RESPONSE OF SELF-CENTERING STEEL MOMENT RESISTING FRAMES WITH POST-TENSIONED COLUMN BASES UNDER SEISMIC LOADING

Hoseok Chi¹ and Judy Liu²

ABSTRACT

The column bases at ground level in self-centering steel moment resisting frames (SC-MRFs) may suffer damage by plastic hinging under the Design Basis Earthquakes (DBE). The formation of plastic hinges at column bases may hinder the self-centering behavior of the SC-MRF. In this research, in order to eliminate damage at column bases and enhance the self-centering capability (i.e., negligible residual drift) of the SC-MRF, a post-tensioned column base connection is proposed. The feasibility of using post-tensioned column base connections in the SC-MRFs was investigated for a 6-story, 4-bay office building in the Los Angeles (LA) region by nonlinear static and dynamic analysis. Based on the analysis results, it was found that if the shear resistance is provided properly, use of the post-tensioned column base connection in the SC-MRFs results in self-centering behavior at column bases and elimination of damage in columns under the DBE.

Keywords: Post-tensioned, column base, self-centering, steel, moment resisting frame

INTRODUCTION

Self-centering steel moment resisting frame systems using post-tensioned (PT) beam-to-column connections have been developed by Ricles et al. (2001) and Garlock (2002). The SC-MRFs can show negligible residual drift after an earthquake compared to traditional steel moment resisting frames. The beams and columns in SC-MRFs are connected by post-tensioning and energy dissipation (ED) elements, and the frame itself has self-centering capability. The experimental studies by Ricles et al. (2002) and Garlock et al. (2005) on the PT beam-column connections proved that the connection provides self-centering behavior and some energy dissipation provided that local damage such as flange buckling, web buckling, and fracture are prevented. The analytical studies for SC-MRF systems by Garlock (2002) showed that the SC-MRF system has an initial lateral stiffness similar to that of the traditional welded steel moment resisting frame and self-centering capability at beam-to-column connections. However, significant damage by plastic hinges is anticipated at grade in the columns in the SC-MRF for a Design Basis Earthquake (DBE). This inelastic deformation is associated with permanent drift after an earthquake and may not be easily repairable. In this study, a PT column base connection for self-centering is introduced, and analysis results of the SC-MRF with PT column base connections for strong ground motions are presented.

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BEHAVIOR OF POST-TENSIONED COLUMN BASE CONNECTION

The main function of a PT column base is to remove the plastic hinge at the column base and to achieve softening instead by gap opening at grade. The PT column base consists of high strength post-tensioned bars, energy dissipating plates and shear resistance keeper elements, as shown in the schematic in Fig. 1. The post-tensioned bars, vertically installed, run from the mid-height of the 1st story columns to close to the bottom of the basement columns. The PT bars are anchored at staggered anchorage plates to distribute the large forces which are developed by the maximum credible earthquake (MCE). During an earthquake, gap opening and closing occurs between the grade beam and 1st story column interface. ED plates, which are attached to the column flange and a beveled keeper plate, dissipate energy by yielding of the reduced section as the gap opens and closes and also increase moment capacity at column bases.

The moment-relative rotation of the PT column base connection is compared to that of a conventional column base in Fig.2. A W14x398 was the original continuous column used at grade level in the prototype building, which has one basement level. A deeper W36 is proposed for use with the PT column base. Md is the decompression moment, at which gap opening occurs. In Fig.2, Md is 20 % of My, the yield moment for a W36x328. The PT column base shows self-centering behavior and does not reach plastic moment capacity of the column.

The moment resistance of the PT column base is determined by the following factors; (1) initial level of the post-tensioning, (2) initial gravity axial force from dead and live loads, (3) strength and stiffness of energy dissipating plates, (4) axial stiffness of the post-tensioned bars. The initial post-tensioning level at column bases should be determined so that the PT bars remain elastic for the DBE and MCE. Also, the ED plates are designed to ensure self-centering at the base as well as in the frame. The shear demand at column bases is typically greater than the friction force developed between one column flange and the grade beam flange. Therefore, an additional shear resistance element, which allows gap opening, is required at column bases. 3-degree beveled keeper plates are proposed for additional shear resistance. An example of the shear demand versus friction resistance at an interior column base is shown in Fig. 3. The friction coefficient used is 0.3.

![Figure 1. Schematic of PT column base](image1)

![Figure 2. Column base moment-rotation](image2)
RESPONSE OF SC-MRFS WITH PT COLUMN BASE CONNECTIONS

Design and model of SC-MRF

A 6-story, 4-bay office building on stiff soil in Los Angeles (LA) region was designed following the performance based design procedure by Garlock (2002). Because of the relatively low decompression moment and relative flexibility due to gap opening of the PT column bases, the perimeter SC-MRF may be too flexible unless the column size increases or the PT column bases are more comparable to fully restrained connections. To overcome the relatively low stiffness issue of the PT column base, deep columns (for example, W36 shapes) were used for interior columns. Deep columns have high moment of inertia, and they can provide a relatively long moment arm for the PT bar forces. For exterior columns, W14 shapes, which have thick flanges, were used because of the high compressive axial forces by overturning moment and gap opening. It was assumed that axial force is applied over the entire one flange of the column after gap opening. In this model, the ED angles introduced by Garlock et al. (2005) were used for the beam-to-column connections, and ED plates were used for the column bases. The schematic view of post-tensioning layout is given in Fig. 4. The member sizes and initial PT forces used in the analyses are given in Table 1. The interior column size was selected based on the design story drift limit of 0.02 (ASCE 7-05) and strong column-weak beam concept. To account

![Figure 3. Shear demand versus friction force (for Loma Prieta MCE)](image)

![Figure 4. Schematic view of post-tensioning layout (Note: Only 4-bay SC-MRF is shown.)](image)
for observed yielding in the nonlinear dynamic analyses at the 1st story columns despite satisfying strong column-weak beam requirements, a factor of 1.6 was used in the Eq.1.

\[ \sum M_c \geq 1.6 \sum M_b \]  
\[ \text{(1)} \]

where, \( \Sigma M_c \) is the sum of the column moment capacity at the beam-column joint, and \( \Sigma M_b \) is the sum of the beam moment demand at the self-centering beam-column joint defined by Garlock (2002).

The grade beam size needs to be large enough to accommodate gap opening about the column weak axis. The size used is listed in the Table 1. The level of the post-tensioning at the column bases was determined so that the yielding of the PT bars does not occur for the DBE and MCE. The DRAIN-2DX program was used for the nonlinear analyses. The modeling method for gap opening at column bases was the same as that of Garlock’s beam-to-column connection (2002). Grade beams and columns were modeled explicitly and P-delta effects due to gravity frames were considered by use of leaning columns. Panel zone deformation for each connection was accounted for in the model. Energy dissipating elements at beam-to-column connections and column bases were assumed to have stable loops. It was assumed that the local buckling does not occur in the model. The total seismic weight (W) of the building was 95000KN and half of the mass was lumped at each story. The first mode period was 1.34 seconds. A992 steels (min. yield strength 344.8MPa) were used for the beams and columns.

Table 1. Member sizes and initial PT forces

<table>
<thead>
<tr>
<th>Story</th>
<th>Column (Exterior)</th>
<th>Column (Interior)</th>
<th>Beam</th>
<th>PT force for beam-to-column connection before elastic shortening (KN)</th>
<th>PT force for column base connection before elastic shortening</th>
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<tr>
<td>6</td>
<td>W14x283</td>
<td>W36x170</td>
<td>W30x90</td>
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<td>5</td>
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<td>W36x194</td>
<td>W36x135</td>
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<td>W36x194 (splice)</td>
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<tr>
<td>2</td>
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<td>W36x328</td>
<td>W36x182</td>
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<td>W36x328</td>
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<td></td>
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<tr>
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<td>W14x398</td>
<td>W36x328</td>
<td>W36x328</td>
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<td></td>
</tr>
</tbody>
</table>

*For each column base, 1557 KN is applied.

Ground motions used in analyses

Six scaled ground motion records for the LA region were used for nonlinear dynamic analysis. These records were scaled to the DBE and MCE level based on the procedure by Somerville et al. (1997). The MCE has probability of exceedance of 2% in 50 years and the DBE has 2/3 intensity of the MCE. The ground motions and scale factors used are presented in Table 2.
Table 2. Ground motions

<table>
<thead>
<tr>
<th>Ground motion</th>
<th>Scale factor</th>
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<tr>
<td></td>
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<tr>
<td>ARTI, ARTIFICIAL</td>
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<td>LP89G03, LOMA PRIETA, 1989</td>
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<td>SE36, MIYAGI-OKI, 1978</td>
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<td>NR94TAR, NORTHRIEDE, 1994</td>
<td>1.091</td>
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</table>

Analysis results for SC-MRFs with post-tensioned column base connections

To investigate the static behavior of the SC-MRF, the frame was pushed laterally to 5% roof drift angle, which is roof displacement divided by the total height. The applied load distribution ratio over the height was based on the IBC 2000. The normalized base shear (V/W), base shear (V) divided by the seismic weight, versus roof drift angle is shown in Fig. 5. First, the SC-MRF was pushed to 3% roof drift angle and then unloaded because 3% roof drift angle, approximately, is expected for the conventional steel moment resisting frames under the DBE based on Garlock (2002). For 3% roof drift angle, the maximum ratio of M/Mp, developed moment (M) divided by the plastic moment (Mp), in beams was 1.02. The top of the exterior 1st story column yielded due to combined axial force and bending moment, and yielding by column flange bearing due to gap opening occurred at column base D. When unloaded, the frame showed negligible residual drift. The residual drift was within the limit, which is set at H/500, or 0.002 radians, based on acceptable out-of-plumbness (AISC, 2005). For 5% roof drift angle, the maximum ratio of M/Mp in beams was 1.19. The top of 1st story column E reached its capacity because of the increase of the PT beam to column connection moment and axial force by overturning. It was assumed in the model that after gap opening the PT beam-to-column connection has constant stiffness proportional to the slope of EA/L (A: total area of strands, L: total length of strands) and beam flange buckling does not occur. Even though the SC-MRF shows residual drift angle of 0.004 for 5% roof drift, overall, the frame showed self-centering behavior. The over-strength of the SC-MRF was 2.4 for the 5% roof drift angle.

Figure 5. Normalized base shear versus roof drift

Nonlinear dynamic analyses were conducted for the six scaled ground motions listed in Table 2 to evaluate the SC-MRF with PT column bases. Residual drifts for the six ground motions and mean residual drifts are given in the Fig. 6 and Fig. 7, respectively. The residual drifts under the DBE were within the limit of 0.002. The residual drift at the 1st story was the smallest because the inelastic
deformation at the column bases was replaced by the gap opening of the PT column bases. Under the MCE, the mean residual drifts were also within the limit. This means that the SC-MRF with the PT column bases shows self-centering behavior under the DBE and MCE. The roof displacement for the Kobe ground motion given in Fig. 8 indicates self-centering behavior for the frame for the DBE and MCE.

The mean maximum story drifts are presented in Fig. 9. The story drifts of the 1st floor and 6th floor were large compared to those of the other stories. Although the 1st and 2nd story drifts for the Loma Prieta DBE were more than 0.03 radians, the columns were within their capacity and showed self-centering behavior. The maximum ratio of M/Mp in the 1st story beams was 1.04. The demand and capacity curves at the top of the 1st story column E are shown in Fig. 10. The axial-moment interaction diagram by Bruneau et al. (1998) was used for the capacity curve.

Figure 6. Residual drifts for six ground motions after earthquake

Figure 7. Mean residual drifts after earthquake

Figure 8. Roof displacement for Kobe earthquake
CONCLUSIONS

A post-tensioned column base connection was proposed. Deep columns are recommended for the interior columns to provide enough stiffness for acceptable drift values. Additional shear elements are required at the column bases for the base shear. The initial level of the post-tensioning should be determined such that the PT bars remain elastic for the DBE and MCE. Energy dissipation plates need to be designed to ensure self-centering at the column base. The response of the SC-MRF with the PT
column base connection was presented. Based on the static and dynamic analysis results, it is concluded that the inelastic deformation at the column bases in the SC-MRF can be removed and the SC-MRF with PT column bases shows self-centering behavior with negligible damage. An experimental study for the PT column base will be conducted to validate the behavior of the PT column base connection.

ACKNOWLEDGMENTS

This material is based on work supported by the National Science Foundation, Award No. CMS-0420974, in the George E. Brown, Jr. Network for Earthquake Engineering Simulation Research (NEESR) program. The authors would like to acknowledge all the project members, and, in particular, Prof. R. Sause, Prof. J.M. Ricles and Prof. M.M. Garlock for their invaluable input and suggestions.

REFERENCES


