Seismic retrofit design method using friction damping systems for old low- and mid-rise regular reinforced concrete buildings

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\textbf{Abstract}

In seismically active regions, many old low- to mid-rise reinforced concrete (RC) buildings are in use that have poor reinforcement details and suffer from material deterioration. Because these buildings are vulnerable to earthquakes, they represent a threat to human lives and economy. To impart such a building with satisfactory seismic performance, a friction damping system can be used, which consists of dampers and a braced frame. In this study, a method for the design of friction damping systems is proposed for the seismic retrofit of old low- to mid-rise regular reinforced concrete (RC) buildings. The proposed method is verified using a six-story RC building designed considering only gravity loads.

\section{1. Introduction}

Many old reinforced concrete (RC) buildings exist in densely populated areas. They usually have poor reinforcement details such as lap splices of column longitudinal bars located just above the floor level, widely spaced column transverse bars, short lap splice length, and insufficient development length of reinforcing bars. In addition, materials deteriorate with time. Such insufficient reinforcement details and deteriorated materials increase the vulnerability of old RC buildings to earthquakes. Most of these structures were designed considering only gravity loads, not seismic loads [1], and have columns weaker than adjacent framing elements that could cause inelastic displacement demands to concentrate on the columns during earthquakes [2]. During large earthquakes, many old RC buildings behaved poorly, suffering significant damage and collapse [3,4].

To improve the seismic performance of old building structures, various types of passive damping systems have been developed [5]. A damping system is defined as the collection of structural elements that includes all individual damping devices, all structural elements or bracing required to transfer forces from damping devices to the base of a structure [6]. Damping systems improve seismic performance and are also simple to install. Unlike conventional seismic retrofit procedures, those using dampers do not require significant demolition and replacement of existing elements; therefore occupancy and building operation are not significantly interrupted during the process of installing damping systems. Friction dampers are one type of displacement-dependent passive damping devices developed to efficiently dissipate the seismic input energy exerted on a structure. The response of a displacement-dependent damping device is primarily a function of the relative displacement between each end of the device, and is substantially independent of the relative velocity (Chapter 18 of ASCE 7–10 [7]). Because friction damping devices require lower cost and less maintenance effort compared to other damping devices, they have been widely used for the seismic retrofit of existing buildings [8,9].

Many friction damping systems have been developed [10–12]. In buildings with friction dampers, most seismic input energy is dissipated by the dampers during earthquakes, preventing significant damage in main structures [8,13]. Monir and Zeynali [14] reported that friction dampers reduced the lateral displacement and base shear force demands for multi-story building frames. The responses of torsionally irregular structures can also be reduced by using friction damping devices, resulting in improved seismic performance [15]. Seismic performance was evaluated using nonlinear response history analyses for structural systems with various types of friction and hysteretic dampers [16–18].
To guarantee that friction damping systems impart satisfactory seismic performance to existing buildings, a proper method should be developed for the design of damping systems [19,20]. In ASCE/SEI 7–10 [7], a design procedure is provided for new structures with damping systems, whereby seismic demands are estimated based on an equivalent linearization method using effective damping and secant stiffness [21,22]. However, the equivalent linearization procedures can lead to multiple results for individual ground motions [23], and produce maximum displacements much different than those obtained from nonlinear response history analyses (RHAs) [24,25]. Also, this procedure does not necessarily guarantee convergence in analyses.

In the present study, a simple and accurate method is proposed for the design of friction damping systems consisting of braced frames and friction dampers for the seismic retrofit of existing old low- to mid-rise RC buildings whose seismic responses are usually dominated by the fundamental mode. Using the proposed design method, friction damping systems are designed for a structure to reduce its maximum inelastic displacement to ensure that it remains within a limiting displacement during a design-level earthquake. The limiting displacement considered herein is the lateral displacement in excess of which the structure experiences a drop in lateral strength. The maximum inelastic displacement demands for original and retrofitted structures are estimated herein using the elastic displacement of an equivalent single-degree-of-freedom (SDF) system and an inelastic displacement ratio, rather than using the equivalent linearization procedure with secant stiffness and equivalent damping. As a model structure, we study a six-story RC building that was designed considering only gravity loads. To verify the proposed design method, we compare the seismic performance of the original and retrofitted model structures based on the results obtained from nonlinear RHA.

2. Structures retrofitted using damping systems with braces and friction dampers

Structures can be retrofitted using friction damping systems to improve their seismic performance during earthquakes. A friction damper activates and dissipates energy when the friction force exerted on the dry surface of the friction damper reaches its maximum friction force (slip force); however, before its activation, the friction damper acts as a rigid element, which normally increases the lateral stiffness of the structure.

A damping device is often installed with a braced frame (Fig. 1). Recently, toggle braces with dampers (Fig. 1d) have been developed for improving the efficiency of dampers [26].

Fig. 2 illustrates original old structure, damping system (braced frame with dampers), and retrofitted structure (original structure retrofitted with the damping system) and their respective pushover curves. The ordinate and abscissa in the pushover curves are the base shear force (V) and the roof displacement (ur), respectively. In Fig. 2, the subscripts O, D, and R respectively refer to the original structure, the damping system and the retrofitted structure. Old RC structures constructed without consideration of seismic loads are vulnerable to earthquakes. Since these structures are not sufficiently ductile, their strength may drop abruptly after reaching their maximum lateral strength (Fig. 2a).

In contrast, damping systems with braced frames and friction dampers (Fig. 2b) behave in a ductile manner similar to that of elastoplastic systems. In Fig. 2b, the elastic stiffness of the damping system (KD) is the same as that of a braced frame because a damper placed in the brace acts as a rigid element before it starts to slip. If V in the damping system reaches the slip force (Vd), the dampers start to slip and dissipate energy due to friction. Once the dampers...
start to slip, the lateral stiffness of the damping system becomes zero. All members of the braced frame (Fig. 2b) are assumed to be pin-connected.

Since the retrofitted structure is a combination of two parallel systems, namely the original structure (Fig. 2a) and the damping system (Fig. 2b), the pushover curve for the retrofitted structure can be represented by a trilinear curve (Fig. 2c).

The first slope ($K_1$) is the sum of the elastic stiffness of the original structure ($K_o$) and that of the damping system ($K_d$); this represents the increased stiffness of the retrofitted structure, which leads to a shorter fundamental period ($T$) compared to that of the original structure. The weight of the damping system is much less than that of the original structure, and thus can be neglected.

In a retrofitted structure, if the base shear force ($V$) reaches $V_{d,x}$ ($=K_{o,d}(A_0 + V_{d,y})$), dampers in the structure start to slip and dissipate energy. Once the dampers slip, the stiffness of the retrofitted structure becomes $K_o$, the lateral stiffness of the original structure (Fig. 2c). If $V$ attains the maximum strength of the retrofitted structure [$V_{max,r} = V_{max, o} + V_{d,y}$], a sudden strength drop occurs as shown in Fig. 2c, similar to that of the original structure (Fig. 2a). Note that the strength drop occurs at the same roof displacement ($u_{y,off}$) in both the original and the retrofitted structure. In the present study, a method is proposed for the design of damping systems to limit the maximum roof displacement for the structure within $u_{y,off}$.

### 3. Estimating inelastic displacements using the inelastic displacement ratio $C_x$

The degree of damage in a structure during an earthquake is closely related to its lateral displacement [27]. Nonlinear RHA can be used to accurately estimate displacement demands for structures subjected to ground motions. However, nonlinear RHA requires considerable computational effort and detailed knowledge on analytical modeling; therefore, it is difficult to use nonlinear RHA for retrofit design practices that may require iterative processes to obtain a final design.

In this study, the maximum displacement of multi-degree-of-freedom (MDF) systems is estimated using equivalent SDF systems. To increase simplicity, the maximum inelastic displacement demand ($D_{m}$) of the equivalent SDF system is calculated using the maximum elastic displacement demand ($D_{e}$) of the system and the inelastic displacement ratio ($C_x$) [Eq. (1)].

$$D_m = C_x \times D_e$$  \hspace{1cm} (1)

Fig. 3 shows a force–displacement curve of an inelastic SDF system and a corresponding elastic SDF system. Both systems have the same natural frequency ($\omega_n = 2\pi/T_n$). In this figure, $A$ and $D$ are the normalized lateral force ($V/m$) and the displacement of the SDF system, respectively, where $m$ is the mass of the system.

![Fig. 3. Force–displacement relations for inelastic and corresponding elastic SDF systems.](image)

### 4. Proposed design method for friction damping systems

In the present study, a seismic retrofit design method is proposed for old low- and mid-rise regular RC building structures using a friction damping system to protect them from brittle failure during design-level earthquakes. The damping system consists of a braced frame and friction dampers (Fig. 2b). In the proposed design method, member sections for braces and slip forces for friction dampers are determined. The proposed procedure is described step-by-step as follows. Appendix A illustrates the entire proposed procedure.

#### 4.1. Seismic performance evaluation of old structures

To decide whether a seismic retrofit is required for old RC structures, the seismic deficiency of these structures should be first investigated against design-level seismic loads by conducting a seismic performance evaluation. For this purpose, the pushover curve of the structure is constructed by conducting nonlinear static analyses for the structure when subjected to lateral forces of a vertical distribution following $S_1 (=m\phi_1)$, where $\phi_1$ is the first mode vector and $m$ is the mass matrix. Fig. 4a shows a pushover curve for an original structure before a seismic retrofit. As mentioned previously, in an old RC structure, a sudden drop in lateral strength occurs when the roof displacement ($u_r$) reaches $u_{y,off}$. To avoid a sudden drop in lateral strength during a design-level earthquake, the maximum inelastic displacement of the structure should not exceed $u_{y,off}$. Therefore, in this study, the performance objective is defined as follows: $u_r$ should be less than $u_{y,off}$ for design-level seismic loads. Note that more stringent performance objective can also be used.

For simplicity, this study uses equivalent SDF systems for estimating the maximum inelastic displacements of MDF systems. The equivalent SDF system has the same natural vibration period ($T_n$) as the fundamental period of the MDF system. The pushover curve of a MDF system should be transformed to the capacity curve of corresponding equivalent SDF system. For this purpose, the pushover curve of a MDF system is idealized by using a bilinear curve as shown in Fig. 4a. The first slope of the idealized pushover curve is obtained from a line connecting the origin and a point on

<table>
<thead>
<tr>
<th>$a$</th>
<th>0.05</th>
<th>0.15</th>
<th>0.25</th>
<th>0.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\xi$</td>
<td>0.05</td>
<td>0.116</td>
<td>0.160</td>
<td>0.093</td>
</tr>
<tr>
<td>$\xi$</td>
<td>0.30</td>
<td>0.195</td>
<td>0.160</td>
<td>0.143</td>
</tr>
</tbody>
</table>

The inelastic displacement ratio can be calculated using Eq. (2), which was proposed by [28] for bilinear systems.

$$C_r = 1 + \frac{(1 - \eta)(R - 1)}{R}$$  \hspace{1cm} (2)

$$a = a_0(2 + 0.45R)$$  \hspace{1cm} (3)

where $\eta$ is the post yielding slope, $R$ is the yield strength reduction factor ($=A_f/A_y$), $T_n$ is the fundamental period ($=2\pi/\omega_n$), $T_s$ is the characteristic period ($=S_0/S_0$), $S_0$ and $S_0$ are the design spectral response acceleration parameter at short and 1 s periods, respectively, and $\xi$ is the equivalent viscous damping ratio. In Eq. (3), the values for $a_0$ can be determined using Table 1. Linear interpolation can be used to find the intermediate values of $a_o$.
Since the proposed procedure is developed to design the friction damping systems for satisfactory global seismic performance, constituent members should be also checked although the seismic performance of a structure is globally satisfactory. Member-level evaluation can be conducted according to the procedure provided in ASCE 41–13 [22]. If there are members that do not meet the acceptance criteria, member-level retrofit could be required.

4.2. Retrofit design strategy using friction damping systems

As mentioned earlier, the objective of the retrofit design is to limit the maximum displacement of a structure to remain below a displacement that causes a sudden strength drop, in the case of design-level seismic loads. If an original structure does not satisfy the performance objective, a seismic retrofit design is conducted with a friction damping systems (Fig. 2c). To determine whether the performance objective is satisfied, the maximum inelastic displacement demand (\(D_m\)) of equivalent SDF systems developed for the retrofitted structures is compared to a limiting displacement (\(D_f\)) [Eq. (8)].

\[
D_m = C_k D_e \leq D_f
\]

Rearranging Eq. (8),

\[
C_k \leq D_f / D_e
\]

In general, the value of \(C_k\) is larger than 1; therefore, the elastic displacement demand (\(D_e\)) of the equivalent SDF system is generally less than \(D_f\). In this study, \(D_e\) is set to 0.9 \(D_f\); this corresponds to \(C_k\) of 1.1 according to Eq. (9). To propose a value for \(D_e\) according to a given level of safety margin, reliability analyses need to be performed. Since reliability analyses are beyond of the scope of this study, no further study is conducted to determine the target design displacements according to safety margin.

If a retrofitted structure is designed to have \(D_e\) equal to 0.9 \(D_f\), the natural frequency (\(\omega_{n, k}\)) of the retrofitted structure can be estimated as shown in Fig. 5b. For \(C_k\) of 1.1, the \(R\) factor can be determined using Eq. (2). Then, a force (\(A_{n, k}\)) required to activate the friction dampers can be calculated (\(A_{n, k} = \omega_{1, k} / R\)). In Fig. 5b, the second slope of the capacity curve is the same as that of the original structure (\(\omega^2_{n, k}\)). As mentioned previously, \(D_f\) of the retrofitted structure is the same as that of the original structure (see Fig. 1).

The capacity curve of the equivalent SDF system can be converted to a pushover curve of the retrofitted MDF structure using Eqs. (10) and (11) for the design of friction damping systems (Fig. 5c). These equations are obtained by simply rearranging Eqs. (5) and (6) as follows.

### Fig. 4. Pushover curves for MDF systems, and capacity and demand curves for equivalent SDF systems.
4.3. Design of friction damping systems

For seismic retrofitting, friction damping systems are placed as shown in Fig. 2. During earthquakes, the damping system and the original structure act as parallel spring systems. Therefore, the lateral stiffness ($K_D$) of the damping system and a lateral force activating the friction dampers ($V_{d,r}$) are calculated using Eqs. (12) and (13), respectively.

$$K_D = K_X - K_0$$  \(12\)

$$V_{d,r} = V_{d,r} - K_0 u_{r_{\text{rid}}}$$  \(13\)

where $K_X$ and $K_0$ are the lateral stiffnesses of the retrofitted and original structures, respectively (Fig. 2), $V_{d,r}$ is the base shear force activating the friction dampers in the retrofitted structure, and $K_0 u_{r_{\text{rid}}}$ is a base shear force resisted by the original structure within the retrofitted structure.

The lateral seismic force ($f_i$) induced at level $i$ of the damping system corresponding to $V_{d,r}$ is calculated using Eq. (14) (Fig. 6).

$$f_i = \frac{m_i \phi_{1,i}}{\sum_{j=1}^{N} m_j \phi_{1,j}} V_{d,D}$$  \(14\)

where $\phi_{1,i}$ is a component of the first mode vector for level $i$ of the original system. The shear force ($V_i$) in story $i$ is calculated using Eq. (15).

$$V_{i} = \sum_{j=1}^{N} f_j$$  \(15\)

The floor displacement ($u_i$) and story drift ($\Delta_i$) induced by $V_{d,r}$ can be obtained using Eqs. (16) and (17), respectively.

$$u_i = u_{r_{\text{rid}}} \frac{\phi_{1,i}}{\phi_{1,r}}$$  \(16\)

$$\Delta_i = u_i - u_{i-1} = u_{r_{\text{rid}}} \frac{\phi_{1,i} - \phi_{1,i-1}}{\phi_{1,r}}$$  \(17\)

The portion of $V_i$ transmitted to individual braces ($F_i$) is calculated using Eq. (18). The sectional area ($A_i$) of the braces can be determined using Eq. (19).

$$F_i = \frac{V_i}{\cos \theta_i}$$  \(18\)

$$A_i = \frac{F_i \sqrt{h_i^2 + l^2}}{E \theta_i}$$  \(19\)

where $\theta_i$ is the angle of a brace placed in story $i$, and $l_i$ is the axial displacement of the brace ($= \Delta_i \cos \theta_i$). It is also noted that a friction damper in story $i$ starts to move when the axial force reaches $F_i$; therefore, $F_i$ is the activation force for the friction damper in story $i$. Note that brace member sections should have sufficient tensile and compressive strength greater than $F_i$ to prevent the braces from yielding and buckling before friction dampers start to move.
5. Verification of the proposed method using a six story RC building

Herein we consider a six-story RC office building to verify the proposed retrofit design method using a friction damping system. Fig. 7 shows the floor plan and elevation of the building.

The building is designed considering only gravity loads, according to ASCE 7–10 [7]. Dead loads for typical floors and the roof are 5.65 and 5.13 kPa, respectively, whereas a live load of 2.4 kPa is used for all floors including the roof. The building is classified into risk category I. The frame is illustrated in Fig. 7 is selected as the model frame. The story height and span length of the frame are 3.6 m and 5.5 m, respectively. The compressive strength of the concrete is 21 MPa, and the yield strengths of reinforcement and braces are both 400 MPa.

The building is assumed to be located in a seismically active area with SDS of 0.49 g and SDs of 0.29 g, respectively. The soil condition of the site is assumed to be of site class D. The building is classified into seismic design category (SDC) D according to ASCE 7–10 [7].

Fig. 8 shows the analytical models used in this study. Columns and beams are modeled using fiber section elements (Fig. 8a). The beam-column joints are modeled as rigid joint, which may not simulate damage accumulated in the joints of old RC building structures during large earthquakes. Failure in the joints in old RC structures during earthquakes has been reported [30]. The use of simple analytical joint model may produce un-conservative results. For conducting more accurate seismic performance evaluation for old RC structures, a realistic joint model needs to be used [31,32]. However, since this example is provided to explain the retrofit procedure proposed in this study, simple models are used although they have limitation in accuracy.

Braces in damping systems are modeled using uniaxial section elements. A brace with a friction damper can be modeled using an elastoplastic spring because the friction damper starts to slip when an axial force reaches a slip force \( F_s \) of the damper. Analyses are conducted using OpenSees software [33].

5.1. Seismic performance of the model structure

For constructing pushover curves, nonlinear static analyses are conducted for the model frame using a lateral force distribution \( m \phi_1 \). The period of the first mode of the frame is 1.85 s and its corresponding mode vector \( \phi_1^T \) is \[0.11, 0.28, 0.46, 0.67, 0.89, 1.0\]. Fig. 9a shows the actual and idealized pushover curves of the frame. Once the base shear force reaches 310 kN, a sudden strength drop occurs. The roof displacement corresponding to this strength drop \( u_{rf} \) is 144 mm. In this study, if the roof displacement of the structure under design-level earthquake ground motions exceeds \( u_{rf} \), the structure is retrofitted using a friction damping system.

The pushover curve is converted to the capacity curve of an equivalent SDF system using Eqs. (5) and (6). In Fig. 9b, the capacity curve is plotted with a demand curve obtained from a design acceleration response spectrum \( (S_{ds} = 0.49g, S_{df} = 0.29g) \). The limiting value \( D_{f} \) of displacements for the equivalent SDF system is 108 mm, which corresponds to \( u_{rf} \) (144 mm) for the original MDF structure. Then, the R factor is determined \( = \frac{A_{f}}{A_{max}} = 1535/1238 = 1.24 \) and the inelastic displacement ratio \( \zeta \) is calculated to be 1.00 using Eq. (2). The second slope of the capacity curve \( \alpha \) and the damping ratio \( \eta \) are respectively set to 0 and 0.05. The elastic displacement demand \( D_{e} \) of the equivalent
SDF system for the original structure is 134 mm (Fig. 9b). Therefore, the inelastic displacement demand \( D_{m-o} \) is 134 mm, which is simply calculated as \( C_R \times D_o = 108 \times 134 = 97 \text{ mm} \). Because \( D_{m-o} \) is larger than \( D_f = 108 \text{ mm} \), seismic retrofitting is required for the model frame.

### 5.2. Design of the friction damping system

Since the original structure is not acceptable as is (\( D_{m-o} > D_f \)), it is retrofitted with a friction damping system consisting of braces and friction dampers. As mentioned in Section 4.2, the elastic displacement demand \( D_f \) of the equivalent SDF system for the retrofitted structure is set to 0.9\( D_f = 108 \text{ mm} \), seismic retrofitting is required for the model frame.

The pseudo spectral acceleration \( A_s \) corresponding to \( D_s \) is 2114 mm/s\(^2\), as obtained from Fig. 9b. Once \( A_s \) and \( D_s \) are obtained, the slope of the capacity curve of the equivalent SDF system for the retrofitted structure can be calculated; this quantity is \( \omega_s^2 = 2114(97) = 21.8 \text{ rad}^2/\text{sec}^2 \). According to this procedure, the natural frequency \( \omega_s = 4.67 \text{ rad/s} \) and period \( T_s = 1.34 \text{ s} \), respectively. Then, the force that activates the friction dampers \( A_{d,s} \) can be calculated as \( A_s \) divided by the R factor for the retrofitted structure; the R factor is determined by using \( C_R \) for the retrofitted system.

A displacement \( D_{s,s} \) corresponding to \( A_{d,s} \) is also calculated as \( D_s \), divided by the R factor. Because the maximum displacement of the retrofitted structure \( D_{s,s} = C_R D_s \) should be less than \( D_f \) to represent acceptable seismic performance, \( C_R \) needs to be 1.1 or less \( (C_R D_{s,s} < D_f) \). To determine the R factor corresponding to \( C_R \) of 1.01, the R factor is determined using Eq. (2) with a second slope \( (\alpha) \) of 0.53 \( = \omega_s^2 / \omega_s^2 \) and \( \xi \) of 0.05, which is 8.0 (Fig. 10). Then, \( A_{d,s} \) and \( D_{d,s} \) are calculated to be 264 mm/s\(^2\) \( = A_{d,s} / R = 2114/8 \) and 12 mm \( = D_{d,s} / R = 97/8 \), respectively.

The capacity curve for the retrofitted system is converted to the pushover curve for the retrofitted MDF system using Eqs. (10) and (11) (Fig. 11). In the retrofitted MDF frame, a roof displacement \( u_{rig} \) and the base shear force \( V_{d,s} \) corresponding to the activation of the friction dampers are 16.0 mm and 86 kN, respectively, whereas the roof displacement \( u_{rig} \) and base shear force \( V_{max,s} \) at the incidence of strength drop are 144 mm and 433 kN, respectively. The elastic stiffness \( (K_s = V_{d,s} / u_{rig}) \) of the retrofitted structure is 5.29 kN/mm \( (=85.9/16.2) \) (Fig. 11a).

### 5.3. Design results of damper systems

The force-pushover curve for the friction damping system can be found from the pushover curve of the retrofitted systems as shown in Fig. 11a. The stiffness of the damping system \( (K_s) \) is calculated using Eq. (12) to be 2.50 kN/mm \( (=5.29 - 2.79) \).

The base shear force \( V_{d,s} \) that activates the friction dampers is calculated using Eq. (13) \( V_{d,s} = V_{d,s} - K_{d,rig-off} = 86.0 - 279 \times 16 = 41 \text{ kN} \). The roof displacement corresponding to \( V_{d,s} \) \( (u_{rig}) \) is also calculated using Eq. (11), and then \( V_{d,s} \) is distributed along the structural height (at each floor level). The distributed force at each floor level is defined as seismic lateral force \( (f) \). The shear force \( (V_s) \) and drift \( (\Delta_s) \) in each story are calculated using Eqs. (15) and (17), respectively.

Then, activation forces for individual friction dampers \( (F_i) \) and sectional areas of braces \( (A) \) in each story \( i \) can be calculated using Eqs. (18) and (19), respectively. Table 2 summarizes the calculated results.

Fig. 11 shows the pushover curve of the retrofitted structure. The fundamental period of the retrofitted structure is 1.37 s, which is close to that of the original model frame \( (=1.35 \text{ s}) \). As shown in Fig. 11, the pushover curves of the original and retrofitted structures do not differ greatly, which indicates that installing the friction damping systems does not greatly change the properties of the original structure. Although, there is no distinctive difference between the pushover curves, the retrofitted structure can dissipate more seismic input energy once the dampers start to move. The seismic performance of the retrofitted system is evaluated in the following section.

### 6. Seismic performance of the original and retrofitted structures

To compare the seismic performance of a structure before and after a seismic retrofit using a friction damping system, nonlinear response history analyses are conducted. The average acceleration method is used for numerical time stepping integration. For this purpose, twenty ground motions recorded at sites with soil profiles

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**Fig. 9.** Pushover curves for a MDF structure and the capacity and demand curves for the corresponding equivalent SDF system.

**Fig. 10.** \( R \) versus \( C_R \).
classified into soil classes D are used as input ground motions, which were selected from the PEER NGA Database [34].

The ground motions are scaled to make their median response spectrum match the target design response spectrum specified in ASCE 7–10 [7] using a procedure developed by Ha and Han [35] as shown in Fig. 12.

\begin{table}[h]
\centering
\caption{Friction force and brace area of the damping system.}
\begin{tabular}{|c|c|c|c|c|c|c|c|}
\hline
Story & \(f_{ij}\) (kN) & \(V_i\) (kN) & \(u_i\) (mm) & \(\Delta_i\) (mm) & \(\delta_i\) (mm) & \(F_i\) (kN) & \(A_i\) (mm²) \\
\hline
1 & 1.33 & 40.60 & 1.78 & 1.78 & 1.49 & 48.52 & 1019 \\
2 & 3.33 & 39.27 & 4.50 & 2.72 & 2.28 & 46.93 & 646 \\
3 & 4.92 & 35.94 & 7.52 & 3.01 & 2.52 & 42.96 & 533 \\
4 & 7.92 & 30.45 & 10.95 & 3.44 & 2.88 & 36.39 & 396 \\
5 & 10.37 & 22.52 & 14.41 & 3.46 & 2.89 & 26.92 & 291 \\
6 & 12.15 & 12.15 & 16.23 & 1.82 & 1.53 & 14.52 & 298 \\
\hline
\end{tabular}
\end{table}

Table 3

Information about selected and scaled ground motions.

\begin{table}[h]
\centering
\caption{Analysis results of original and retrofitted structures using 20 ground motions.}
\begin{tabular}{|c|c|c|}
\hline
& Number of ground motions & Median roof displacement (mm) \\
\hline
Original structure & 17 (85%) & Unbounded \\
Retrofitted structure & 5 (25%) & 140 mm \\
\hline
\end{tabular}
\end{table}

Table 4

Information about selected and scaled ground motions.

\begin{table}[h]
\centering
\caption{Design and actual response spectra.}
\end{table}
The information about selected and scaled ground motions is summarized in Table 3.

Table 4 summarizes the number of collapses for the original and retrofitted structures subjected to the 20 ground motions, and the median roof displacements for each case. Fig. 13 shows the median floor displacement.

For the 20 ground motions, the median roof displacement of the original structure significantly exceeded the target displacement ($u_{\text{ref}} = 144$ mm) whereas that of the retrofitted structure was maintained ($u_{\text{ref}} = 140$ mm) below the target displacement (Fig. 13).

For the original structure, seventeen of the twenty ground motions cause collapse whereas for the retrofitted structure, only five of the twenty ground motions causes structural collapse [$5/20 = 0.25$ (25%), which is less than 50% (median)]. In this study, a structure is considered to collapse if its floor displacement increases without bound during the analysis. Note that although
the median response spectrum of selected 20 ground motions matches the design response spectrum, some individual ground motions have response spectrum much larger than the design response spectrum, which could cause structural collapse.

Fig. 14a shows hysteretic curves for the original and retrofitted structures subjected to a ground motion CHY058 of the 1999 Chi-Chi earthquake. As shown in Fig. 14a, the displacement of the original structure increases without bound whereas the retrofitted structure exhibits relatively small displacement and does not experience global dynamic instability. If a structure is in a state of global dynamic instability, displacements increase without bound in response to any slight increase in ground motion intensity. For comparison purpose, static pushover curve was included in Fig. 14a.

Fig. 14b shows the roof displacement history under the same motion (CHY058) for both the original and retrofitted structures. The displacement of the original structure is observed to become unbounded at 57 s. Contrastingly, the maximum roof displacement of the retrofitted structure is 137 mm. The structure always oscillates about its original equilibrium position, indicating that the permeant displacement after ground motion ends is negligible in the retrofitted structure.

![Fig. 15](image1.png)

Fig. 15. IDA curves of original and retrofitted structures.

![Fig. 16](image2.png)

Fig. 16. Maximum roof displacements, energy absorption, maximum shear forces of original and retrofitted structures for the twenty ground motions.
Fig. A.1. Overall procedure for the proposed seismic retrofit method.
Fig. 14c shows the cumulative energies dissipated by the original and retrofitted structures under the same ground motion. The friction damper is observed to dissipate as much as 62% (=133/213) of the total energy dissipated by the retrofitted structure. Fig. 14d shows the hysteretic curves of braces with friction dampers located in different stories of the retrofitted structure during the same ground motion. It is observed that all friction dampers dissipate the large amount of energy.

In the present study, the collapse intensity of the original and retrofitted structures is also estimated by conducting incremental dynamic analyses using the same 20 ground motions. Herein, collapse intensity is defined as the pseudo spectral acceleration (A) measured when a structure reaches a state of global dynamic instability. Solid circles in Fig. 15 denote collapse intensities.

The median collapse intensity of the original structure is 0.13 g whereas that of the retrofitted structure is 0.22 g; therefore, the median collapse intensity of the retrofitted structure is 1.77 times that of the original structure, solely owing to the friction damping system added in seismic retrofit.

Fig. 16 shows the maximum roof displacements, dissipated energy, and maximum shear forces of original and retrofitted structures for the twenty ground motions. The median value of maximum roof displacements for the original and retrofitted structures measured at an instance prior to global dynamic instability are 133 mm and 158 mm, respectively. At this instance, the median values of maximum shear forces and cumulative dissipated energy of the original structure are 345 kN and 108 kNm, respectively. For the retrofitted structure, these values become as large as 113% (=351 kN) and 151% (=171 kNm) of those of the original structure, respectively.

7. Conclusions

This study proposed a method for designing a friction damping system consisting of a braced frame and friction dampers to retrofit low- to mid-rise existing old RC buildings mainly governed by the first mode. The proposed design method is used to determine two design values: the sectional area of the braces and the activating forces for the friction dampers. The objective of the design is to limit the maximum displacement of a structure to remain within a displacement corresponding to a drop in strength in the structure. The maximum displacement of a structure is calculated using equivalent SDF systems and an inelastic displacement ratio. To verify the design method, we considered a six-story RC building originally designed considering only gravity loads. It was assumed that the structure was located in a seismically active area (\( \delta_{DN} = 0.49 \) g). Nonlinear response history analyses were conducted to compare the seismic responses of the original and retrofitted structures. Twenty ground motions were used as input, modified to match the design response spectrum specified in ASCE 7–10 [7]. The median roof displacement of the original structure was unbound when subjected to the 20 ground motions selected and scaled in this study, whereas that of the retrofitted structure was 140 mm. This arose solely from the contributions of the friction damping system in the retrofitted structure. The friction damper dissipated as much as 62% of the total energy dissipated by the retrofitted structure. The collapse intensity of the retrofitted structure was 1.77 times than that of the original structure.

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Appendix A

Fig. A.1 illustrates the entire proposed procedure to design a seismic retrofit based on a friction damping system.

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