) analysis was performed by subjecting a structure to monotonically increasing lateral forces with an invariant height-wise distribution and for this purpose, OpenSees program was used. Indeed, in the pushover analysis, selecting an appropriate lateral load distribution is an important step [18]. As such, the analysis was conducted using life safety structural performance level as well as the nonlinear behavior of elements as suggested by FEMA-356 (Fig. 11) [24]. The post-yield stiffness of beams, columns and TADAS elements was initially assumed to be 2%. In Fig. 11, Q, Q_y and ⊖ are the generalized component load, expected strength and component rotation, respectively. The structural performance level means the post-earthquake damage status in which, significant damage occur but there remains some margin of partial or total structural collapse. In other words, structural elements and components are severely damaged due tremor, but still it has not resulted in large falling of

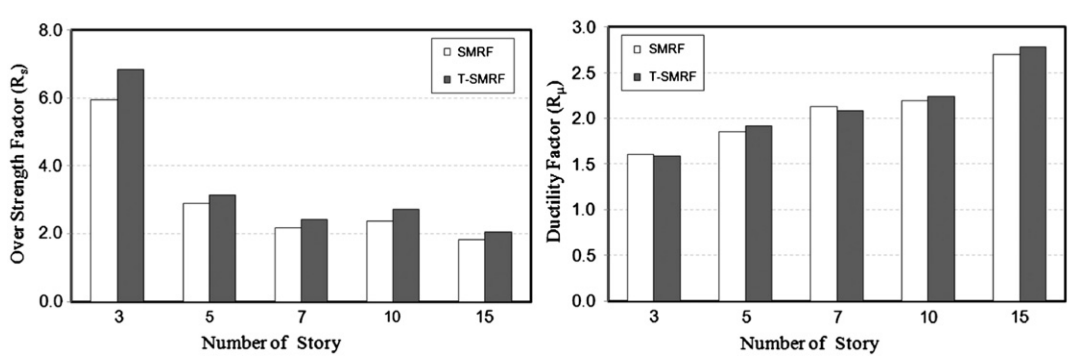

Fig. 16. Overstrength factor and ductility factor of structures from nonlinear static pushover analysis.

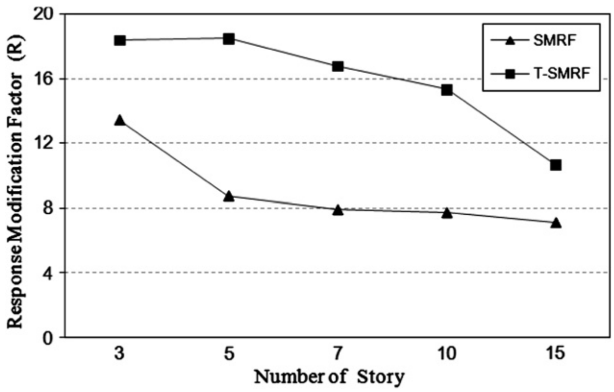

Fig. 17. Number of story-response modification factor from incremental dynamic analysis.

debris, within or outside the building. Injuries might occur but the overall risk of life-threatening injury seems to be low. It could possibly repair the structure but may not be reasonable from economic point of view [24].

7. Dynamic analysis

In this paper, factors R_s and R_μ have been calculated through dynamic analysis:

7.1. Overstrength factor (R_s)

An incremental nonlinear dynamic analysis of the models subjected to strong ground motions, matched with the design spectrum was carried out to calculate V_y. Fig. 7 shows response spectrums of time history of Tabas, Abhar and Elcentro earthquakes. 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 structural performance level as well as the nonlinear behavior of elements as suggested by FEMA-356 (Fig. 11) [24]. The maximum nonlinear base shear of this time history is the inelastic base shear of the structure [1]. Finally, the material overstrength factor of 1.155 was considered for the actual overstrength factor.

7.2. Ductility reduction factor (R_μ)

To calculate R_μ, nonlinear and linear dynamic analyses were taken into account. As mentioned, the nonlinear base shear V_y was

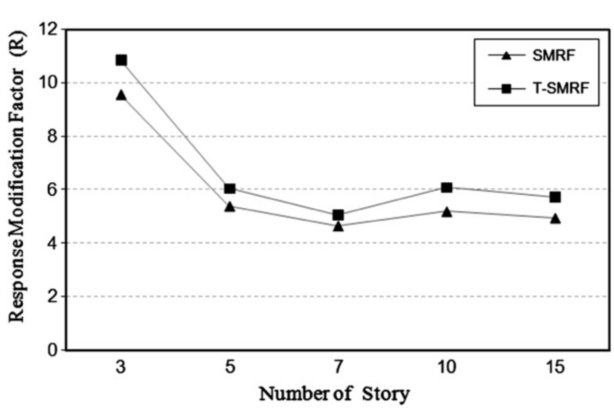

Fig. 18. Number of story-response modification factor from nonlinear static analysis.

calculated using the nonlinear dynamic analysis as well as try and error on PGA of earthquake time histories. Further, applying the linear dynamic analysis of the structure under the same time history, the maximum linear base shear V_e was calculated and the ductility reduction factor was evaluated [1].

8. Results

The nonlinear static analysis results in terms of base shear-roof displacement for SMRFs and T-SMRFs have been shown in Figs. 12 and 13. Fig. 14 indicates the incremental dynamic analysis results and their comparison with the static pushover curve in terms of roof displacement-base shear for 5-story T-SMRF. Table 1 highlights the ultimate base shear V_y from nonlinear dynamic analysis under Elcentro, Abhar and Tabas occurrences for SMRFs and T-SMRFs.

Table 2 shows the maximum elastic base shear V_e, resulted from the linear dynamic analysis under above-mentioned time histories. Tables 3 and 4 show the incremental dynamic analysis results of overstrength, ductility and response modification factors. Finally, Tables 5 and 6 give nonlinear static analysis results of overstrength, ductility and response modification factors. It can be seen that these three factors decrease with an increase in the height of building.

Fig. 15 shows the variation in overstrength and ductility factors with respect to the incremental dynamic analysis results of SMRFs and T-SMRFs. It is found that the influence of TADAS dampers on the ductility factor is more than the overstrength factor hence; this influence in middle stories is greater than the others. Fig. 16 shows variation of overstrength and ductility factor by nonlinear static analysis results of SMRFs and T-SMRFs. It could be seen that influence of TADAS dampers on increase of ductility factor by performing pushover analysis is low. The two above-mentioned analyses for the response modification factor have been presented in Figs. 17 and 18. It is found that the response modification factor decreases gradually with an increase of the height of building.

9. Conclusion

In sum, the current research could evaluate the overstrength, ductility and response modification factors of SMRFs and T-SMRFs with various stories with the help of static pushover analysis, linear dynamic analysis as well as incremental nonlinear dynamic analysis. The result of the proposed study can thus be summarized as follows:

(1) The obtained the overstrength factor for special moment resisting frames with TADAS devices (T-SMRFs) is 3.1.
(2) The obtained ductility factor for special moment resisting frames with TADAS devices (T-SMRFs) is 5.68.
(3) The response modification factor for special moment resisting frames with TADAS devices (T-SMRFs) is 15.92.
(4) Both overstrength and ductility factors are decreased as the number of stories is increased.

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