Vertical Seismic Load Effects on the Response of Structures with Toggle Brace Dampers

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Abstract

Earthquakes; such as the 1995 Kobe, 1994 Northridge and 1986 Kalamata, show that vertical component contributes a destructive energy and even exceeds horizontal component effects. For that reason Earthquake Hazards Reduction Programs, such as the one run by the US National Institute of Standards and Technology, are highlighting the need for modelling vertical ground motion effects in nonlinear analysis. Consequently, the aims of this study are to analyse the combined vertical and horizontal components of earthquake effects on floor drift and acceleration response in a steel frame structure. The structure is an eight story three bay steel “moment resisting frame” located in Bucharest. Toggle-brace dampers and diagonal viscous damper were used as energy dissipation devices. The structure model is prepared and analysed in Simulia Abaqus FEA software, as the non-linear finite element method general purpose system. Three earthquake records are applied to the model. The selected records have strong vertical ground motions with field evidence of their damage to structural and non-structural elements. A total of 16 tests were conducted throughout this research. One test was devoted for frequency analysis, three for quantifying critical damping and twelve for seismic response. Simulation results show the difference in the efficiency of the selected dampers in reducing floor accelerations and drifts. Furthermore, the structural response is assessed for Eurocode Damage limit state. This research provides novel in-depth focus on structural response and mitigating damage induced by vertical seismic excitations.

Keywords: nonlinear structural response, seismic load, time-history analysis, finite element method, Simulia Abaqus FEA, toggle-brace dampers, floor accelerations, inter-story drifts.

1 Introduction

Structures, in general, are vulnerable to three-dimensional earthquake actions. The horizontal excitations have been extensively studied, whereas the vertical seismic
load effects has not gained yet importance, both in design and research [1]. Earthquakes; such as the 1995 Kobe, 1994 Northridge, 1989 Loma Prieta and 1986 Chi-Chi show that vertical component contributes a destructive energy and even exceeds the horizontal component effects [1]. Vertical seismic components are typically more complex than horizontal components in high-frequency spectrum range. In addition, recent studies show that vertical structural response for high-frequency is greater than horizontal structural response [2]. For that reason Earthquake Hazards Reduction Programs, such as the one run by the US National Institute of Standards and Technology [2], are highlighting the need for modelling vertical ground motion effects in nonlinear analysis.

Recent earthquakes revealed that modern seismic designed buildings may be able to protect structural elements, and hence, saving occupants from catastrophic collapse; However, the damage to non-structural elements may be severe, costly and in certain cases life threatening [3]. Non-structural elements (such as fire sprinklers, false ceiling, printers and other building contents) are affected by floor acceleration (Acceleration sensitive). Whereas components (such as cladding and partition walls) are drift sensitive[4]. Although these elements are usually protected during serviceability-level earthquakes (SLE) in seismically designed buildings, they might suffer from extensive damage under ultimate-level Earthquakes (ULE). Figure 1 shows cracked partition wall and collapsed duct due to an earthquake[5].

![Figure 1 Cracked partition wall and collapsed duct (adopted from [5])](image)

Shrestha [1] highlights the fact that “the vertical component of the ground motion found to be exceeding the horizontal component, which directly contradicts the current codal provision that assumes the value of the vertical ground motion to be 1/2 to 2/3 of the horizontal component”. In opposition, there is a thought that vertical earthquake component has less damage potential due to their low energy[6]. However, Papazoglou and Elnashai [6] confirming that “dismissing the vertical component is inadequate, since the horizontal energy content is dominated by long period pulses which are non-existent in vertical strong-motion records”. Therefore, the objectives of this study are to analyse the combined vertical and horizontal
components of earthquake effects on floor drift and acceleration response in a steel frame structure.

Nowadays, different energy dissipative devices are being used. However, fluid viscous dampers are widely used as passive dampers due to their availability and easiness of installation. They can be used in different configurations such as diagonal, chevron, scissor-jack and toggle-brace. In this study, Toggle brace dampers (TBD) are used. The reason behind the usage of this type of dampers is their ability to amplify the displacement and velocity; and hence; increasing the efficiency of energy dissipation [7]. Then, diagonal viscous dampers (DVD) are modelled to compare the efficiency of both types. Table 1 shows the magnification factors of different configurations. It can be noticed from Table 1, that toggle brace configuration amplifies the damper displacement for a given inter-story drift. Consequently, damping force is magnified through the toggle mechanism and transferred to the structure as tension or compression forces through the braces [7].

<table>
<thead>
<tr>
<th>Sketch</th>
<th>Diagonal</th>
<th>Chevron</th>
<th>Scissor-Jack</th>
<th>Reverse Toggle</th>
</tr>
</thead>
<tbody>
<tr>
<td>![Diagonal Sketch]</td>
<td>![Chevron Sketch]</td>
<td>![Scissor-Jack Sketch]</td>
<td>![Reverse Toggle Sketch]</td>
<td></td>
</tr>
<tr>
<td>Factor Equation</td>
<td>$f = \cos\theta$</td>
<td>$f = 1$</td>
<td>$f = \frac{\cos\phi}{\tan\theta}$</td>
<td>$f = \frac{a\cos\theta_1}{\cos(\theta_1 + \theta_2)} - \cos\theta_2$</td>
</tr>
<tr>
<td>Factor Value</td>
<td>$\theta = 37^\circ$</td>
<td>$f = 0.8$</td>
<td>$\theta = 9^\circ, \phi = 70^\circ$</td>
<td>$\theta_1 = 30^\circ, \theta_2 = 49^\circ, \alpha = 0.7$</td>
</tr>
</tbody>
</table>

Table 1 Magnification factors for different damper configurations, adopted from [7]

Eurocode limits inter-storey drift to $(dr.v \leq 0.005\ h)$ for buildings having non-structural elements of brittle materials attached to the structure (EC8: 4.4.3.2(1)(a)). Where h is storey height, $dr$ is Design Inter-storey drift, which is equal to the difference between the average lateral displacement at the top and bottom of the storey under consideration (EC8: 4.4.2.2(2)) and $v$ is the reduction factor which is recommended to be equal to 0.4 by EC8 for buildings of importance classes III “Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions” (EC8: Table 4.3 Importance classes for buildings).

Meeting EC8 damage limit state allows the assessment of the structural response, and then, the efficiency of the used dampers. However, NEHRP recommended seismic provisions [5], states that there are no implicit performance goals associated with the ultimate-level earthquakes for non-structural components. Therefore, within the scope of this study, if the structural response (under SLE) meets EC8 limit, then it will be considered as the performance goal when applying ULE with using the selected energy dissipating devices, TBD and DVD.
2 Structure Description

The structure under consideration is an eight-storey, three bays steel moment resisting frame located in Bucharest. The building structural properties are taken from Mociran study [8]. The frame has a typical floor height of 3.5m with taller first storey (4.5m). The office building is designed according to Eurocode 3 and Romanian seismic design code P100-1/2006. The building is symmetric in both directions and has 6m bays. Member sizes are described in Table 2 and are made of steel grade S355. Fixed base is considered at the foot of the columns. All beam-column connections are assumed to be rigid with total inherited structural damping of 0.02. A uniformly distributed dead load of 4kN/m² and a uniformly distributed live load of 2kN/m² were considered. While the building is symmetric, an exterior frame would be modeled and analyzed as a two-dimensional frame. The 6 kN/m² total distributed load has been converted into 9 kN/m line load applied on the beams. Figure 2 shows frame dimensions and loadings.

<table>
<thead>
<tr>
<th>Storey</th>
<th>Beam Section</th>
<th>Column Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Interior Columns</td>
</tr>
<tr>
<td>1 to 4</td>
<td>IPE500</td>
<td>HEB600</td>
</tr>
<tr>
<td>5 &amp; 6</td>
<td>IPE450</td>
<td>HEB500</td>
</tr>
<tr>
<td>7 &amp; 8</td>
<td>IPE360</td>
<td>HEB400</td>
</tr>
</tbody>
</table>

Table 2 Specification of frame structural elements (adopted from[8])

![Figure 2 Frame dimensions and loadings (Abaqus modeling)]
Simulia Abaqus FEA, as a non-linear finite element method general purpose system, is used in this study to model and analyse the frame response. Table 3 below highlights technical points for modeling the frame in Simulia Abaqus:

<table>
<thead>
<tr>
<th>Division</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Part</td>
<td>Drawing the frame as one part, to ensure the connections’ rigidity.</td>
</tr>
<tr>
<td>Section Property</td>
<td>All members are modeled as “beam” finite elements with uniform section along the element</td>
</tr>
<tr>
<td>Material Properties</td>
<td>Steel material properties involve elasticity (E=210GPa, Poisson’s ratio=0.33), density (7850kg/m3) and damping ($\alpha$=0.035, $\beta$=0.0093).</td>
</tr>
<tr>
<td>Profile</td>
<td>All IPE and HEB section dimensions were entered manually.</td>
</tr>
<tr>
<td>Step</td>
<td>Three steps are added; (initial), (static, general) and (dynamic, implicit).</td>
</tr>
<tr>
<td></td>
<td>The “Nlgeom” option is set as “ON” in all steps to account for the geometric non-linearity of the model.</td>
</tr>
<tr>
<td>Load</td>
<td>Dead and live loads are added in static step and propagated to the dynamic step.</td>
</tr>
<tr>
<td>Boundary Condition</td>
<td>In the static step, foundations are fixed against all degree of freedoms.</td>
</tr>
<tr>
<td></td>
<td>In the dynamic step, earthquakes are applied as acceleration-time histories on the foundation (for both x and y global directions).</td>
</tr>
<tr>
<td>Interactions</td>
<td>Rigid diaphragms are implemented for each floor using rigid body constraint for all beams in a single floor.</td>
</tr>
<tr>
<td>Mesh</td>
<td>Mesh size = 0.5m, which is adequate for the scope of this study.</td>
</tr>
</tbody>
</table>

Table 3 Technical notes for modeling the frame in Simulia Abaqus FEA

The use of Reyleigh coefficients is the most appropriate method for including structural damping in non-linear analysis [9]. That’s why $\alpha$ and $\beta$ mentioned in Table 3 are calculated according to Wilson’s method [9] to achieve the 0.02 inherited critical damping ratio, as follow:

When the critical damping has an equal value for both frequencies of dominating modes, then $\zeta_i = \zeta_j = \zeta$, where i and j are the first two modes of vibration, $\zeta$ is critical damping. Therefore:

$$\beta = \frac{2\zeta}{\omega_i + \omega_j} \quad (1)$$
$$\alpha = \omega_i \cdot \omega_j \cdot \zeta \quad (2)$$
$$\omega = 2\pi \cdot f \quad (3)$$

After conducting modal analysis, the natural frequencies of the model have been found as follow: $f = 0.193$ (cycles/sec) for the 1st mode and $f = 0.49$ (cycles/sec) for the 2nd mode. Therefore, $\omega_1 = 1.212$ and $\omega_2 = 3.07$. Substituting these results in equation (1) and (2) with target critical damping $\zeta$ of 0.02, $\beta$ was calculated as 0.0093 and $\alpha$ as 0.035.
3 Passive Dampers

3.1 Toggle-Brace Dampers (TBD)

As mentioned before, TBD are used due to their magnification capability. Figure 3 shows the configuration and elements of TBD. It consists of two RHS 350x250x16 bracing with one fluid viscous damper that has 2.65m length. Dampers are usually modelled as a linear spring and a dash-pot in parallel (known as Kelvin model) where the spring represents stiffness and the dash-pot represents damping[10]. The fluid viscous damper has the same properties as that recommended by Marko[10]. It was modelled as two pipes sliding inside each other with a ‘Translator’ connector that has stiffness coefficient $k_d=10000$ kNs/m and damping coefficient $C_d=50000$ kN/m. All the elements of the TBD are pin connected by using the constraint called in Abaqus MPC Pin. This is simply to allow the rotation at the joints and transfer the energy as axial forces.

According to Table 1, the magnification coefficient can be calculated according to the angles provided in Figure 3. While $L = \alpha L$, then $\alpha = 1$. In addition, $\theta_1 = 49^\circ$ and $\theta_2 = 25^\circ$. Therefore, the magnification factor is $f = 2.63$. This research focuses on the effect of this factor on efficiency of the damper in reducing floor accelerations and storey drifts in both vertical and horizontal direction. Figure 5 shows TBD distribution in the whole frame of the building.

3.2 Diagonal Viscous Dampers (DVD)

This type is implemented to compare its influence on structure with that of the TBD. Figure 4 shows the configuration and elements of DVD. It consists of two RHS 350x250x16 bracing with one fluid viscous damper that has the same length as that of TBD (which is 2.65m). It was important to keep the same properties of the fluid
viscous damper to understand the configuration effect. It was modelled as two pipes sliding inside each other with a ‘Translator’ connector that has stiffness coefficient $k_d=10000$ kNs/m and damping coefficient $C_{d}=50000$ kN/m. The ends of the DVD bracing are pin connected by using the same MPC Pin constraint.

Returning to Table 1, the magnification coefficient can be calculated according to the angles provided in Figure 3. While $\theta = 30.25^\circ$, the magnification factor is $f = 0.86$. The effect of this factor, in reducing floor accelerations and inter-storey drifts in both vertical and horizontal direction is shown in the result section. Figure 6 shows DVD distribution in the whole frame of the building.
4 Selected Earthquakes

The earthquake records used in this study are selected critically. They are major earthquakes with field evidence of their damage potential in vertical direction. The first one is the 1995 Kobe earthquake in Japan. It caused numerous bridge failures which were strongly attributable to the sudden rise in pier axial compression forces caused by vertical ground motion [6]. Its vertical to horizontal peak ground acceleration ratio was even more than one. The second record is the Kalamata, Greece earthquake of 13 September 1986. The epicentre of this event was located less than 9 km from the city center with focal depth of 7 km. The comprehensive field analysis of Papazoglou and Elnashai [6] mention that “This type of shallow near-field event, having a relatively short recurrence interval, is a major threat to densely populated regions, especially in Europe and other intraplate zones”. The third one is the Northridge, California earthquake of 17 January, 1994. This event documented as one of the most destructive earthquakes that hit the west coast of USA [11]. Table 4 lists the main properties of the selected records.

<table>
<thead>
<tr>
<th>No.</th>
<th>EQ. name</th>
<th>Year</th>
<th>Station</th>
<th>PGA$^v$</th>
<th>PGA$^h$</th>
<th>V/H</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Kobe</td>
<td>1995</td>
<td>Fukushima</td>
<td>1.96</td>
<td>1.74</td>
<td>1.13</td>
</tr>
<tr>
<td>2</td>
<td>Kalamata</td>
<td>1986</td>
<td>OTE-Building</td>
<td>1.86</td>
<td>2.34</td>
<td>0.80</td>
</tr>
<tr>
<td>3</td>
<td>Northridge</td>
<td>1994</td>
<td>Arleta</td>
<td>1.17</td>
<td>1.56</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Table 4 Main properties of the selected earthquakes

According to Eurocode 8 [12], the acceleration-time history of these records should be multiplied by the following factor (f):

\[
f = \frac{PGA \text{ of the EQ that has max. } PGA}{PGA \text{ of the earthquake under consideration}}
\]  

(4)

This scaling procedure maintains compatibility between the records and validates taking the average results of the structural response. Based on the PGA$^v$ and PGA$^h$ given in Table 4, the corresponding factors are listed in Table 5 for each earthquake.

<table>
<thead>
<tr>
<th>No.</th>
<th>EQ. name</th>
<th>$f_\nu$</th>
<th>$f_\nu^{max}$</th>
<th>$f_\nu^{p}$</th>
<th>$f_\nu^{s}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Kobe</td>
<td>1.96/1.96=</td>
<td>1</td>
<td>2.34/1.74</td>
<td>1.34</td>
</tr>
<tr>
<td>2</td>
<td>Kalamata</td>
<td>1.96/1.86=</td>
<td>1.05</td>
<td>2.34/2.34</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>Northridge</td>
<td>1.96/1.17=</td>
<td>1.68</td>
<td>2.34/1.56</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Table 5 Scaling factors for the selected earthquakes

Figure 7 and 8 represent the scaled acceleration-time history data for the three selected earthquakes in horizontal and vertical directions, respectively. Earthquakes are generally classified to 2 categories according to their intensities [12]:

- Serviceability level earthquakes (SLE): are earthquakes with probability of exceedance equal to 10% in 10 years (which means 95 years return period).
- Ultimate level earthquakes (ULE): are earthquakes with probability of exceedance equal to 10% in 50 years (which means 475 years return period).
Scaling Factors between ULE and SLE intensities are multiplying SLE by 2.5 to get ULE, or multiplying ULE by 0.4 to get SLE [13]. Therefore, while the input data in Figure 7 and 8 are considered as ULE, they were simply reduced by 60% to get SLE.

5 Methodology

5.1 Conducted Tests

A total of 16 tests were done within this study. Figure 9 shows tree diagram of the testing scheme. One test is related to quantify the dynamic properties of the model such as modal shapes and frequencies. This is achieved through the frequency modal analysis provided by Simulia Abaqus FEA.

Then, three tests are devoted for finding the critical damping of the unequipped structure, equipped structure with TBD and that with DVD. This is conducted through the application of a pulse displacement at the structure foundation and examining the rate at which the top floor displacement decay.

In addition, 12 dynamic tests (4 for each earthquake) are implemented to examine the structural response and the efficiency of the selected dampers in both vertical and horizontal directions. The structural response (acceleration and drift) of the unequipped model under the SLEs were recorded and dealt with as the target performance. The response to ULEs was higher and then the efficiency of attaching TBD and DVD were found using the following equation:
where \( R \) is the response (either acceleration or drift)

Each damping configuration has efficiency in reducing horizontal drift, vertical drift, horizontal acceleration and vertical acceleration.

\[
\text{Efficiency (\%)} = \frac{R_{\text{Uneq.ULE}} - R_{\text{with damper.ULE}}}{R_{\text{Uneq.ULE}} - R_{\text{Uneq.SLE}}} \times 100
\]  

(5)

\( \frac{dr}{h} \leq 0.0125 \)  

Inter-storey drifts are divided by the storey height to overcome the difference in storey height between the first story and other typical stories. As mentioned before, EC8 specifies the damage limit state as: \( dr.v \leq 0.005 \text{ h} \). While \( v=0.4 \), then:

\[
\frac{dr}{h} \leq 0.0125
\]  

(6)

Figure 16 Tree diagram for testing scheme

5.2 Analysis of outcomes

While each floor diaphragm is assumed to be rigid, then, all the points in a specific floor would have the same accelerations and displacements. Therefore, one reference point is selected at each floor. The responses at each reference point (R.P) are recorded for the whole period of Kobe excitation. Then, for every R.P, the maximum response is selected. This single value is added to its corresponding values for Kalamata and Northridge and divided by 3 to get the average response at each R.P.

Again, to overcome the difference in storey height between the first story and other typical stories, floor elevations are divided by total structure height to get the relative height. Accelerations are divided by PGA. Results are compared with current codal provisions and the efficiency of TBD and DVD are estimated.
6 Results and Discussion

6.1 Frequency

Results for the frequency analysis of the eight-storey frame revealed that the structure has three dominating modes of vibration. These mode shapes are shown in Figure 10. The corresponding frequencies are used to quantify Reyleigh coefficients as elaborated in section 2 previously.

![Mode shapes and their frequencies](image)

Figure 10 Mode shapes and their frequencies (cycles/sec.)

6.2 Damping Ratios

A pulse load of 0.04 m within 1 sec was applied on the foundations of the structure in both directions to record the free vibration of the building. Figure 11 and 12 below show the rate at which the vibration decay for the unequipped, TBD and DVD cases (for horizontal and vertical directions respectively).

![Horizontal free vibrations](image)

Figure 11 Horizontal free vibrations for the unequipped, TBD and DVD cases

![Vertical free vibrations](image)

Figure 12 Vertical free vibrations for the unequipped, TBD and DVD cases
Then, corresponding logarithmic decrement values were calculated to quantify the critical damping ratio of the unequipped and equipped structure. Table 6 shows damping ratios in both directions. Two significant points can be noticed. The first is that the unequipped structure reflected the same assumed inherited structural damping of 2%. The second point is that TBD and DVD has stronger influence in vertical direction than that in horizontal one.

<table>
<thead>
<tr>
<th></th>
<th>Horizontal</th>
<th>Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unequipped</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>TBD</td>
<td>5.8</td>
<td>7.6</td>
</tr>
<tr>
<td>DVD</td>
<td>13.7</td>
<td>21.8</td>
</tr>
</tbody>
</table>

Table 6 Damping ratios (in %)

### 6.3 Inter-storey drifts

#### 6.3.1 Horizontal inter-storey drifts

Horizontal drifts at each storey for each earthquake were calculated, then, the average value was taken and written in Table 7 (columns 2 to 9). These values were used to draw Figure 13. The last column in Table 7 represent the mean horizontal drift of all floors together, which was used to draw Figure 14 as shown below.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Elevation / Total Height</th>
<th>8th</th>
<th>7th</th>
<th>6th</th>
<th>5th</th>
<th>4th</th>
<th>3rd</th>
<th>2nd</th>
<th>1st</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unequipped SLE</td>
<td>1</td>
<td>0.0083</td>
<td>0.1111</td>
<td>0.0061</td>
<td>0.0071</td>
<td>0.0054</td>
<td>0.0061</td>
<td>0.0092</td>
<td>0.0074</td>
<td></td>
</tr>
<tr>
<td>TBD ULE</td>
<td>0.0085</td>
<td>0.0118</td>
<td>0.0114</td>
<td>0.0121</td>
<td>0.0105</td>
<td>0.0105</td>
<td>0.0110</td>
<td>0.0167</td>
<td>0.0116</td>
<td></td>
</tr>
<tr>
<td>DVD ULE</td>
<td>0.0048</td>
<td>0.0086</td>
<td>0.0093</td>
<td>0.0108</td>
<td>0.0089</td>
<td>0.0095</td>
<td>0.0101</td>
<td>0.0147</td>
<td>0.0096</td>
<td></td>
</tr>
<tr>
<td>EC</td>
<td>0.0125</td>
<td>0.0125</td>
<td>0.0125</td>
<td>0.0125</td>
<td>0.0125</td>
<td>0.0125</td>
<td>0.0125</td>
<td>0.0125</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unequipped ULE</td>
<td>0.0206</td>
<td>0.0274</td>
<td>0.0150</td>
<td>0.0178</td>
<td>0.0136</td>
<td>0.0152</td>
<td>0.0152</td>
<td>0.0228</td>
<td>0.0184</td>
<td></td>
</tr>
</tbody>
</table>

Table 7 Horizontal drift ratios for different structural cases and their mean

![Figure 13 Horizontal Drift ratios with relative height](image)
6.3.2 Vertical inter-storey drifts

Vertical drifts at each storey for each earthquake were calculated, then, the average value was taken and written in Table 8 (columns 2 to 9). These values were used to draw Figure 15. The last column in Table 8 represent the mean vertical drift of all floors together, which was used to draw Figure 16 as shown below.

<table>
<thead>
<tr>
<th>Floor</th>
<th>8th</th>
<th>7th</th>
<th>6th</th>
<th>5th</th>
<th>4th</th>
<th>3rd</th>
<th>2nd</th>
<th>1st</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation/Total Height</td>
<td>29/29</td>
<td>25.5/29</td>
<td>22/29</td>
<td>18.5/29</td>
<td>15/29</td>
<td>11.5/29</td>
<td>8/29</td>
<td>4.5/29</td>
<td></td>
</tr>
<tr>
<td>Unequipped SLE</td>
<td>0.0002</td>
<td>0.0006</td>
<td>0.0006</td>
<td>0.0007</td>
<td>0.0007</td>
<td>0.0008</td>
<td>0.0009</td>
<td>0.0011</td>
<td>0.0007</td>
</tr>
<tr>
<td>TBD ULE</td>
<td>0.0005</td>
<td>0.0010</td>
<td>0.0015</td>
<td>0.0022</td>
<td>0.0024</td>
<td>0.0028</td>
<td>0.0030</td>
<td>0.0032</td>
<td>0.0021</td>
</tr>
<tr>
<td>DVD ULE</td>
<td>0.0003</td>
<td>0.0007</td>
<td>0.0009</td>
<td>0.0014</td>
<td>0.0014</td>
<td>0.0016</td>
<td>0.0018</td>
<td>0.0022</td>
<td>0.0013</td>
</tr>
<tr>
<td>Unequipped ULE</td>
<td>0.0008</td>
<td>0.0017</td>
<td>0.0015</td>
<td>0.0018</td>
<td>0.0017</td>
<td>0.0020</td>
<td>0.0024</td>
<td>0.0027</td>
<td>0.0018</td>
</tr>
</tbody>
</table>

Table 8 Vertical drift ratios for different structural cases and their mean
Similarly, the efficiency for TBD and DVD were calculated as follow:

\[
Efficiency_{TBD} (%) = \frac{0.0018 - 0.0016}{(0.0018 - 0.0007)} \times 100 = 18.2\%
\]

\[
Efficiency_{DVD} (%) = \frac{0.0018 - 0.0013}{(0.0018 - 0.0007)} \times 100 = 45.5\%
\]

Although all drift values lies within EC limit, it can be noticed that TBD has a less impact on the structure. It reduces the vertical drift by around 18.2 %. On the other hand, DVD was able to reduce about half the drift required to be eliminated.

### 6.4 Floor accelerations

#### 6.4.1 Horizontal floor acceleration

Horizontal floor accelerations at each storey for each earthquake were calculated, then, the average value was taken and written in Table 9 (columns 2 to 9). These values were used to draw Figure 17. The last column in Table 9 represent the mean floor acceleration of all floors together, which was used to draw Figure 18.
Table 9 Values of (Horizontal acceleration/PGA) and their mean

<table>
<thead>
<tr>
<th>Floor</th>
<th>8th</th>
<th>7th</th>
<th>6th</th>
<th>5th</th>
<th>4th</th>
<th>3rd</th>
<th>2nd</th>
<th>1st</th>
<th>Base</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elev.</td>
<td>29/</td>
<td>25.5/</td>
<td>22/</td>
<td>18.5/</td>
<td>15/</td>
<td>11.5/</td>
<td>8.0/</td>
<td>4.5/</td>
<td>0/</td>
<td>0/</td>
</tr>
<tr>
<td>Total</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
</tr>
</tbody>
</table>

Unequipped SLE | 0.485 | 0.223 | 0.317 | 0.242 | 0.255 | 0.225 | 0.266 | 0.305 | 0.377 | 0.290 |
TBD ULE | 0.773 | 0.540 | 0.425 | 0.493 | 0.511 | 0.556 | 0.585 | 0.690 | 0.942 | 0.572 |
DVD ULE | 0.346 | 0.269 | 0.199 | 0.246 | 0.296 | 0.310 | 0.388 | 0.515 | 0.942 | 0.321 |
Unequipped ULE | 1.202 | 0.555 | 0.787 | 0.613 | 0.631 | 0.551 | 0.643 | 0.775 | 0.942 | 0.720 |

Figure 17 Values of (Horizontal acceleration/PGA) with relative height

As mentioned in section 1, there is no codal implicit performance goals associated with the ULE for non-structural components. Therefore, within the scope of this study, the structural response (under SLE) considered as the performance goal. It is evident from Figure 17 that DVD performed very well in reducing the acceleration. They have an efficiency of:

\[
\text{Efficiency of } TBD \% = \frac{0.7195 - 0.5715}{(0.7195 - 0.2896)} \times 100 = 34.4\% \\
\text{Efficiency of } DVD \% = \frac{0.7195 - 0.3211}{(0.7195 - 0.2896)} \times 100 = 92.7\% 
\]
6.4.2 Vertical floor acceleration

Vertical floor accelerations at each storey for each earthquake were calculated, then, the average value was taken and written in Table 10 (columns 2 to 9). These values were used to draw Figure 19. The last column in Table 9 represent the mean vertical floor acceleration of all floors together, which was used to draw Figure 20.

<table>
<thead>
<tr>
<th>Floor</th>
<th>8th</th>
<th>7th</th>
<th>6th</th>
<th>5th</th>
<th>4th</th>
<th>3rd</th>
<th>2nd</th>
<th>1st</th>
<th>Base</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation/Total Height</td>
<td>29/29</td>
<td>25.5/29</td>
<td>22/29</td>
<td>18.5/29</td>
<td>15/29</td>
<td>11.5/29</td>
<td>8/29</td>
<td>4.5/29</td>
<td>0/29</td>
<td></td>
</tr>
<tr>
<td>Unequipped SLE</td>
<td>1</td>
<td>0.879</td>
<td>0.759</td>
<td>0.638</td>
<td>0.517</td>
<td>0.397</td>
<td>0.276</td>
<td>0.155</td>
<td>0.000</td>
<td>0.367</td>
</tr>
<tr>
<td>TBD ULE</td>
<td>0.488</td>
<td>0.453</td>
<td>0.389</td>
<td>0.355</td>
<td>0.339</td>
<td>0.276</td>
<td>0.323</td>
<td>0.314</td>
<td>0.331</td>
<td>0.848</td>
</tr>
<tr>
<td>DVD ULE</td>
<td>1.135</td>
<td>1.086</td>
<td>0.955</td>
<td>0.851</td>
<td>0.771</td>
<td>0.672</td>
<td>0.636</td>
<td>0.676</td>
<td>0.827</td>
<td>0.708</td>
</tr>
<tr>
<td>Unequipped ULE</td>
<td>0.958</td>
<td>0.795</td>
<td>0.635</td>
<td>0.592</td>
<td>0.562</td>
<td>0.607</td>
<td>0.805</td>
<td>0.711</td>
<td>0.827</td>
<td>0.957</td>
</tr>
</tbody>
</table>

Table 10 Values of (vertical acceleration/PGA) and their mean

![Figure 19](image1.png)

Figure 19 Values of (vertical acceleration/PGA) with relative height

![Figure 20](image2.png)

Figure 20 Mean values of (vertical acceleration/PGA) of all floors

The corresponding efficiencies are calculated as follow:

\[
Efficiency \ of \ TBD \ (\%) = \frac{0.9569 - 0.8477}{0.9569 - 0.3672} \times 100 = 18.5\%
\]

\[
Efficiency \ of \ DVD \ (\%) = \frac{0.9569 - 0.7080}{0.9569 - 0.3672} \times 100 = 42.2\%
\]
6.5 Interpretation of outcomes

The following points can be highlighted from previous results:

- Both TBD and DVD configuration were able to limit horizontal inter-storey drifts under EC damage limit.
- DVD added structural damping more than triple that of TBD (in both directions).
- In vertical direction, the efficiencies of TBD and DVD are almost the same for drift and acceleration. This may reflect influence compatibility. Figure 21 shows a summary of efficiencies for both TBD and DVD.

![Figure 21: Efficiencies for TBD and DVD](image)

- Generally, efficiencies of both configurations were higher in horizontal direction than that in the vertical one. For example, DVD had efficiency of 93% in reducing horizontal acceleration compared to only 42% for vertical acceleration. This might be due to the high frequency field available in vertical component of ground acceleration that may not be well-caught by the damper. This also reflects the energy potential imbedded in vertical excitations.
- As mentioned in section 1, the magnification factor of DVD was only 0.68 compared to 2.63 for TBD. This supposed to reflect better performance for TBD. In contrast, the efficiency of DVD was higher than that of TBD. The reason behind that might be the ductility of the steel frame. In ductile frames, large inter-storey drifts are magnified in TBD configuration, and hence, the fluid viscous damper used in it reaches its capacity before that of DVD. The case might be the opposite for concrete frames where the displacements are small. Boyle and Constantinou [7] state that “toggle-brace configuration is suitable for applications of wind-response reduction and seismic risk mitigation for stiff structures”.

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7 Conclusions

A total of 16 tests were conducted throughout this research. One test was devoted for frequency analysis, three for damping and twelve for seismic response. Three earthquakes were applied as acceleration-time histories on the foundation of the building and the mean response value were quantified. Simulation results show the difference in the efficiency of the selected dampers in reducing floor acceleration and drift. Results revealed that TBD and DVD configuration were able to limit horizontal inter-storey drifts under EC damage limit. In addition, toggle-brace damper (TBD) configuration has less efficiency than that of diagonal viscous damper (DVD) configuration. The reason behind that might be the ductility of the steel frame. In ductile frames, large inter-storey drifts are magnified in TBD configuration, and hence, the fluid viscous damper used in TBD reaches its capacity before that of DVD. Literature confirms that the use of TBD configuration is more efficient in stiff structures where floor displacements are relatively small. Results also showed that efficiencies of both configurations were higher in horizontal direction than that in the vertical one. This might be due to high-frequency spectrum range available in vertical component of ground acceleration that may not be well-caught by the damper. This also reflects the energy potential imbedded in vertical excitations.

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References


