DESIGN OF SELF-CENTERING STEEL CONCENTRICALLY-BRACED FRAMES

Richard Sause¹, James M. Ricles², David Roke³, Choung-Yeol Seo⁴, and Kyung-Sik Lee⁵

ABSTRACT

Self-centering concentrically-braced frame (SC-CBF) systems are being developed with the goal of providing adequate nonlinear drift capacity without significant damage or residual drift under the design basis earthquake. Analytical pushover and dynamic analyses were performed on several SC-CBF configurations to evaluate their response to earthquake loading. Each SC-CBF self-centered under earthquake loading. Some loss of post-tensioning occurred in one frame configuration. The dynamic response of the SC-CBF systems, however, was consistent with the intended behavior.

Keywords: Post-tensioned steel braced frame, self-centering

INTRODUCTION

Steel concentrically-braced frame (CBF) systems are stiff and economical earthquake-resistant steel frame systems, which often exhibit limited system ductility capacity. Ductility capacity can be increased through the use of buckling-restrained braces (e.g., Fahnestock et al. 2003); however, the buckling-restrained braced frame system sometimes exhibits significant residual drift after an earthquake. To increase the ductility and reduce the residual drift of CBFs, self-centering concentrically-braced frame (SC-CBF) systems are being developed.

Recent research on self-centering (SC) unbonded post-tensioned (PT) precast wall systems (e.g., Kurama et al. 2002), on frames with inclined and vertically oriented draped PT tendon systems (Pekcan et al. 2000), and on SC steel moment-resisting frames with PT connections (e.g., Ricles et al. 2001, 2002; Garlock et al. 2005; Rojas et al. 2005) suggests that self-centering system concepts can be applied to CBFs. As part of a larger project on self-centering steel frame systems, ongoing research at Lehigh, Princeton, and Purdue Universities is developing concepts, details, and design criteria for SC-CBF systems.

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SYSTEM BEHAVIOR

The SC-CBF system is shown schematically in Fig. 1(a). The system consists of beams, columns, and braces in a conventional arrangement, with column base details that permit the columns to uplift at the foundation (Fig. 1(c)). Gravity loads and post-tensioning (PT) forces (from PT steel arranged along the column lines in the system shown in Fig. 1(a)) resist column uplift and provide a restoring force after uplift. The beams, columns, and braces are intended to remain elastic under the design earthquake, and the column uplift provides a mechanism for controlling the force levels that develop in the frame under earthquake loading.

Idealized SC-CBF behavior under lateral load is shown in Fig. 1. Fig. 1(a) shows the SC-CBF configuration and the loads used to simulate earthquake loading in a pushover analysis. Design dead loads and live loads \(g\) are applied at the columns at each floor level. The lateral load profile \(F_i\) is based on an equivalent lateral force procedure (ICC 2003).

Under low levels of lateral load, the structure deforms elastically as shown in Fig. 1(b). This deformation is similar to that of a conventional CBF. Under higher levels of lateral load, the overturning moment at the base of the frame becomes large enough for the “tension” column to decompress, and uplift of the column occurs, as shown in Fig. 1(c). After column decompression and uplift, the lateral displacement of the frame is dominated by rigid body rotation of the frame about the base of the compression column, although some additional forces and deformations develop in the beams, columns, and braces of the frame. The PT steel elongates from the uplift and rotation of the frame, which leads to an increase in PT force which provides a positive stiffness to the lateral force-lateral drift behavior.

Figure 1. SC-CBF system: (a) schematic of members and loads; (b) elastic response prior to column decompression; (c) rigid-body rotation after column decompression.

DESIGN SUMMARY

A prototype structure used for the studies described later was designed to demonstrate the behavior of the SC-CBF system. The structure is a six-story office building designed for a site in Los Angeles, CA. The floor plan of the building is shown in Fig. 2. Braced frames were located within the interior of the plan. For the initial stages of the study, simple estimates of the gravity loads and seismic masses have been made and are shown in Fig. 2.

This data is based on several simplifying assumptions. The dead load of each floor includes the concrete floor slab, steel floor deck, mechanical equipment, floor and ceiling finishes, cladding weight, and an estimated weight per square foot of structural steel. The seismic weight of each floor is taken as the dead load plus 20 psf for partitions, as per IBC 2003 (ICC 2003). The live load is also given in IBC 2003.
Since four braced frames act along each axis of the structure, each frame was designed to resist one fourth of the lateral loading determined from an equivalent lateral force procedure (ICC 2003). The floor diaphragms of the building were assumed to be rigid.

### Prototype building design loads

<table>
<thead>
<tr>
<th>Floor</th>
<th>Seismic Load (N/m²)</th>
<th>Dead Load (N/m²)</th>
<th>Live Load (N/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>5267</td>
<td>5267</td>
<td>2394</td>
</tr>
<tr>
<td>5</td>
<td>5267</td>
<td>4309</td>
<td>2394</td>
</tr>
<tr>
<td>4</td>
<td>5267</td>
<td>4309</td>
<td>2394</td>
</tr>
<tr>
<td>3</td>
<td>5267</td>
<td>4309</td>
<td>2394</td>
</tr>
<tr>
<td>2</td>
<td>5267</td>
<td>4309</td>
<td>2394</td>
</tr>
<tr>
<td>1</td>
<td>5267</td>
<td>4309</td>
<td>2394</td>
</tr>
</tbody>
</table>

Figure 2. Properties of prototype structure.

**LIMIT STATES AND PERFORMANCE-BASED DESIGN**

The complete behavior of a SC-CBF will include a large number of possible limit states, which are the subject of ongoing research. For these initial studies and the development of the related analytical models, the following primary limit states were considered:

1. Decompression and uplift of the “tension” column at the base of the SC-CBF.
2. Yielding of the PT steel.
3. Significant yielding of the beams, columns, or braces of the SC-CBF.
4. Failure of the beams, columns, or braces of the SC-CBF.

In this paper, an initial performance-based design approach for SC-CBFs is proposed to control the occurrence of these four limit states, as shown schematically in Fig. 3. Two seismic input (ground motion intensity) levels, as defined by FEMA 450 (BSSC 2003), are considered: (1) a design-basis earthquake (DBE) and (2) a maximum considered earthquake (MCE). The performance objectives are immediate occupancy (IO) performance under the DBE and life safety (LS) performance under the MCE. To reach these objectives, the four primary limit states were considered further.

Due to the self-centering nature of the SC-CBF system, decompression and uplift of the SC-CBF columns is not a limit state associated with structural damage. Of course, the SC-CBF system must be designed and detailed so that decompression and uplift of the columns does not produce nonstructural damage. Assuming the system is detailed in this manner, decompression and uplift of the columns is considered to be a limit state that conforms to the IO performance level.

The post-tensioning force in the PT steel is important to the self-centering behavior of SC-CBFs. Therefore, significant yielding of the PT steel may have serious consequences. Some types of PT steel and associated anchorage systems, for example, certain high strength PT bars systems, can yield and provide ductile behavior without immediate failure (Perez et al. 2003). However, other types of PT steel and associated anchorage systems, for example, certain high strength PT strand systems, will begin to fail at the anchorages shortly after yielding (Garlock et al. 2005). In the present study, the SC-CBF system employs high strength PT bars, and, therefore, the primary consequence of PT steel yielding is the subsequent loss of prestress force under cyclic loading. As shown later, this loss of prestress is a form of structural damage that does not compromise the safety of the SC-CBF system, however, it must be repaired after an earthquake (by re-stressing or replacing the PT steel). Therefore,
yielding of the PT steel is a limit state that conforms to the LS performance level, but not to the IO performance level.

Significant yielding of the beams, columns, or braces of the SC-CBF is a limit state associated with initiation of structural damage to SC-CBF system members other than the PT steel. To control the amount of damage that must be repaired after ground motions at an intensity level between the DBE and MCE levels, this limit state should be reached only after yielding of the PT steel, as shown in Fig. 3. Yielding of the PT steel provides a mechanism to control force levels in the SC-CBF, making it possible to control significant yielding of these structural members. The preliminary performance-based design approach proposed herein employs a capacity design method for the beams, columns, and braces of the SC-CBF to keep these members essentially elastic at the force levels that develop at yielding of the PT steel.

Failure (buckling or fracture) of the beams, columns, or braces of the SC-CBF is a limit state that does not conform to the LS performance level, as shown in Fig. 3. In the preliminary performance-based design approach, the force level control provided by yielding of the PT steel, as well as the inherent ductility of these steel members, is used to control this limit state. Fig. 3 summarizes the design objectives of the preliminary performance-based design approach.

![Figure 3. Limit states, performance levels, and ground motion intensity levels.](image)

**PROTOTYPE FRAME DESIGN**

Three different prototype frame designs were considered in this study. Frame A is a typical braced-frame with PT steel added along each column line, as shown in Fig. 4(a). Frame A requires significantly stronger columns than a typical braced frame, because of the post-tensioning forces, and because each member of the frame is designed to remain essentially elastic at force levels corresponding to PT yielding.

Frame A has 12 PT bars at each column line and Frame B12 the same number of bars, 12, located at the center of the frame, as shown in Fig. 4(b). Note that the brace member sizes for this frame are very large because the PT steel force is transferred to the column lines through the upper story brace members. A variation on Frame B12, denoted Frame B12ED, includes energy dissipation (ED) elements at the base of the columns, as shown in Fig. 4(c). The energy dissipation elements reduce the drift of the frame under earthquake loading and reduce the associated column uplift. For this preliminary study, the ED element is modeled as a simple elastic-plastic friction element. The capacity of the ED element was chosen so that the columns of Frame B12ED decompress at approximately the same level of lateral force as the columns of Frame A.
Figure 4. Prototype frames: (a) Frame A; (b) Frame B12; (c) Frame B12ED.

Design Procedure

Member selection for each frame was based on the aforementioned performance-based design (PBD) criteria. To meet these criteria, the frame members must be able to withstand the internal member forces at the level of lateral force at which the PT steel yields. To determine the magnitude of the lateral force at PT yield, a simple static analysis is used to determine the base overturning moment, \( OM_{yld} \), required to yield the PT steel. The equivalent lateral force profile, \( \{ F_{des} \} \), is based on IBC 2003 (ICC 2003). The height of floor \( j \) is denoted \( h_j \). This load profile is then scaled by the factor \( L_{des,yld} \) to induce the base overturning moment at PT yield as follows:

\[
L_{des,yld} \sum_{j=1}^{N} F_{des,j} h_j = OM_{yld}
\]

This load profile, \( L_{des,yld} \{ F_{des} \} \), is then applied to the structure to determine the internal member force design demands. Member sizes are based on these demands. Member capacities were calculated using the Load and Resistance Factor Design (LRFD) code specifications (AISC, 2001).

Table 1. Summary of six-story prototype frames

<table>
<thead>
<tr>
<th>Frame</th>
<th>Total Column Weight (kN)</th>
<th>Total Beam Weight (kN)</th>
<th>Total Brace Weight (kN)</th>
<th>Total Weight (kN)</th>
<th>Total # of PT Bars</th>
<th>( \frac{PT_0}{PT_j} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>155.2</td>
<td>83.6</td>
<td>76.1</td>
<td>314.9</td>
<td>24</td>
<td>20%</td>
</tr>
<tr>
<td>B12</td>
<td>149.5</td>
<td>63.2</td>
<td>81.0</td>
<td>293.6</td>
<td>12</td>
<td>20%</td>
</tr>
<tr>
<td>B12ED</td>
<td>149.5</td>
<td>63.2</td>
<td>87.2</td>
<td>299.8</td>
<td>12</td>
<td>20%</td>
</tr>
</tbody>
</table>

The designs of the frames are summarized in Table 1 in terms of the total weight per frame. Note that Frame A has the lightest braces and the heaviest columns due to the PT steel being located along the column lines rather than bearing directly on the braces.

Pushover analyses were used to evaluate the structural response, and assess the occurrence of the primary limit states. The lateral load profile (\( F_i \)) used in the pushover analyses is based on an equivalent lateral force procedure (ICC 2003). The analyses were performed using the DRAIN-2DX program. Results from the pushover analyses are shown in Fig. 5 as a plot of normalized base shear versus roof drift. The base shear, \( V \), is normalized by the design base shear, \( V_{des} \), which is determined from the equivalent lateral force procedure using a response modification coefficient, \( R \), equal to 8 (ICC 2003).
PUSHOVER RESPONSE

As seen in Fig. 5(a), the first limit state for each frame is tension column decompression, which is reached before the design base shear is reached. Note that the frame stiffness prior to decompression depends on the member sizes and Frame A has greater initial stiffness than Frames B12 and B12ED. The second limit state is PT yielding. The amount and location of PT steel affects the force level at which this limit state is reached, and Frame A yields at a higher lateral force level than Frames B12 and B12ED. The addition of the energy dissipation elements influences the strength of Frame B12, increasing the lateral force at decompression and PT yielding without requiring additional PT steel.

A cyclic pushover was also performed, in which the frame was pushed to 1% roof drift. The structure was then loaded in the opposite direction until zero total load was applied on the structure. This level of drift was selected so that the frame would decompress but the PT steel would not yield, similar to the expected behavior under the DBE. As seen in Fig. 5(b), each frame self-centers in this analysis. Frame B12ED also exhibits a hysteresis loop with energy-dissipation from the ED elements, while the other frames exhibited nonlinear elastic behavior.

![Figure 5](image)

Figure 5. Pushover analysis results: (a) Monotonic pushover past PT yield; (b) Cyclic pushover.

SEISMIC RESPONSE

The nonlinear time history responses of Frames A, B12, and B12ED subjected to a series of ground motions were determined using DRAIN-2DX (Prakash et al. 1993). These ground motions were scaled to the DBE level using the approach by Seo (Seo and Sause 2005a). Rayleigh damping was used with a 2% damping ratio in the first and third modes. The beams, columns, and braces of the frames were modeled as linear elastic to enable the determination of the strength levels required to keep them elastic. The PT steel was modeled using nonlinear truss-bar elements and the gap opening behavior was modeled using contact elements.

The roof drift responses of Frames A, B12, and B12ED are compared in Fig. 6(a). These results and other time history results presented in this paper are responses to the 1994 Northridge, CA earthquake record NR0NRG090, scaled to the DBE level by Seo’s procedure (Seo and Sause 2005a). Comparing the response of Frames A and B12 shows the effect of the amount and location of the PT steel. As shown in Fig. 5(a), Frame A is significantly stiffer and stronger, resulting in significantly less drift. Comparing the response of Frames B12 and B12ED reveals the effect of the ED elements. Initially, the ED elements slightly reduce the amplitude of the roof drift. However, later in the analysis, ED elements significantly reduce the roof drift amplitude.

The column uplift behavior is quantified in terms of the “gap opening” displacement, which is the relative displacement between the column base and the column base support. Greater roof drift can be
expected when a larger gap opening displacement is observed, due to the rigid-body rotation. Similar to the roof drift, then, the gap opening displacement for Frame A is less than that of Frame B12, as seen in Fig. 6(b), which shows results for the left column; the right column gap opening displacement results were similar. A comparison of Frames B12 and B12ED shows a significant amplitude reduction for Frame B12ED similar to that shown by the roof drift response.

![Figure 6. Dynamic response: (a) roof drift; (b) gap opening.](image)

The PT steel force response of the prototype frames is shown in Fig. 7(a). Fig. 7(a) compares the PT steel force in the left column of Frame A with the total PT steel force at the center of Frames B12 and B12ED. As was the case with the gap-opening response, the results for the right column of Frame A are similar to those of the left column. As expected, the response of the PT steel is in phase with the gap-opening displacement. However, the positive peaks in the PT steel force of Frames B12 and B12ED occur twice as frequently as those of Frame A, and the Frame B12 PT steel force response does not exhibit the negative peaks seen in the response of Frame A, due to the PT steel location in Frame B12. More significantly, the PT steel in Frame A and Frame B12 yield at approximately 10 and 22 seconds into the ground motion, respectively. This behavior does not meet the design objectives for the DBE level ground motion. The resulting loss of post-tensioning force is evident in the response. Comparing the results of Frames B12 and B12ED, again the ED elements significantly reduce the response amplitude. The PT steel in Frame B12ED does not yield, and the post-tensioning force is maintained. Therefore, the amplitude reduction caused by the ED element has the effect of helping to maintain the PT steel force at its initial level. This amplitude reduction due to ED is typical of the PT steel force response, as shown in Fig. 7(b).

![Figure 7. Dynamic response: (a) PT steel force; (b) maximum PT steel force values for 12 ground motion records.](image)
The effect of the ED element was evident in each of the response quantities described earlier. Recent research has studied the effect of energy dissipation levels on the drift response of self-centering structural systems (Seo and Sause 2005b), and ongoing research is investigating the energy dissipation levels needed for SC-CBF systems as well as the advantages and disadvantages of different structural damper types for this system.

Fig. 8(a) compares the total base overturning moment (OM) response of Frame A with its first mode OM response. The total OM was calculated from the equivalent static forces at each time step. These forces were determined as the structural mass times the total acceleration at each floor level. The first mode OM is determined from first mode equivalent static forces, which were decomposed from the total equivalent static forces. Note that, as expected, the OM response is effectively capped by the PT yield threshold of the structure. This effect is representative of the fact that the higher modes contribute little to the OM, which can be observed in the comparison of total OM to first mode OM.

While OM response is controlled by the PT yield level, base shear and story shear responses are not so dependent upon the first mode response. This is illustrated in Fig. 8(b), which is a comparison of total base shear response ($V_b$) of Frame A and first mode base shear response. Unlike the OM response, the total base $V_b$ is not closely correlated to the first mode $V_b$ response. Therefore, the higher modes affect the $V_b$ response without significantly affecting the OM response.

Figure 8. Dynamic response: (a) total OM and first mode OM, Frame A; (b) total base shear and first mode base shear, Frame A.

Though story shear may not be correlated with the first mode response, it is strongly correlated to brace force. Each story, with the exception of the sixth story of Frames B12 and B12ED, has a required brace force demand which closely corresponds to the story shear. The sixth story brace force of Frames B12 and B12ED closely corresponds to the PT force, because the PT steel reacts against the
bracing at the roof level of the frame. The brace force design demands are based on lateral forces that induce an overturning moment that yields the PT steel (OM<sub>yield</sub>). Therefore, the design demands are closely correlated with the first mode story shears at PT yielding, but not with the total story shears at PT yielding. Since the total story shears do not closely correlate with the first mode story shears at yielding, the brace force demands are beyond the capacities, as illustrated in Fig. 9, which shows the maximum demand for each dynamic analysis in comparison to the member capacity. These member capacities were selected based on the design demand, which was derived from the story shears at PT yield under a first mode load pattern.

**SUMMARY AND CONCLUSIONS**

The paper describes the expected lateral force-lateral drift behavior of the self-centering concentrically braced frame (SC-CBF) system concept. The primary limit states of the SC-CBF system were identified and briefly described, and a preliminary performance-based design approach was proposed. Pushover response and seismic response analysis results were presented, illustrating the two primary system limit states of decompression and PT steel yielding.

These preliminary response analysis results show that the SC-CBF behaves as expected. In addition, the results show that certain SC-CBF designs can satisfy the performance objectives of the preliminary design approach. In particular, Frame A and Frame B12 did not always satisfy the performance objectives with respect to PT yielding, while Frame B12ED did satisfy those objectives. It is noted that the preliminary design approach considered a small number of limit states, and other limit states will be incorporated into the performance-based design approach as ongoing research on the SC-CBF system is completed. The seismic response analyses also show that energy dissipation elements have an important effect on the system response. In-depth research on effective energy dissipation systems for SC-CBFs is currently in progress.

Ongoing work is studying the seismic response of SC-CBFs under a variety of seismic input levels, including both DBE and MCE levels, to more rigorously characterize the performance of SC-CBFs. In addition, the ongoing research on SC-CBFs will include design studies, numerous seismic response analyses, the development of performance-based design procedures and criteria, and large-scale experiments and simulations of seismic response in the laboratory. However, the preliminary results presented in this paper suggest that with further research, the overall goals of increasing the ductility and reducing the residual drift of concentrically braced frames can be reached, and that the specific performance objectives for SC-CBFs of immediate occupancy performance under the DBE and life safety performance under the MCE can be achieved.

![Figure 10. Frame configurations: (a) Frame A, (b) Frame B, (c) Frame C.](image)

The analysis results presented in this paper indicate that the present design method does not achieve the performance-based design objectives. The dynamic analysis results clearly demonstrate the effects of higher mode response in the frames, which must be included in the design process. Each frame
considered in this study is designed to limit the first mode dynamic response of the structure. The control mechanism for Frame A and Frame B12 is the yielding of the PT steel. Frame B12ED also incorporates an ED element, which resists only base overturning moment, a first mode response quantity.

Additional research is ongoing to develop frame configurations which incorporate ED elements throughout the height of the structure. These ED elements are intended to reduce the effects of higher modes in the structural response. One such frame configuration is Frame C, shown in Fig. 10(c). In this configuration, there are two smaller frames in one bay, with a series of ED elements positioned at each floor level. As the interior columns of the two frames displace vertically relative to one another, energy will be dissipated in the ED elements, reducing the higher mode response. Future work will compare the dynamic response of Frame C with the frames presented in this paper.

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