Seismic Design of Friction-Damped Precast Concrete Frame Structures

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Abstract

This paper is on the seismic design of unbonded post-tensioned precast concrete frame structures that use friction dampers for supplemental energy dissipation. A procedure is described to determine the friction damper slip force and the area of post-tensioning steel needed in a frame to satisfy prescribed lateral strength and energy dissipation requirements. The proposed design procedure assumes that the lateral strength requirements for the frame have been obtained from a linear elastic analysis of the structure under equivalent lateral forces. Nonlinear reversed cyclic analyses of friction-damped precast concrete beam-column subassemblies and multi-story frame structures under lateral loads are conducted to evaluate the design procedure and to identify areas where improvement may be needed. The analytical results show that friction-damped precast concrete frames can achieve significant energy dissipation levels while maintaining a large level of self-centering capability due to the post-tensioning force.

Introduction and Background

Recent experimental and analytical research (Morgen and Kurama 2003, 2004a,b, 2005) has shown that unbonded post-tensioned precast concrete frame structures with friction dampers have desirable seismic characteristics such as a self-centering capability (i.e., ability of the structure to return towards its original undisplaced position upon unloading from a nonlinear lateral displacement) and an ability to undergo large nonlinear lateral displacements with little damage. The post-tensioning steel is placed inside the beams at the floor and roof levels to provide
lateral strength and stiffness to the frame as well as the elastic restoring force needed to develop the self-centering capability.

As shown in Fig. 1, the friction dampers are placed locally at selected beam ends. In the event of a large earthquake, gap opening occurs at the joints between the precast concrete beam and column members. The friction dampers utilize these gap opening displacements to achieve slip displacements at the friction interfaces and provide supplemental energy dissipation to the frame (Morgen and Kurama 2004a). The friction dampers also provide a part of the lateral strength and stiffness of the structure. Some of the benefits of using friction as a supplemental energy dissipation mechanism for seismic protection include: (1) repeatable and reliable hysteretic behavior that is relatively independent of velocity and displacement amplitude; (2) close-to-rectangular force versus displacement characteristics providing a large amount of energy dissipation per cycle; and (3) large damper initial stiffness allowing slip to occur early in the response, and thus, providing energy dissipation beginning at small lateral displacements of the frame.

Prior research on the use of supplemental energy dissipation in precast concrete construction is limited and has primarily focused on shear wall and braced frame structures. In comparison, the dampers described in this paper are used locally at the beam-to-column joints without the need for bracing members in the structure. The application of similar “local-damper” configurations in cast-in-place reinforced concrete frames, steel frames, and steel-concrete coupled walls has been recently investigated (e.g., Way 1996; Martinez-Rueda 1998; Mulas et al. 2000; Christopoulos et al. 2002; Ricles et al. 2002; Rojas et al. 2002; Shen and Kurama 2002).

Large-scale experiments of friction-damped precast concrete beam-column subassemblies under reversed cyclic loading (Morgen and Kurama 2004a) and full-scale experiments of isolated friction dampers under dynamic loading (Morgen and Kurama 2004b) show that the dampers can provide a large amount of supplemental energy dissipation to a frame, while the self-centering capability of the structure is preserved. Based on these recent experimental investigations, this paper describes an analytical study on the seismic design of friction-damped precast concrete frame structures. The focus of the paper is the determination of the damper slip forces and the area of the post-tensioning steel to achieve prescribed design lateral strength and inelastic energy dissipation requirements for a frame. It is assumed that the design beam end moment demands have been determined from a linear-elastic analysis of the structure under equivalent lateral forces (e.g., as described in ICC 2003).

The proposed design approach is developed below in two steps: (1) design of friction-damped beam-to-column joints; and (2) design of multi-story frames. Nonlinear reversed cyclic analyses of friction-damped beam-column subassemblies and multi-story frame structures under lateral loads are conducted to critically evaluate the design procedures and identify areas where improvement may be needed.
Design of Friction-Damped Beam-to-Column Joints

This section describes the development and verification of a design procedure to satisfy prescribed flexural strength and energy dissipation requirements for a friction-damped precast concrete beam-to-column joint.

**Damper Slip Force and PT Steel Area.**

In order to determine the damper slip force and post-tensioning steel area needed, the nominal moment strength of a friction-damped precast concrete beam-to-column joint is divided into two components, $M_{bs}$ and $M_{bp}$, representing the contributions of the friction dampers and the beam post-tensioning steel reinforcement, respectively, to satisfy the total design beam end moment demand, $M_{bd}$ [Equation (1)].

Using the equilibrium of the forces in Fig. 2 at the beam end, Equation (1) can be written as in Equations (2)-(4), where, $F_{ds}$ is the slip force of the friction dampers (assumed to be equal for the two dampers and acting in a direction parallel to the beam) given by Equation (5), $n$ is the number of friction slip interfaces in each damper; $\mu$ is the coefficient of friction for the damper slip interfaces (see Morgen and Kurama 2004b), $F_{dn}$ is the damper normal force (i.e., the damper “clamping” force acting normal to the friction slip interfaces), $h_b$ is the beam depth, $h_d$ is the distance from the damper normal bolt (see Morgen and Kurama 2004a) to the extreme concrete fiber of the beam, $C_c$ is the concrete compressive stress resultant given by Equation (6), $N_b$ is the axial force in the beam from the post-tensioning force, $A_p$ is the total area of the post-tensioning steel, $f_{pi}$ is the design initial stress in the post-tensioning steel after losses, $a$ is the depth of the assumed uniform concrete compression stress block given by Equation (7), $c$ is the neutral axis depth, $f'_c$ is the design unconfined concrete compressive strength, and $b_b$ is the beam width.

![Figure 2. Equilibrium of forces at a beam-to-column joint.](image-url)
In order to determine the required damper slip force and post-tensioning steel area, a new parameter, referred to as the design damper moment ratio, is defined as \( \beta_d = \frac{M_{bs}}{M_{bd}} \). Substituting \( \beta_d \) into Equation (1) yields Equations (8) and (9). Then, by substituting Equations (8) and (9) into Equations (2) and (3), respectively, Equations (10) and (11) can be derived for the required damper slip force, \( F_{ds} \), and post-tensioning steel area, \( A_p \), for a prescribed design beam end moment, \( M_{bd} \) and selected value of the design damper moment ratio, \( \beta_d \). Note that the determination of the post-tensioning steel area in Equation (11) requires an iterative process using the rectangular stress block depth \( \alpha \) in Equation (7).

Relative Energy Dissipation Ratio. Recommendations on the energy dissipation requirements for moment frame structures can be found in the ACI T1.1-01 document Acceptance Criteria for Moment Frames Based on Structural Testing (ACI 2001). According to ACI T1.1-01, the relative energy dissipation ratio, \( \beta \), is defined for a beam moment versus rotation cycle as the ratio of the area \( D_h \) enclosed by the hysteresis loop for that cycle [e.g., shaded area enclosed by the hysteresis curve in Fig. 3(a)] to the area \( A_h \) of the circumscribing parallelogram [dashed lines in Fig. 3(a)]. The relative energy dissipation ratio, \( \beta \), is a measure of the amount of viscous damping in an equivalent linear-elastic system that would result in a similar amount of energy dissipation as the nonlinear system. ACI T1.1-01 recommends that if \( \beta \) is smaller than 0.125, there may be inadequate damping for the frame as a whole, and the oscillations of the frame may continue for a considerable time after an earthquake, possibly producing low-cycle fatigue effects and excessive displacements.

Fig. 3(a) depicts the idealized hysteretic moment versus rotation behavior of a friction-damped beam-to-column joint satisfying the design beam end moment, \( M_{bd} \). The basis for this idealized behavior can be seen from the experimentally obtained results presented in Morgen and Kurama (2004a). Figs. 3(b) and 3(c) show the idealized contributions from the friction dampers, \( M_{bs} \), and the post-tensioning steel, \( M_{bp} \), respectively, to the total moment resistance in Fig. 3(a). It is assumed that the moment contribution from the friction dampers possesses an elastic-perfectly-plastic hysteretic behavior and that the entire energy dissipation in Fig. 3(a) is provided by the friction dampers [as experimentally validated in Morgen and Kurama (2004a,b)]. Thus, the moment contribution from the post-tensioning steel is represented using an elastic though nonlinear (i.e., nonlinear-elastic) moment-rotation behavior.

### Equations

1. \[ M_{bs} = \frac{\beta_d}{1 + \beta_d} M_{bd} \]  
2. \[ M_{bp} = \frac{M_{bd}}{1 + \beta_d} \]  
3. \[ F_{ds} = \frac{\beta_d M_{bd}}{(1 + \beta_d)(h_b + 2h_d)} \]  
4. \[ A_p = \frac{M_{bd}}{f_{pl} (1 + \beta_d)\left(\frac{h_b - \alpha}{2}\right)} \]
behavior as shown in Fig. 3(c). This nonlinear-elastic component provides self-centering capability to the structure, while allowing softening and period elongation to occur. Note that the nonlinear behavior in Fig. 3 is governed by the opening of a gap at the beam-to-column joint, and thus, occurs with little damage in the structure.

For design purposes, a relationship can be developed between the damper moment ratio \( \beta_d \) used in Equations (8)-(11) and the relative energy dissipation ratio, \( \beta \), defined by ACI T1.1-01. Two types of relative energy dissipation ratios are used in this paper: (1) \( \beta_b \) is the relative energy dissipation ratio at a beam-to-column joint (i.e., at a beam end); and (2) \( \beta \) is the relative energy dissipation ratio for an entire multi-story frame. Ignoring the “post-yield” stiffness of the moment-rotation relationship in Fig. 3(a) (i.e., assuming \( \alpha = 0 \)), the relative energy dissipation ratio, \( \beta_b \) at a beam end can be written as in Equation (12). Then, combining Equations (8) and (12), the design damper moment ratio, \( \beta_d \) can be related to \( \beta_b \) using Equation (13).

\[
\beta_d = \frac{\beta_b}{1 - \beta_b} \tag{13}
\]

The proposed design procedure requires that a value for the beam end relative energy dissipation ratio \( \beta_b \geq 0.125 \) is selected as required by ACI T1.1-01. Then, the damper slip force and post-tensioning steel area for the joint can be determined by substituting Equation (13) into Equations (10) and (11).

**Analytical Verification of Beam Design Equations.** The beam design procedure described above is verified based on nonlinear reversed cyclic analyses of a series of twelve cruciform-shaped (extending between mid-story heights of columns and mid-span lengths of beams) friction-damped precast concrete beam-column subassemblies (see Fig. 4). It is assumed that two friction dampers are used at each beam-to-column joint.

The main parameters varied in the study are the design beam end relative energy dissipation ratio, \( \beta_b \) and the beam end moment, \( M_{bd} \). As shown in Table 1, four relative energy dissipation ratios (\( \beta_b = 1/8, 1/4, 5/16, \) and \( 5/12 \)) and three beam end moment values (\( M_{bd} = 500, 1000, \) and \( 1500 \) kip-ft \( [678, 1358, \) and \( 2034 \) kN-m]) are used. The increasing \( M_{bd} \) values are assumed to correspond to increasing beam depths of \( h_b = 24, 32, \) and \( 40 \) in. \( (609, 813, \) and \( 1016 \) mm) in a multi-story frame.

The required damper slip force, \( F_{ds} \) and post-tensioning steel area, \( A_p \) for the twelve subassemblies, as determined using Equations (10), (11), (13), and (7), are given in Table 1. It is assumed that the critical sections are located at the beam ends. Additional structural properties assumed for the design of the subassemblies are: \( f_{pi} = 0.65 f_{pui} \) where \( f_{pui} = 270 \) ksi \( (1862 \) MPa) is the design ultimate strength of the PT steel; \( h_d = 11.5 \) in. \( (292 \) mm); \( f'_{c} = 6 \) ksi \( (41.4 \) MPa); and \( b_b = 24 \) in. \( (610 \) mm).
Table 1. Required subassembly $F_{ds}$ and $A_p$. Note: 1 kip = 4.448 kN; 1 in = 25.4 mm.

<table>
<thead>
<tr>
<th>$M_{bd}$ (kip-ft)</th>
<th>$h_b$ (in)</th>
<th>$F_{ds}$ (kips)</th>
<th>$A_p$ (in$^2$)</th>
<th>$F_{ds}$ (kips)</th>
<th>$A_p$ (in$^2$)</th>
<th>$F_{ds}$ (kips)</th>
<th>$A_p$ (in$^2$)</th>
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<th>$A_p$ (in$^2$)</th>
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<td>51</td>
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<td>64</td>
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<td>2.86</td>
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<tr>
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<td>30</td>
<td>2.51</td>
<td>37</td>
<td>2.27</td>
<td>50</td>
<td>1.87</td>
</tr>
</tbody>
</table>

The nonlinear reversed cyclic lateral load analyses of the subassemblies were conducted using the DRAIN-2DX (Prakash et al. 1993) model in Fig. 4. This analytical model includes fiber beam-column elements to represent the precast concrete beam and column members, truss elements to represent the unbonded post-tensioning steel, and elastic-perfectly-elastic truss elements to represent the friction dampers. More detailed information on the analytical model can be found in Morgen and Kurama (2004a). It is assumed that the subassemblies are properly designed such that all premature failure modes (e.g., shear failure) are prevented.

As examples of representative behavior, Figs. 5(a) through 5(d) plot the beam end moment, $M_b$, versus beam chord rotation, $\theta_b$ behaviors corresponding to $\beta_b = 1/8$, 1/4, 5/16, and 5/12, respectively, for the subassembly with $h_b = 32$ in. (813 mm) and $M_{bd} = 1000$ kip-ft (1358 kN-m). The four different subassembly designs in Fig. 5 have similar beam end moment strengths satisfying the design moment, $M_{bd} = 1000$ kip-ft (1358 kN-m), but the energy dissipation of the structures are different.

The relative energy dissipation ratios, $\beta_b$, of the twelve beam-column subassemblies, as determined from the analysis results according to the ACI T1.1-01 definition (ACI 2001), are listed in Table 2. Each calculated $\beta_b$ value in Table 2 represents the average value from all of the rotation cycles conducted during the analysis of each structure (i.e., $\theta_b = \pm 0.5\%$, $\pm 1.0\%$, $\pm 1.5\%$, and $\pm 2.0\%$). It is observed that the calculated $\beta_b$ values are close to the prescribed values used in design. Thus, it is concluded that the proposed design formulation based on Equations (10), (11), (13), and (7) results in a predictable beam end moment versus beam chord rotation hysteretic behavior, including the moment strength and energy dissipation characteristics.
Figs. 6(a) and 6(b) plot the calculated damper slip forces, $F_{ds}$ and post-tensioning steel areas, $A_p$, from Table 1 against the calculated beam relative energy dissipation ratio, $\beta_b$ values from Table 2. As expected, the relationships between $F_{ds}$ and $\beta_b$ and between $A_p$ and $\beta_b$ are close to linear, with $F_{ds}$ increasing and $A_p$ decreasing as $\beta_b$ is increased.

**Design of Friction-Damped Frames**

In an attempt to extrapolate the above beam-to-column joint design methodology to multi-story frame structures, this section describes a design approach in which the number of the friction dampers in the frame is flexible and is a choice in the design procedure.

**Development of Frame Design Equations.** As an example, Fig. 7 depicts two floor levels in a multi-story friction-damped precast concrete moment frame. The lower floor (level $i$) has friction dampers at four beam ends whereas the upper floor (level $i+1$) has friction dampers at every beam end.

One method would be to design each beam end in the structure using Equations (10), (11), (13), and (7). The difficulty with this approach is that different amounts of post-tensioning steel would result for each beam end, requiring the floor post-tensioning area to be varied from span to span and resulting in an impractical design. Therefore, the goal of the friction-damped frame design formulation below is to provide a constant $A_p$ over each floor level and to provide the required damper slip force at the friction-damped beam ends to satisfy the prescribed lateral strength and energy dissipation requirements.

To develop design equations for use in multi-story frame structures, the following four new variables are introduced: (1) $n_{jt}$ is the total number of beam ends at a floor level (e.g., $n_{jt} = 8$ for levels $i$ and $i+1$ in Fig. 7); (2) $n_{js}$ is the total number of friction-damped beam ends at the floor level (e.g., $n_{js} = 4$ for level $i$ and $n_{js} = 8$ for level $i+1$ in Fig. 7); (3) $M_s$ is the damper contribution to the beam end moment resistance, assumed to be constant at all of the friction-damped beam ends at the floor level; and (4) $M_p$ is the post-tensioning contribution to the beam end moment resistance, assumed to be constant at all of the beam ends at the floor level. It is assumed that two dampers are used at each friction-damped beam end.

As shown in Equations (14) and (15), the sum of the prescribed design beam end moments at a floor level, $\Sigma M_{bd}$ is assumed to be equal to the sum of the damper contribution, $M_s$ times the number of friction-damped beam ends, $n_{js}$ and the post-
tensioning contribution, $M_p$ times the total number of beam ends, $n_{jt}$ (i.e., $n_{js} M_s + n_{jt} M_p$). Then, Equations (16)-(19) can be developed to determine the required amounts of damper slip force $F_{ds}$ and post-tensioning steel area $A_p$ for floor levels with different number of friction-damped beam ends. Note that $F_{ds}$ and $A_p$ are assumed to remain constant within the floor being designed. This approach allows friction dampers to be used selectively in a floor, while keeping the post-tensioning area constant across the spans. It is assumed that the location of the friction-damped beam ends at a floor level does not have an effect on the behavior of the frame.

It can be seen that when $n_{js} = 0$ (i.e., floor level with no friction dampers), then, $M_p = \Sigma M_{bd}/n_{jt}$ from Equation (17), indicating, as expected, that all of the beam end moment resistance at the floor level is provided by the post-tensioning force.

\[
\begin{align*}
n_{js} M_s + n_{jt} M_p &= \Sigma M_{bd} \\
n_{js} \beta_d M_p + n_{jt} M_p &= \Sigma M_{bd} \\
M_s &= \frac{\Sigma M_{bd}}{n_{jt} + \beta_d n_{js}} \beta_d \\
M_p &= \frac{\Sigma M_{bd}}{n_{jt} + \beta_d n_{js}} \\
F_{ds} &= \frac{\Sigma M_{bd}}{(n_{jt} + \beta_d n_{js})(h_b + 2h_d)} \beta_d \\
A_p &= \frac{\Sigma M_{bd}}{f_{pi}(n_{jt} + \beta_d n_{js})\left(\frac{h_b - a}{2}\right)}
\end{align*}
\]

**Analytical Verification of Frame Design Equations**

Based on the multi-story frame design procedure described above, a series of nonlinear reversed cyclic lateral load pushover analyses were conducted using a six story frame structure with four different damper configurations as shown in Fig. 8. More details on the general properties of the frame (e.g., span lengths, story heights, column dimensions, etc.) can be found in Morgen and Kurama (2004a, 2005). Fig. 8(a) depicts the baseline configuration with friction dampers located at every beam end, except at the roof level (resulting in a total number of friction-damped beam ends in the frame, $n_d = 40$). This baseline frame targeted the ACI T1.1-01 minimum of $\beta = 1/8$ and was designed using a constant design beam end relative energy dissipation ratio of $\beta_b = 1/8$ at every damper location in the

\[
\begin{align*}
\beta &= 1/8; \beta_b = 1/8; n_d = 40 \\
\beta &= 1/8; \beta_b = 1/4; n_d = 20 \\
\beta &= 1/8; \beta_b = 5/16; n_d = 16 \\
\beta &= 1/8; \beta_b = 5/12; n_d = 12
\end{align*}
\]

**Figure 8.** Friction-damped frame damper layouts – (a) $\beta_b = 1/8$; (b) $\beta_b = 1/4$; (c) $\beta_b = 5/16$; (d) $\beta_b = 5/12$. 

\[\text{Figure 8. Friction-damped frame damper layouts – (a) $\beta_b = 1/8$; (b) $\beta_b = 1/4$; (c) $\beta_b = 5/16$; (d) $\beta_b = 5/12$.}\]
structure, with \( M_{bd} = 1500 \text{ kip-ft (2034 kN-m)} \) and \( h_b = 40 \text{ in. (1016 mm)} \) at floor levels 1 through 3; \( M_{bd} = 1000 \text{ kip-ft (1358 kN-m)} \) and \( h_b = 32 \text{ in. (813 mm)} \) at levels 4 and 5; and \( M_{bd} = 500 \text{ kip-ft (678 kN-m)} \) and \( h_b = 24 \text{ in. (609 mm)} \) at level 6 (i.e., roof level). Note that these design beam end moment demands and beam sizes are the same as the used in the beam-column subassembly investigation described previously. The \( M_{bd} \) values were assumed to be the same at every beam end at a floor level.

Additional frame configurations were considered with different number of friction-damped beam ends (i.e., \( n_d = 20, 16, \) and \( 12 \)) as shown in Figs. 8(b)-(d). The design beam end relative energy dissipation ratios (prescribed only at the friction-damped beam ends) were scaled proportionally based on the number of dampers used such that \( \beta_b = 1/4, 5/16, \) and \( 5/12 \) were used for the frames with \( n_d = 20, 16, \) and \( 12 \), respectively. The analyses investigated, for example, if decreasing the total number of dampers by a factor of two and increasing the beam end relative energy dissipation ratio, \( \beta_b \) by a factor of two results in roughly the same relative energy dissipation ratio for the entire frame. Table 3 lists the calculated required damper slip forces and post-tensioning steel areas from Equations (18) and (19) for the varied frame parameter combinations and floor levels.

### Table 3. Required frame \( F_{ds} \) and \( A_p \). Note: 1 kip = 4.448 kN; 1 in = 25.4 mm.

<table>
<thead>
<tr>
<th>Level</th>
<th>( \beta = 1/8 )</th>
<th>( \beta = 5/16 )</th>
<th>( \beta = 5/12 )</th>
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<tr>
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<td>( \beta_b = 1/4 )</td>
<td>( \beta_b = 5/16 )</td>
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<td></td>
<td>( n_d = 40 )</td>
<td>( n_d = 20 )</td>
<td>( n_d = 16 )</td>
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<td>( F_{ds} ) (kips)</td>
<td>( A_p ) (in(^2))</td>
<td>( F_{ds} ) (kips)</td>
<td>( A_p ) (in(^2))</td>
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<td>5</td>
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Analytical models for the frames were constructed by combining the subassembly models in Fig. 4 at the beam mid-span and column mid-height locations. The columns were allowed to yield at the bases but not over the height of the structures. The resulting base shear force, \( V \) versus roof drift, \( \Delta \) relationships for the four friction-damped frame configurations are plotted in Fig. 9. It can be observed that all four frame layouts with different post-tensioning steel areas, damper slip forces, and locations have nearly identical base shear resistances and overall shape of the hysteresis loops.

The resulting frame relative energy dissipation ratios, \( \beta \) calculated for the individual roof drift cycles in Fig. 9 (i.e., \( \Delta = \pm 0.5\%, \pm 1.0\%, \pm 1.5\%, \) and \( \pm 2.0\% \)), as well as the average frame \( \beta \) values, are presented in Table 4. The average \( \beta \) values are slightly higher than the target value of \( \beta = 1/8 \). This small difference may be due to the contribution of yielding at the column bases to the overall frame energy dissipation, which is not included in the proposed design formulation.

In addition to the frame investigations targeting the \( ACI T1.1-01 \) minimum of \( \beta = 1/8 \), configurations with larger \( \beta \) values (\( \beta = 5/16 \) and \( 5/12 \)) were also considered.
The total number of friction-damped beam ends was kept constant at \( n_d = 40 \) and the prescribed beam end relative energy dissipation ratios were varied as \( \beta_b = 5/16 \) and 5/12. Similar to the frames in Fig. 8, Equations (18) and (19) were utilized to determine the required \( F_{ds} \) and \( A_p \) values, respectively, for these additional frame configurations, as provided in Table 3.

Based on the analysis results, Fig. 10 shows the base shear versus roof drift behaviors for the three frame configurations with \( n_d = 40 \) and \( \beta_b = 1/8, 5/16, \) and 5/12 [note that Fig. 10(a) is the same as Fig. 9(a)]. The hysteresis loops for \( \beta_b = 5/16 \) and 5/12 show that friction-damped precast concrete frames can be designed to have energy dissipation levels significantly higher than the ACI T1.1-01 minimum, while maintaining the self-centering capability due to the post-tensioning force. The combination of the friction dampers for energy dissipation with the post-tensioning steel for self-centering provides designers with flexibility in achieving the desired hysteretic characteristics for the structure.

The resulting calculated average relative energy dissipation ratios for the frames in Figs. 10(b) and 10(c) are \( \beta = 0.276 \) and 0.343 (Table 4), respectively. These \( \beta \) values are smaller than the target values of \( \beta = 5/16 \) and 5/12, respectively, indicating that the design energy dissipation requirement is not satisfied (contrary to the frames with \( \beta = 1/8 \)). Comparing the results in Table 4, the unconservative discrepancy between the

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**Table 4. Calculated frame energy dissipation ratios.**

<table>
<thead>
<tr>
<th>roof drift, ( \Delta )</th>
<th>( \beta = 1/8 )</th>
<th>( \beta = 5/16 )</th>
<th>( \beta = 5/12 )</th>
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<tr>
<td>( n_d = 40 )</td>
<td>( \beta_b = 1/8 )</td>
<td>( \beta_b = 1/4 )</td>
<td>( \beta_b = 5/16 )</td>
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<tr>
<td>( n_d = 40 )</td>
<td>0.147</td>
<td>0.141</td>
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<td>0.151</td>
<td>0.148</td>
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<tr>
<td>( n_d = 16 )</td>
<td>0.161</td>
<td>0.167</td>
<td>0.164</td>
</tr>
<tr>
<td>( n_d = 12 )</td>
<td>0.171</td>
<td>0.178</td>
<td>0.174</td>
</tr>
<tr>
<td>average</td>
<td>0.156</td>
<td>0.159</td>
<td>0.156</td>
</tr>
</tbody>
</table>

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**Figure 9. Frame base shear-roof drift behaviors – (a) \( n_d = 40, \beta_b = 1/8 \); (b) \( n_d = 20, \beta_b = 1/4 \); (c) \( n_d = 16, \beta_b = 5/16 \); (d) \( n_d = 12, \beta_b = 5/12 \).**

**Figure 10. Frames with \( n_d = 40 \), (a) \( \beta_b = 1/8 \) [same as Fig. 9(a)]; (b) \( \beta_b = 5/16 \); (c) \( \beta_b = 5/12 \).**
target and calculated frame relative energy dissipation ratios increases for increasing values of \( \beta \), with the largest discrepancy occurring for \( \beta = 5/12 \). It is concluded that the effectiveness of the dampers decreases for these large \( \beta \) configurations and improvement in the design equations is needed. Note that a similar trend can also be observed in the beam-column subassembly results in Table 2.

**Summary and Conclusions**

A seismic design approach for friction-damped precast concrete frame structures is presented. The structures use unbonded post-tensioning steel to provide a part of the flexural resistance at the floor and roof levels. The focus of the paper is to determine the damper slip forces and post-tensioning steel areas needed in a frame to satisfy prescribed lateral strength and energy dissipation requirements. For design purposes, the moment resistance at the end of a friction-damped beam is decomposed into two components: (1) resistance due to the friction dampers; and (2) resistance due to the post-tensioning steel.

A series of prototype friction-damped beam-column subassemblies and multi-story frame structures are designed following the proposed procedures. The *ACI T1.1-01* (ACI 2001) document is used to prescribe the amount of energy dissipation in the structures. The parameters studied in the investigation are: (1) the beam depth; (2) the number of friction-damped beam ends in a frame; (3) the prescribed energy dissipation requirements; and (4) the prescribed lateral strength requirements.

Nonlinear reversed cyclic analyses of the prototype structures under lateral loads indicate that both the design strength and energy dissipation requirements can be satisfied. The results also show that friction-damped precast concrete frames can achieve energy dissipation levels significantly higher than the *ACI T1.1-01* minimum while maintaining a high level of self-centering capability due to the post-tensioning force; however, the design approach may need to be improved for these cases. The combination of the friction dampers for energy dissipation with the post-tensioning steel for self-centering provides designers with flexibility in achieving the desired hysteretic characteristics for a structure.

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