INFLUENCE OF THE TADAS DAMPERS ON THE DUCTILITY REDUCTION FACTOR OF STEEL FRAMES

Mussa Mahmoudi¹, Mohammad Ghasem Abdi² and Farid Mahmoudi³

ABSTRACT

Seismic design codes consider a reduction in design loads, taking advantage of the fact that the 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. In this paper, ductility reduction factors of moderate moment resisting frames with TADAS (triangular-plate added damping and stiffness) hysteretic dampers were evaluated. For that matter, buildings with various stories were considered. The nonlinear incremental dynamic analysis and linear dynamic analysis have been performed using OpenSees software. In this article, ductility reduction factors for moderate moment resisting frames with TADAS devices (T-MMRFs) and without them have been determined separately. The results showed that the ductility reduction factors for T-MMRFs were higher than the MMRFs ones. It was also found that the number of stories of buildings has had greater effect on the ductility reduction factors.

INTRODUCTION

The moment resisting frames are one of the most commonly used methods to resist lateral loads especially during an earthquake (Asgarian and Shokrgozar, 2009). 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 (Inoues 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 dampers with elastic-plastic behavior, 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

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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 (1995) and ATC-34 (ATC-34, 1995). The response modification factor for moderate moment resisting frames with TADAS devices (T-MMRFs) 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 moderate moment resisting frames (MMRFs). The present study focuses on ductility reduction factors of MMRFs and T-MMRFs. These were designed in accordance with the Iranian Earthquake Resistant Design Codes BHRC (2005) and the Iranian National Building Code (part 10) for Structural Steel Design (MHUD, 2006). To obtain the proposed factors, nonlinear incremental dynamic analysis and linear dynamic analysis were carried out.

TADAS HYSTERETIC DAMPERS

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 (Aiken et al., 1993 and Hanson 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. 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 (Nakashima et al., 1996).

![Figure 1. Schematic hysteretic behavior of structure with a hysteretic damper (Nakashima et al., 1996)](image)

Again, Fig. 2 shows the cyclic force versus deformation relationship and details of a typical TADAS device. 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 et al., 1998). The mechanical property of a TADAS device is highly predictable and has been documented in the reference section (Tsai et al., 1993 and Tsai et al., 1995).
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. 3(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):

\[
P_Y = K_d \Delta y_d = SRK_s \left(1 + \frac{1}{B/D}\right) \Delta y_d
\]

Where \(K_s\), \(\Delta y_d\) 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. \(SR = Kbd/K_s\) is the ratio of the assembly stiffness \(Kbd\) to story stiffness \(K_s\). Fig. 3(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:

\[
K_{bd} = \frac{1}{\left(\frac{1}{K_b}\right) + \left(\frac{1}{K_d}\right)} = \frac{K_d}{\left(1 + \frac{1}{B/D}\right)}
\]

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 \(\Delta y_d\), and stiffness ratios SR and B/D. Only three of these variables are independent as the fourth one could be determined by Eq.(1). (Moreschi and Singh, 2003).
The elastic analysis of structures exposed to earthquake could create base-shear force and stresses which noticeably are bigger than the real structural response. Seismic codes reduce design loads, taking advantage of the fact that structures possess overstrength and ductility (Asgarian and Shokrgozar, 2009). As shown in Fig. 4, usually the real nonlinear behavior is idealized by a bilinear elasto-plastic relation (Kim and Choi, 2005). Here, the yield force and the yield displacement of structure are shown by \( V_y \) and \( \Delta y \), respectively. In this figure, \( V_e \) (\( V_{\text{max}} \)) corresponds to the elastic response strength of the structure (Asgarian and Shokrgozar, 2009). The reduction factor due to ductility (\( R_\mu \)) 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 \( V_y \):

\[
R_\mu = \frac{V_e}{V_y}
\]

Several proposals have been put forward for \( R_\mu \). In a simple version proposed by Fajfar (2002), the reduction factor is written as:

\[
R_\mu = \begin{cases} 
(\mu-1) \frac{T}{T_c} + 1 & (T < T_c) \\
\mu & (T \geq T_c)
\end{cases}
\]

(4)

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) BHRC (2005), and \( \mu \) is the structural ductility factor defined as:
\[ \mu = \frac{\Delta_{\text{max}}}{\Delta_y} \]  

(5)

Where, \( \Delta_{\text{max}} \) and \( \Delta_y \) are the maximum displacement and the observed yield displacement, respectively.

Figure 4. Lateral load-roof displacement relationship of structure (Asgarian and Shokrgozar, 2009)

**DESIGN OF MODEL STRUCTURES**

To evaluate the ductility reduction factors of moderate moment resisting frames with TADAS devices (T-MMRFs) 3, 5, 7, 10 and 15-story buildings with the bay length of 5m were designed as per the requirement of the Iranian Earthquake Resistance Design Code BHRC (2005), and the Iranian National Building Code (MHUD, 2006). Fig. 5 shows the typical configuration of models used during the present study. The height of structural 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 (BHRC, 2005). The dead and live loads of 6 and 2 kN/m² were used, respectively. The base shear design was computed as:

\[ V = CW \implies C = \frac{A\, I \, B}{R} \]  

(6)

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, 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 \( I = 1 \), preliminary response modification factors \( R = 7 \), 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 BHRC (2005), has instructed to design vertical columns for following load combinations:
(a) Axial compression according to:

\[ P_{DL} + 0.85P_{LL} + 2.8P_E < P_{SC} = 1.7F_a A \]  

(b) Axial tension according to:

\[ 0.8P_{DL} + 2.8P_E < P_{ST} = F_y A \]

In which \( F_a \) is allowable compressive stress, \( F_y \) is the yield stress, \( A \) is column area. \( P_{DL}, P_{LL} \) and \( P_{PE} \) 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 (BHRC, 2005).

Figure 5. Configuration of model structures

TADAS DESIGN PARAMETERS

As mentioned, there are three independent parameters that need to be calculated for the optimal usage of the TADAS dampers. Herein, stiffness ratio \( SR \), yield level \( \Delta yd \), and stiffness ratio \( B/D \) are chosen as the design parameters (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 et al. 1990) and Xia and Hanson (1992), admissible values considered for the device yield level varied between 0.0014\( H \) and 0.002\( H \), where \( H \) 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. 6 shows the response spectra related to Tabas, Abhar and Elcentro earthquakes. The acceleration time records 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 hence; it is assumed that there could be one device in each story. For the results in Fig. 7 (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 \( \Delta yd \) and \( B/D \) were fixed at \( \Delta yd = 0.05m \) and \( B/D = 2 \). For the results in Fig. 8, the \( SR \) parameter was fixed at \( SR = 3 \). According to the Figs. 4.3 and 4.4, parameters \( B/D \) and \( SR \) were assumed at 2 and 3, respectively.
Figure 6. Variation of response spectra with period of structure

Figure 7. Variation of roof displacement with SR parameter

Figure 8. Variation of roof displacement with B/D parameter
MODELING OF STRUCTURE WITH OPENSEES SOFTWARE

The computational model of structures was developed using the OpenSees software (Mazzoni et al., 2007). 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. 9).

![Steel01 material for nonlinear elements (Mazzoni et al., 2007)](image)

Figure 9. Steel01 material for nonlinear elements (Mazzoni et al., 2007)

DYNAMIC ANALYSIS

To calculate $R_\mu$, An incremental nonlinear dynamic analysis of the models subjected to strong ground motions, matched with the design spectrum was carried out to calculate $V_y$. As noted, Fig. 6 shows response spectrums of time history of Tabas, Abhar and Elcentro earthquakes. While analyzing the above-mentioned time histories, their PGA's 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. 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). Consequently, an OpenSees software was used. The post-yield stiffness of beams, columns and TADAS elements was initially assumed to be 2%. In Fig. 10, $Q$, $Q_y$ and $\phi$ 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 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 (FEMA-356, 2000).
DUCTILITY REDUCTION FACTOR (Rµ)

To calculate Rµ, nonlinear and linear dynamic analyses were taken into account. As mentioned, the nonlinear base shear Vᵧ 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

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Year</th>
<th>PGA (g)</th>
<th>Duration (s)</th>
</tr>
</thead>
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<tr>
<td>Elcentro</td>
<td>1940</td>
<td>0.348</td>
<td>19.18</td>
</tr>
<tr>
<td>Tabas</td>
<td>1978</td>
<td>0.933</td>
<td>25.00</td>
</tr>
<tr>
<td>Abhar</td>
<td>1990</td>
<td>1.000</td>
<td>29.52</td>
</tr>
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</table>

RESULTS

The incremental dynamic analysis results in terms of base shear-roof displacement for 5-story T-MMRF have been shown in Fig. 11. Table 2 highlights the ultimate base shear Vᵧ from nonlinear dynamic analysis under Elcentro, Abhar and Tabas occurrences for MMRFs and T-MMRFs. Table 3 shows the maximum elastic base shear Ve, resulted from the linear dynamic analysis under above-mentioned time histories. Table 4 shows the incremental dynamic analysis results of ductility factors. It can be seen that the ductility factors decrease with an increase in the height of building. Fig. 12 shows the variation in ductility factors with respect to the incremental dynamic analysis results of MMRFs and T-MMRFs. It is found that the influence of TADAS dampers on the ductility factor is considerable, this influence in middle stories is greater than the others.
Figure 11. Incremental dynamic roof displacement-base shear curve for 5 story T-MMRFs

Table 2. Ultimate base shear V_y from nonlinear dynamic analysis under Elcentro, Abhar and Tabas ground motion

| No. | MMRFs | T-MMRFs | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Abhar | Elcentro | Tabas | V_y (avg.) (kN) | Abhar | Elcentro | Tabas | V_y (avg.) (kN) |
| 3 | 300.7 | 274.9 | 360.0 | 311.9 | 326.9 | 325.5 | 408.6 | 353.7 |
| 5 | 339.1 | 311.6 | 337.6 | 329.4 | 350.4 | 354.0 | 356.1 | 353.5 |
| 7 | 377.1 | 326.3 | 419.1 | 340.8 | 340.0 | 310.8 | 386.3 | 345.7 |
| 10 | 487.6 | 526.5 | 430.7 | 481.6 | 384.8 | 460.6 | 511.1 | 452.2 |
| 15 | 567.2 | 494.9 | 589.9 | 550.7 | 493.4 | 618.5 | 689.6 | 600.5 |

Figure 12. Ductility factor of structures from incremental dynamic analysis
Table 3. Maximum elastic base shear $V_e$ from linear dynamic analysis under Elcentro, Abhar and Tabas ground motion

<table>
<thead>
<tr>
<th>No. Story</th>
<th>MMRFs</th>
<th>T-MMRFs</th>
<th>$V_e$ (avg.) (kN)</th>
<th>MMRFs</th>
<th>T-MMRFs</th>
<th>$V_e$ (avg.) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>Elcentro</td>
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<td>3</td>
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<td>635.7</td>
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<td>1355.4</td>
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<td>1194.8</td>
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<td>1890.8</td>
<td>2764.9</td>
<td>2029.5</td>
<td>3128.0</td>
<td>2637.0</td>
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Table 4. Ductility reduction factors ($R\mu$) from incremental dynamic analysis

<table>
<thead>
<tr>
<th>No. Story</th>
<th>MMRFs</th>
<th>T-MMRFs</th>
<th>MMRFs</th>
<th>T-MMRFs</th>
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<td>3.25</td>
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<td>5.15</td>
<td>3.69</td>
<td>4.63</td>
</tr>
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</table>

CONCLUSION

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

1. It is found that the influence of TADAS dampers on the ductility factor is considerable (Fig. 12).
2. The ductility reduction factor for moderate moment resisting frames with TADAS devices (T-MMRFs) can be assumed to equal 4.94.

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