Seismic retrofit of structures using steel frames with viscoelastic hinges

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ABSTRACT

In this paper a new seismic retrofit system composed of a steel frame with viscoelastic hinges is proposed and its applicability and efficiency are evaluated in a theoretical framework. First, the mechanical behavior of viscoelastic hinges and the system are studied and the related formulas are derived. The analytical model of the proposed seismic damper is established first, and the damper is subjected to cyclic loads to compare its hysteretic behavior with the one from the formulation. In order to further evaluate the efficiency of this system, the validated analytical model is used in seismic retrofit of a case study structure to reach the collapse prevention performance under the maximum considered earthquake hazard level. The seismic performance of the analysis model is compared before and after retrofit in terms of maximum interstory drift ratio, residual displacement, and energy dissipation of structural members and dampers. The results show that the proposed damper can be used to reach the stipulated drift limit state. Furthermore, the inelastic energy dissipated by the structural members is reduced drastically and the structure can be effectively protected against irreversible damages due to inelastic deformations.

1. Introduction

Using damping devices in seismic retrofit of structures without blocking the bays and providing open spaces has been of great interest in the field of seismic engineering. For instance, Sahoo and Rai [1] studied different retrofit strategies for soft-first-story structures by implementing the column jacketing technique and using chevron braces with aluminum shear links. While seemingly effective, the strengthened frames with steel cages could not withstand strong ground motions due to plastic hinge formation at the ground story columns. Agha Beigi et al. [2–4] investigated the seismic retrofit of soft-story structures in different studies and proposed a bracing system which can be installed beside existing columns to provide the required open space. The proposed system includes an inclined brace with a gap mechanism which is activated at a certain drift. It was observed that this system can increase the ductility and post-yield stiffness, whereas there were no significant changes in the lateral resistance.

The 2017 Pohang earthquake in Korea with the magnitude of 5.4 $M$ showed that the low-rise residential structures with soft-first story are susceptible to seismic damages under minor earthquakes. These structures have been built using the reinforced concrete load-bearing wall system with columns at the first story which is usually used as a parking lot. The main challenging part of retrofitting such structures is to provide an open space and not to block the bays when installing damping devices. Since then, different retrofit systems and strategies have been proposed and studied. Javidan and Kim [5] proposed a retrofit system composed of a steel frame with hinges and friction devices at the upper corners similar to knee-branches. They tested the friction device under cyclic loads and showed its efficiency applying it to an analysis model of a soft-story structure. They further developed and tested a column-type steel damper which can be installed beside existing columns [6,7]. This seismic device consists of a steel column and flexural fuses at both ends, which are made of plates with reduced sections. A reinforced concrete frame was equipped with the proposed damper and its performance was compared with that of an identical bare frame under cyclic loading. It was observed that the steel column damper can efficiently dissipate the input energy by increasing the lateral resistance, stiffness and ductility of the retrofitted frame. There have been similar attempts to improve the efficiency of the dampers and installation scheme, like the low-damage friction devices studied by Hashemi et al. [8,9] and Yousefbeik [10,11], or the slit plate dampers with vertical installation scheme tested by Naeem and Kim [12] and Javidan and Kim [13]. By using a vertical installation scheme, it was possible to develop a steel slit damper which can be placed inside partition walls or beside columns.

Among different types of seismic energy dissipation devices such as friction, metallic [14–16], and viscous [17,18] dampers, viscoelastic...
dampers have proven to be efficient in terms of versatility and dissipating vibrations under very small amplitudes. He et al. [19] developed a limb-like viscoelastic seismic device which induces both mechanical and geometrical nonlinearities. Xu et al. [20–22] have studied various viscoelastic devices with high damping properties that can be incorporated in bolted connections and seismic isolation.

The seismic retrofit system proposed in this research consists of a steel frame with viscoelastic hinges at corners as depicted in Fig. 1. The main advantage of the proposed system over conventional seismic energy dissipation systems is in providing an empty space for windows. The vertical and horizontal members of this frame consist of H-sections and are welded to viscoelastic hinges. The hinges are composed of steel plates and viscoelastic pads which are placed alternately, and a bolt going through the plates and pads, which is the center of rotational displacement. Using viscoelastic pads instead of friction pads can provide energy dissipation under small vibrations like wind loads. On the contrary, friction dampers may not be activated in small vibrations.

This system can be installed in the retrofit of existing reinforced concrete (RC) structures using chemical anchors. Holes are drilled in beams and columns of the structures, and then anchor bolts are inserted in the holes and are filled with chemical adhesive. Another sets of anchor bolts are welded to the steel frame and the gaps between the existing structure and the steel frame are filled with high strength non-shrink cement grout.

In order to study the feasibility and efficiency of this system, first the theoretical formulas for its mechanical behavior are derived and hysteresis curves are plotted using them. The analytical model of the damper is provided which is suitable for implementing in a common structural engineering analysis software. The hysteresis behavior of this model under cyclic loads is compared with the one obtained from theoretical formulas in order to validate the accuracy of the model and formulation. The applicability of this system is further investigated by applying it to seismic retrofit of a pre-code case study structure and thoroughly evaluating its seismic responses before and after retrofit using nonlinear time history analysis.

2. Theoretical formulation

The front view of the proposed system subjected to the lateral load \( F(t) \) and its deformed shape are depicted in Fig. 2(a). As the hysteresis behavior of the viscoelastic hinges and consequently the system are velocity-dependent, the applied force is considered to be a function of time \( t \). Translational displacements of at the lower horizontal member level are constrained, and the axial deformations of the members are considered to be negligible. The free body diagram of the system is depicted in Fig. 2(b), which is used for structural analysis and formulation of its mechanical behavior. Due to symmetry, the inflexion points are located at the center of the members and the free body diagram is simplified.

Let the lateral displacement corresponding to the applied force be \( \Delta(t) \). If the resisting moment provided by each viscoelastic hinges is \( M(t) \), the required lateral force equals

\[
F(t) = 4M(t)/H
\]

where \( H \) is the frame height which is the center-to-center distance of the viscoelastic hinges as indicated in Fig. 2. By finding the relationship between the moment \( M(t) \) and the applied displacement \( \Delta(t) \), the force-displacement curve can be obtained.

The resisting moment provided by the viscoelastic pads can be calculated using a double integral:

\[
M(t) = n \int \int dM = n \int \int \rho \tau dA
\]

where \( n \) is the number of viscoelastic pads in each hinge; \( \rho \) is the radial distance of the polar subrectangle viscoelastic element, as indicated in Fig. 3; \( \tau \) is the shear stress; and \( dA \) is the area of the subrectangle element. Hence, the moment can be determined in the polar coordinate as

\[
M(t) = n \int \int \rho \tau dA = n \int \int \rho \tau \phi d\phi d\rho
\]

where \( \phi \) is the polar angle as shown in Fig. 3. The applied deformation and hinge rotation have to be transformed to the shear stress in viscoelastic pads. Different material models can be used and implemented for analysis of the proposed system. In this paper, the Kelvin-Voigt model is utilized which is a well-known and simple model for viscoelastic dampers, and more details on fitting it to experimental data can be found in previous works [23]. This model consists of a spring and a dashpot placed in parallel, which gives the relationship between the shear stress \( \tau \) and shear strain \( \gamma \) of viscoelastic materials as follows

\[
\tau = G\gamma + G' \frac{d\gamma}{dt}
\]

In this formula, \( G' \) and \( G'' \) are the storage and loss moduli, respectively [23]. The shear strain \( \gamma \) at the indicated subrectangle element equals

![Fig. 1. Configuration of the proposed system: (a) details of the system; (b) installation scheme.](image-url)
where $\theta$ is the hinge rotation and $T$ is the thickness of the viscoelastic pad. By substituting the shear strain from Eq. 5, the shear stress at the polar subrectangle is expressed as

$$\tau = \frac{G'}{T} \left( \dot{\theta} + \frac{G'}{\omega} \frac{d\theta}{dt} \right)$$

(6)

By substituting this in Eq. 12, the force-displacement response of the system is determined as

$$F(t) = \frac{4\delta}{H^2} (K_\theta \sin \omega t + C_\theta \cos \omega t)$$

(14)

This equation can provide the theoretical force-displacement response of the proposed retrofit system. For example, given a sinusoidal lateral displacement with the amplitude of $\delta$ and the angular frequency of $\omega$, the rotational deformation of the viscoelastic hinge equals

$$\theta(t) = \Delta(t)/H = \delta \sin \omega t / H$$

(13)

By substituting this in Eq. 12, the force-displacement response of the system is determined as

$$F(t) = \frac{4\delta}{H^2} (K_\theta \sin \omega t + C_\theta \cos \omega t)$$

(14)
The maximum force is equal to $\frac{4\pi}{\omega} \sqrt{K_o^2 + (\omega C_o)^2}$, which is proportional to the drift and has an inverse relationship with the square of the height. The maximum force is rate independent, except for the material properties that can be slightly influenced by the loading rate. The dissipated energy inside one hysteresis loop is equal to:

$$\int Fd\Delta = \int_0^{2\pi} F(t)\omega \cos \omega t dt = \frac{4\pi}{\omega} \int_0^{2\pi} (K_o \sin \omega t \cos \omega t + \omega C_o \cos^2 \omega t) dt$$

which leads to the following relationship:

$$\int Fd\Delta = \frac{2n \pi^2 \ddot{G}}{T} (r_o^4 - r_i^4) (\frac{\dot{\Delta}}{H})^2$$

This is similarly a rate independent formula, except for the loss modulus of the viscoelastic material that can be affected by the strain rate. It should be noted that unlike the force $F$, the drift ratio $(\delta/H)$ is what matters in energy dissipation and the lateral displacement and the story height cannot independently affect it.

3. Analysis modeling and verification

To implement the proposed seismic retrofit system in the analysis of structures, its analytical model needs to be established. Furthermore, by subjecting the model to lateral displacements and comparing its hysteretic behavior with the theoretical formulation, it is possible to verify both derived formulas and analytical model. To this end, the well-known commercial software SAP2000 [24] is used and a frame with the height of 2.5 m and the bay length of 4.0 m is prepared. The dimensions and the analysis model of the retrofit system are depicted in Fig. 4. In order to reduce elastic deformations and properly dissipate seismic energy, a relatively strong frame section needs to be chosen. The H-sections H400 x 400 x 13 x 21 (mm) are deemed suitable for both horizontal and vertical members and are modeled using the conventional elastic elements. It is assumed that the hinges consist of four viscoelastic pads, $n = 4$, with the inner and outer radii of $r_i = 15$ mm and $r_o = 200$ mm, respectively, and the thickness of $T = 10$ mm. Based on ASCE 41–13 [25], the material properties of viscoelastic dampers and related calculations are required to be based on the fundamental period of the structure. According to the results of previous experimental tests [23], the storage and loss moduli of the viscoelastic pads are obtained as $G^\prime = 1.13$MPa and $G^\prime = 0.41$MPa. The hinges are modeled using two nodes exactly placed at the same location with constrained translational degrees of freedom. The viscoelastic hinges are modeled following the Kelvin-Voigt model with rotational damper exponential and linear elastic elements in parallel at the hinge location. The damper model provided in SAP2000 is the well-known Maxwell model which is used to simulate viscous dampers. This model consists of a dashpot and a spring in series. From the practical perspective, a very large stiffness should be assigned to the spring of the Maxwell model to use its dashpot for the Kelvin-Voigt model. The properties of the frame and viscoelastic hinges are summarized in Table 1. The rotational stiffness $K_o$ and the rotational damping coefficient $C_o$ are calculated in accordance with the derived formulas and are assigned to hinge elements.

![](https://via.placeholder.com/150)

Fig. 4. Details of the analysis model of the frame in SAP2000 (unit: millimeter).
4. Application of the proposed system to a case study structure

4.1. Details of the structure

In this section, the seismic performance of a case study structure is evaluated before and after seismic retrofit using the developed system to further validate its applicability and efficiency. The case study structure is a 5-story steel building constructed in Korea in the 80’s using only gravity loads. The plan of this structure is shown in Fig. 6 and the story height of all floors is 3.6 m. The cross-sectional details of beams and columns are listed in Table 2 and their locations are indicated in Fig. 6. The yield strength of steel is 235 MPa with the elastic modulus of $2.1 \times 10^5$ MPa, and the dead and live loads are 5.0 kN/m² and 2.5 kN/m², respectively. It is assumed that the structure is located at a site with the response spectrum parameters of $S_{\text{XS}} = 0.75$ g and $S_{\text{X1}} = 0.43$ g for the Maximum Considered Earthquake (MCE) hazard level, corresponding to 2 % exceedance probability in 50 years. It is assumed that the structure needs to satisfy the 2.0 % interstory drift ratio under this hazard level.

The analysis model of the structure is established in SAP2000. The gravity loads are applied following the load combination stipulated by ASCE 41–13 and are directly distributed to beams by modeling slabs using the membrane elements. All beam-column elements are modeled using the lumped plasticity assumption and plastic hinges in accordance with ASCE 41–13, and a rigid diaphragm constraint is applied at each story level.

Table 1
Properties of the analyzed retrofit system.

<table>
<thead>
<tr>
<th>$n$</th>
<th>$G'$ (MPa)</th>
<th>$G''$ (MPa)</th>
<th>$r_s$ (mm)</th>
<th>$r_i$ (mm)</th>
<th>$T$ (mm)</th>
<th>$H$ (m)</th>
<th>$\delta$ (mm)</th>
<th>$\omega$ (rad/s)</th>
<th>$K_\theta$ (kN m)</th>
<th>$C_\theta$ (kN m s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>1.13</td>
<td>0.41</td>
<td>200</td>
<td>15</td>
<td>10</td>
<td>2.5</td>
<td>50</td>
<td>$\pi$</td>
<td>1143</td>
<td>132</td>
</tr>
</tbody>
</table>

![Fig. 5. Comparison of the hysteresis curves obtained from the derived theoretical formulas and SAP2000.](image)

Table 2
Cross sectional details of the structure.

<table>
<thead>
<tr>
<th>Element</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>H310 × 200 × 200 × 200</td>
</tr>
<tr>
<td>C2</td>
<td>H310 × 200 × 200 × 200</td>
</tr>
<tr>
<td>C3</td>
<td>H300 × 300 × 10 × 10</td>
</tr>
<tr>
<td>B1</td>
<td>H300 × 200 × 8 × 13</td>
</tr>
<tr>
<td>B2</td>
<td>H300 × 200 × 10 × 16</td>
</tr>
<tr>
<td>G1</td>
<td>H350 × 175 × 7 × 11</td>
</tr>
<tr>
<td>G3</td>
<td>H350 × 175 × 7 × 11</td>
</tr>
<tr>
<td>G3A</td>
<td>H400 × 200 × 8 × 13</td>
</tr>
<tr>
<td>G6</td>
<td>H400 × 200 × 8 × 13</td>
</tr>
<tr>
<td>G7</td>
<td>H400 × 200 × 9 × 14</td>
</tr>
<tr>
<td>G8</td>
<td>H400 × 200 × 8 × 13</td>
</tr>
</tbody>
</table>

![Fig. 6. Plan of the 5-story case study structure.](image)
4.2. Seismic performance before retrofit

The case study structure is evaluated using the Eigenvalue analysis, and it is observed that its first mode is in the x-direction with the period of 1.71 s and the mode shape of \( \phi^x = [0.21, 0.49, 0.73, 0.91, 1] \). This provides a general insight into the dynamic seismic performance of the structure. It shows that elastic drifts in the 1st mode are concentrated at the first three stories, and accordingly it is expected that these floors experience greater interstory drifts in nonlinear time history analysis. The second period of the structure is 1.64 s with a mode shape in the perpendicular direction. In order to evaluate the seismic performance of the structure, 22 pairs of earthquake records recommended by FEMA P695 [28] are obtained from the PEER NGA database [29] and are scaled to the MCE response spectrum accordingly. Some details of these records, such as the earthquake name, year, magnitude, shear wave velocity to the 30 m depth \( V_{s30} \), and the maximum value of the two horizontal components of the peak ground acceleration (PGA) and peak ground velocity (PGV) are summarized in Table 3. The records are normalized so that their peak ground velocities match the median PGV to eliminate the differences in magnitude, distance to source, site conditions while keeping the record-to-record and aleatory variability. Then the normalized suite is scaled at the fundamental period of the structure in such a way that the median spectral acceleration of the square root of the sum of the squares (SRSS) spectra matches the MCE design spectrum. The MCE hazard level spectrum defined earlier is multiplied by 1.3 to take into account the safety factor for scaling and the requirements of the Korean Building Code. The MCE spectrum and the scaled SRSS spectra of the earthquake records are shown in Fig. 7.

The structure is subjected to the 22 pairs of earthquake records with the 5% inherent Rayleigh damping, and its maximum interstory drift ratios (MIDRs) in the x- and y-directions are shown in Fig. 8. It is observed that the MIDRs are between 0.6% and 2.9% in the y-direction. In the x-direction, the MIDRs are between 0.3% and 5.9% with their mean value exceeding the 2.0% interstory drift ratio limit state at the first story. Drift ratios greater than 3.0% are usually associated with instability and global collapse, and their simulation is also unreliable due to high nonlinearity and convergence issues. Hence, the structure is retrofitted using the proposed system and its performance is studied in more detail in both before and after retrofit cases.

![Fig. 7. SRSS spectra of the 22 earthquake pairs scaled to the MCE hazard level spectrum.](image)

<table>
<thead>
<tr>
<th>Record Sequence Number</th>
<th>Earthquake name</th>
<th>Year</th>
<th>Magnitude (( M_w ))</th>
<th>( V_{s30} ) (( m/sec ))</th>
<th>( PGA_{\text{max}} ) (( m^2/s^2 ))</th>
<th>( PGV_{\text{max}} ) (( cm/s ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>68</td>
<td>San Fernando</td>
<td>1971</td>
<td>6.6</td>
<td>316</td>
<td>0.21</td>
<td>19</td>
</tr>
<tr>
<td>125</td>
<td>Friuli, Italy</td>
<td>1976</td>
<td>6.5</td>
<td>505</td>
<td>0.35</td>
<td>31</td>
</tr>
<tr>
<td>169</td>
<td>Imperial Valley</td>
<td>1979</td>
<td>6.5</td>
<td>242</td>
<td>0.35</td>
<td>33</td>
</tr>
<tr>
<td>174</td>
<td>Imperial Valley</td>
<td>1979</td>
<td>6.5</td>
<td>196</td>
<td>0.38</td>
<td>42</td>
</tr>
<tr>
<td>721</td>
<td>Superstition Hills</td>
<td>1987</td>
<td>6.5</td>
<td>192</td>
<td>0.36</td>
<td>46</td>
</tr>
<tr>
<td>725</td>
<td>Superstition Hills</td>
<td>1987</td>
<td>6.5</td>
<td>317</td>
<td>0.45</td>
<td>36</td>
</tr>
<tr>
<td>752</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.9</td>
<td>289</td>
<td>0.53</td>
<td>35</td>
</tr>
<tr>
<td>767</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.9</td>
<td>350</td>
<td>0.56</td>
<td>45</td>
</tr>
<tr>
<td>828</td>
<td>Cape Mendocino</td>
<td>1992</td>
<td>7.0</td>
<td>422</td>
<td>0.55</td>
<td>44</td>
</tr>
<tr>
<td>848</td>
<td>Landers</td>
<td>1992</td>
<td>7.3</td>
<td>353</td>
<td>0.42</td>
<td>42</td>
</tr>
<tr>
<td>900</td>
<td>Landers</td>
<td>1992</td>
<td>7.3</td>
<td>354</td>
<td>0.24</td>
<td>52</td>
</tr>
<tr>
<td>953</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>356</td>
<td>0.52</td>
<td>63</td>
</tr>
<tr>
<td>960</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>326</td>
<td>0.48</td>
<td>45</td>
</tr>
<tr>
<td>1111</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>6.9</td>
<td>609</td>
<td>0.51</td>
<td>37</td>
</tr>
<tr>
<td>1116</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>6.9</td>
<td>256</td>
<td>0.24</td>
<td>38</td>
</tr>
<tr>
<td>1148</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>7.5</td>
<td>523</td>
<td>0.22</td>
<td>40</td>
</tr>
<tr>
<td>1158</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>7.5</td>
<td>282</td>
<td>0.36</td>
<td>59</td>
</tr>
<tr>
<td>1244</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>7.6</td>
<td>259</td>
<td>0.44</td>
<td>115</td>
</tr>
<tr>
<td>1485</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>7.6</td>
<td>205</td>
<td>0.51</td>
<td>39</td>
</tr>
<tr>
<td>1602</td>
<td>Duzez, Turkey</td>
<td>1999</td>
<td>7.1</td>
<td>294</td>
<td>0.82</td>
<td>62</td>
</tr>
<tr>
<td>1633</td>
<td>Manjil, Iran</td>
<td>1999</td>
<td>7.4</td>
<td>724</td>
<td>0.51</td>
<td>54</td>
</tr>
<tr>
<td>1787</td>
<td>Hector Mine</td>
<td>1999</td>
<td>7.1</td>
<td>726</td>
<td>0.34</td>
<td>42</td>
</tr>
</tbody>
</table>

Table 3: Details of the earthquake ground motion records used in this study.

4.3. Seismic performance after retrofit

As the structure shows larger mean drift ratio than the target value in the x-direction, it is retrofitted with the steel frames with rotational VED in the x-direction. The sufficiency of the considered distribution can be validated using further analyses and controlling MIDRs. Based on the maximum interstory drift ratios obtained earlier, the dampers are better to be installed on the first two stories which possess greater interstory drift ratios. Therefore, four bays at the first and the second stories are chosen to be equipped with the proposed frames as depicted in Fig. 6. These bays are 6 m wide and the story height is 3.6 m which is greater than the example frame analyzed earlier. As mentioned earlier, the lateral capacity has an inverse relationship with the square of height and this can significantly reduce the capacity of the frame. Therefore, slightly stronger section H414 × 405 × 18 × 28 is chosen with ten
5 mm viscoelastic pads. It has been observed in previous experimental studies that the storage and loss moduli of viscoelastic dampers are decreased at large strains [23]. The maximum interstory drift ratios need to be limited to 2.0 % which equals the rotation of viscoelastic hinges. Given the outer radius \( r_o = 200 \) mm and the thickness \( T = 10 \) mm, the shear strain of the pads equals \( \gamma = 0.02 \times \frac{200}{10} = 0.4 \). The storage and loss moduli can be approximated using this strain and the natural frequency of the structure and the data from previous experimental studies. The storage and loss moduli of the viscoelastic damper are assumed to be \( G' = 0.68 \) MPa and \( G'' = 0.24 \) MPa based on the previous experimental study conducted by the authors on the viscoelastic material made of high damping rubber [23]. The frame with the given properties has the maximum capacity of 81 kN under the 2.0 % drift ratio corresponding to the lateral displacement of 72 mm.

After installing 8 frames at the designated locations of the structure using rigid links for seismic retrofit, the seismic performance of the structure is reevaluated. The first mode of the structure is changed to the \( y \)-direction with the period of 1.64s. As the period of the structure changes after the retrofit, the earthquake records need to be rescaled accordingly for seismic evaluation. The target spectrum is the same 1.3 x MCE spectrum, to which the median SRSS spectrum is scaled at the natural period of the retrofitted structure. The MIDRs in both directions are depicted in Fig. 9, where it can be observed there are no tangible changes in the MIDRs in the \( y \)-direction. MIDRs in the \( x \)-direction are between 0.3 % and 4 % and the mean maximum value of the MIDRs is 1.8 % at the first story.

The MIDRs of the case study structure in the \( x \)-direction subjected to the ground motion records are compared before and after retrofit in Fig. 10. The records are denoted by the event names and the record sequence numbers (RSN) which are used in the PEER NGA database to access them more easily. The maximum MIDRs are between 0.5 % and 5.9 % with the mean value of 2.2 %. These values are decreased after retrofit and the MIDRs are between 0.5 % and 4 % with the mean value of 1.8 %. The mean value of MIDRs at critical stories are reduced by 21 %. The maximum and minimum reduction in the MIDRs correspond to the RSN169-Imperial Valley and the RSN1602-Duzce with 68 % and 0 % reduction, respectively.

The top displacement time histories of the structure under the

![Fig. 8. Maximum interstory drift ratios in the \( x \)- and \( y \)-directions before retrofit.](image)

![Fig. 9. Maximum interstory drift ratios in the \( x \)- and \( y \)-directions after retrofit.](image)
RSN752-Loma Prieta and the RSN169-Imperial earthquake records are shown in Fig. 11. As observed in MIDRs, there is not any significant change in the seismic performance in the y-direction due to the installation of the frames just in the x-direction. It is seen that the top displacements in the two cases are generally reduced while there is a considerable reduction in residual displacements. The residual displacements in the x-direction before and after retrofit are illustrated in Fig. 12.

The residual displacements at the top story before retrofit are between 12 mm and 223 mm with the mean value of 78 mm. After retrofit these displacements are decreased to between 1 mm and 166 mm with the mean value of 50 mm. It is observed that there is a slight increase in the residual displacement for some cases like the RSN68-San Fernando ground motions; but overall residual displacements are reduced with the maximum reduction for the RSN1602-Duzce and RSN169-Imperial Valley records.
Another important parameter that should be studied to validate the efficiency of the retrofit system is the energy dissipation of members which demonstrate the extent of irreversible damage. The input seismic energy, modal damping energy, and the seismic energy dissipated by the retrofit devices are shown in Fig. 13 for the RSN1158-Kocaeli ground motion record. It can be observed that 71% and 29% of the seismic input energy are dissipated by the structural members and by modal damping, respectively, before retrofit. On the contrary, the share of the structural members in dissipating the input energy is reduced to 12%, and the installed rotational VEDs dissipate 58% of the seismic energy. The head-to-head assessment of the seismic energy dissipation is carried out in more detail in Fig. 14. Before retrofit the energy dissipated by the inelastic deformation of structural members is between 26% and 71% of the input seismic energy with average value of 51%. The share of inelastic energy dissipated by the structural members is reduced by 81% on average and is between 3% to 22% with the mean value of 9% after retrofit. Before retrofit the maximum inelastic energy share belongs to the RSN 1158-Kocaeli record whose energy dissipation time history was explained earlier. The mean inelastic energy is 51% before retrofit and is reduced to 9% after retrofit. This shows that the structural members remain almost in the elastic range and the input energy is mostly dissipated by the VEDs. Overall, it can be concluded that the damage to structural members is significantly reduced after application of the proposed retrofit system.

Compared with rigid steel frame retrofit system, the proposed system will provide less stiffness and strength; however, as the induced seismic force is less than the rigid steel frame system, less number of anchor bolts are needed to install the proposed system into existing structures. The added stiffness and damping of the proposed system can be enhanced by increasing the number or the diameter of the viscoelastic pads installed in one joint. The proposed system may be effectively applied in the structures which need additional energy dissipation capacity rather than added stiffness or strength.

It should be noted that the steel sections need to be strong enough to reduce the elastic deformation so that the proper rotation can be exerted to the viscoelastic hinges for energy dissipation. It should be also pointed out that comparison of the cost-effectiveness of this system with other damping devices need information such as the manufacture and

Fig. 12. Head-to-head comparison of residual displacements in the x-direction before and after retrofit.

Fig. 13. Energy dissipation time history before and after retrofit under the RSN1158-Kocaeli earthquake.
installation costs, which was not tried in this study.

5. Conclusions

In this study, a retrofit system consisting of a frame with viscoelastic hinges at corners was proposed and its efficiency and capabilities were evaluated in a theoretical framework. To this end, first the formulas for its mechanical behavior and hysteresis behavior were derived at length. The analysis modeling of the system was established in the well-known commercial software SAP2000 and an example frame was subjected to a harmonic load. Details of the frame and the calculations were fully explained. The analytical model and the theoretical formulation were validated by comparing the hysteresis behavior and other parameters obtained from the two analysis approaches. The results showed that both methods are in agreement and the derived formulas can sufficiently capture the hysteresis behavior of the proposed system.

In order to further assess the applicability and efficiency of the proposed system, it was used in seismic retrofit of a case study structure built in the 80 s in Korea. The seismic performance of the structure under the Maximum Considered hazard level was evaluated in terms of maximum interstory drift ratio, residual displacement, and energy dissipation. As the structure showed maximum interstory drift ratios greater than the considered 2.0 % drift ratio in the x-direction, eight frames were installed in this direction. The seismic performance of the retrofitted structure was investigated again and was compared with the bare structure. It was observed that in addition to reducing the maximum interstory drift ratios, the residual displacements can also be decreased. The inelastic energy dissipated by the nonlinear deformation of structural members accounted for 52 % of the total input seismic energy on average, which was reduced to 9 % after retrofit. This indicates that the structure remains mostly in the elastic range with insignificant damage. Overall, the proposed system seems to be theoretically sound and it needs future experimental investigations for further developments and application in practice.

CRediT authorship contribution statement

Jinkoo Kim: Conceptualization, Funding acquisition, Methodology, Project administration, Supervision, Writing – original draft. Seunghee Park: Funding acquisition, Project administration, Supervision, Writing – review & editing.

Declaration of Competing Interest

No conflict of interest

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References


[28] PEER, PEER NGA Database, PEER Ground Motion Database 2014. [https://ngawest2.berkeley.edu/].