REDUCING THE FORCES IN CONTROLLED ROCKING STEEL BRACED FRAMES USING PARTIAL DUCTILE BEHAVIOR

Taylor C. Steele\(^1\) and Lydell D. A. Wiebe\(^2\)

ABSTRACT

The frame members in low-damage controlled rocking steel braced frames (CRSBFs) are normally selected to remain elastic during design-level earthquakes. This implies that they must be designed for the peak forces not only from the first-mode pushover response, but also from higher-mode forces. To mitigate these higher-mode effects, past research has suggested using multiple nonlinear mechanisms, such as additional rocking joints above the base or a self-centering energy dissipative brace at the first storey. As an alternative, one might instead consider permitting limited member buckling and yielding. This paper presents nonlinear time history analysis results for three 12-storey CRSBFs that were designed for the first-mode pushover forces plus the higher-mode forces at varying hazard levels. The first CRSBF was designed for only the first-mode pushover forces, but collapsed during more than three quarters of the ground motions. The second CRSBF was designed considering the higher-mode forces at the DBE level, which reduced the number of collapses caused by frame member buckling, but the members were often damaged such that their compressive resistances reduced to less than 50% of their initial values. The third CRSBF was designed for the higher-mode forces at the MCE level, which limited the frame member forces to the median demand from the ground motion suite, prevented collapse of the frame during all MCE-level ground motions, and reduced the severity of member buckling. This study suggests that buckling in the frame members of CRSBFs can cap the peak member forces, but this can quickly lead to collapse, and it is difficult to control the level of structural damage after buckling initiates. Therefore, it is generally not advisable to design the frame members for less than the MCE-level higher mode forces.

\(^1\)Ph.D. Candidate, Dept. of Civil Engineering, McMaster University, Hamilton, Canada
\(^2\)Assistant Professor, Dept. of Civil Engineering, McMaster University, Hamilton, Canada (wiebel@mcmaster.ca)

Reducing the forces in controlled rocking steel braced frames using partial ductile behavior

Taylor C. Steele\(^1\) and Lydell D. A. Wiebe\(^2\)

**ABSTRACT**

The frame members in low-damage controlled rocking steel braced frames (CRSBFs) are normally selected to remain elastic during design-level earthquakes. This implies that they must be designed for the peak forces not only from the first-mode pushover response, but also from higher-mode forces. To mitigate these higher-mode effects, past research has suggested using multiple nonlinear mechanisms, such as additional rocking joints above the base or a self-centering energy dissipative brace at the first storey. As an alternative, one might instead consider permitting limited member buckling and yielding. This paper presents nonlinear time history analysis results for three 12-storey CRSBFs that were designed for the first-mode pushover forces plus the higher-mode forces at varying hazard levels. The first CRSBF was designed for only the first-mode pushover forces, but collapsed during more than three quarters of the ground motions. The second CRSBF was designed considering the higher-mode forces at the DBE level, which reduced the number of collapses caused by frame member buckling, but the members were often damaged such that their compressive resistances reduced to less than 50% of their initial values. The third CRSBF was designed for the higher-mode forces at the MCE level, which limited the frame member forces to the median demand from the ground motion suite, prevented collapse of the frame during all MCE-level ground motions, and reduced the severity of member buckling. This study suggests that buckling in the frame members of CRSBFs can cap the peak member forces, but this can quickly lead to collapse, and it is difficult to control the level of structural damage after buckling initiates. Therefore, it is generally not advisable to design the frame members for less than the MCE-level higher mode forces.

**Introduction**

Controlled rocking steel braced frames (CRSBFs) are self-centering lateral force resisting systems that are designed to mitigate structural damage during earthquakes at and beyond than the design level. Prestressed post-tensioning (PT) tendons and the frame self-weight provide the restoring force to self-center the system after rocking, while energy dissipation (ED) may be used to reduce the displacement demands; the combination of these mechanisms results in a flag-shaped hysteresis as shown in Fig. 1(a). The frame members in CRSBFs are all capacity-protected elements and are normally designed to remain elastic during design-level earthquakes. Based on traditional capacity design principles, the frame members must be designed to resist the peak forces from the first-mode pushover response, which depend on the PT and ED elements. However, several researchers have shown that because the force-limiting mechanism is rocking only at the

---

\(^1\)Ph.D. Candidate, Dept. of Civil Engineering, McMaster University, Hamilton, ON L8S 4L7
\(^2\)Assistant Professor, Dept. of Civil Engineering, McMaster University, Hamilton, ON L8S 4L7

Several methods have been proposed to account for higher mode effects during the design of controlled rocking steel braced frames, with varying levels of simplicity and accuracy [1, 2, 3, 5, 6]. Despite the number of studies on the forces that can develop due to the higher-mode response, there is no consensus on which intensity should be used for design. Regardless of the method used, the large capacity design forces often lead to large frame member sizes. As an alternative to designing for the full contribution of these higher modes to the design forces, past research has suggested mitigating these forces by using additional nonlinear mechanisms, such as an additional rocking joint above the base or a self-centering energy dissipating brace at the first story [4, 5]. Hybrid ductile-rocking systems can also be used to limit the higher-mode forces by providing buckling-restrained braces in place of the capacity protected braces [7]. As an alternative, and recognizing that the higher modes are associated with small high-frequency displacements, a designer might instead consider permitting limited member buckling and yielding to control the frame member forces and reduce the required frame member sizes.

This study evaluates the ability of limited frame member buckling and yielding to reduce the frame member forces in a 12-story controlled rocking steel braced frame while still maintaining a low level of seismic damage. Three example CRSBFs are designed for the forces associated with the higher modes at intensities lower than what have been used in previous studies. The three frames are all designed using the same base rocking joint, but with different frame member sizes depending on the intensity considered for the higher modes. For each of the three frames, a unique set of ground motions is conditionally selected to represent the Maximum Considered Earthquake (MCE) level based on the site conditions and the first-mode period. Numerical models of the frames are developed for nonlinear dynamic analysis using the selected ground motions, and the peak frame member forces and the peak interstory drifts are compared for the three designs.

**Design of Example CRSBFs**

Three example CRSBFs serve as alternative lateral force resisting systems for a 12-story building on a site of high seismicity in the western USA, with a short-period spectral acceleration of $S_s = 1.5 \text{ g}$ and a one-second period spectral acceleration of $S_1 = 0.6 \text{ g}$. The site is Class D as

![Figure 1. Characteristic behavior of controlled rocking steel braced frames during design-level earthquakes](image)
defined in ASCE 7-16 [8]. The building has equal story heights of 4.57 m, where each floor and roof has a total seismic weight of 10 200 kN and 6430 kN, respectively, over a plan area of 2010 m$^2$.

The design of CRSBFs can be separated into two components (e.g. [5, 9]). First, the ED and PT are designed to meet selected displacement limits at specified hazard levels and to provide acceptable safety against collapse caused by over-rotation of the CRSBF. Second, the frame members and their connections are capacity designed for the forces that develop in the frame when rocking, including an estimate of the higher-mode forces. Further details are provided in the following subsections.

**Base Rocking Joint Design**

The 12-story structure was designed with four frames in each direction. Special connection details were assumed to decouple the uplift of the CRSBF from the gravity system during rocking (e.g. in [1, 2, 10]). Therefore, none of the frame members in the CRSBFs carry any of the tributary gravity loads, and the frames were designed instead to be 8.23 m wide to fit between the gravity columns at each end of the 9.15 m wide bays. The equivalent lateral forces were calculated using the modeled first-mode period and a response modification factor of $R = 9$ and were distributed over the height in accordance with ASCE 7-10 [8], resulting in an overturning moment of 30 850 kN-m per frame. While it has been suggested that even larger response modification factors could be used to design flexible CRSBFs [9, 11], the response modification factor for the 12-story structure was limited to $R = 9$ to prevent uplift of the frames under wind loading.

The base rocking joint was designed to be fully self-centering, with an hysteretic ED ratio, $\beta$ (defined as the ratio of the height of the flag to the linear limit), of 0.8. The ED mechanism was selected to be a set of frictional interfaces, each with a specified slip load of 1510 kN, located at the base of each vertical strut. The PT was prestressed to 35% of its ultimate strength and anchored at the top of the ninth story, such that the strands would remain elastic up to a base rotation of at least 2.5%, and such that the frame would have a positive tangent stiffness while the PT responds in the elastic range [6]. Two sets of 23 high-strength steel PT strands were required for each frame, each having an elastic modulus of 195 GPa, yield strength of 1670 MPa, and ultimate strength of 1860 MPa [12]. Based on the parameters that define the flag-shaped hysteresis for this base rocking joint design, the displacements were expected to be much less than an acceptable 2.5% limit at the MCE level [11].

**Capacity Design of the Frame Members**

Many methods have been proposed in past research to estimate the forces that develop in CRBSFs considering the higher modes [1, 2, 3, 4, 5, 6]. The dynamic method proposed by Steele and Wiebe [6] was selected for this study because it has been shown to be a practical and accurate method to estimate the force demands for a specific intensity level. In this paper, the intensity used to estimate the higher-mode forces for design is expressed as a ratio, $\gamma_{HM}$, of the DBE-level intensity. The first frame was designed for only the peak forces expected from a first-mode pushover analysis (i.e. using $\gamma_{HM} = 0$). Two other designs included higher-mode responses at the DBE level, amplified by factors of $\gamma_{HM} = 1.0$ (equivalent to the DBE level), and $\gamma_{HM} = 1.5$ (equivalent to the MCE level).
Fig. 2 shows the frame member compression force design envelopes for the braces, vertical struts, and horizontal struts. When sizing the frame members, the effective length factors of the braces and horizontal struts relative to the center-line dimensions were initially assumed to be 0.8 and 1.0, respectively. The effective length factors were updated based on their actual modelled length after each design iteration to minimise the weight of the frame members while still providing a capacity-to-demand ratio larger than 1.0 in accordance with AISC 360-16 [13], assuming a yield stress of $F_y = 345 \text{ MPa}$.

**Numerical Modeling of Example CRSBFs**

The non-linear time history analyses for each design case were performed using OpenSees [14]. Fig. 3 shows a schematic of the model used in the analyses. The uplift due to rocking and transfer of the base shear were modelled using gap elements connected to the base nodes in the vertical and horizontal directions, with axial stiffness equal to that of the vertical strut section between the top of the foundation and the center line of the horizontal strut at the base; springs with negligible stiffness were included in parallel with the gap elements for numerical stability. The braces, beams and columns were modelled using force-based beam-column elements with the Gauss-Lobatto integration scheme and five integration points to account for the spread of plasticity. Uriz and Mahin [15] showed that three integration points is the minimum number that can maintain the accuracy of the solution, given that a sufficient number of elements is provided. The buckling response was modelled using four elements for each member, with initial out-of-straightness equal to typical design tolerances of $L/1000$. Sensitivity analyses not presented here showed that four elements were sufficient to simulate inelastic buckling at the appropriate critical load. The vertical struts and horizontal struts were given a sinusoidal initial out-of-straightness in alternating directions to perturb frame buckling modes where these continuous members may buckle over multiple storeys. The out-of-straightness was specified in the same direction for all of the brace members, as they can buckle independently of one another. Although the CRSBFs were modelled in a 2D plane, a 3D model space was specified to allow the members to buckle out-of-plane about their weak axes. An elastic-frame model was also made for comparison, which was identical to the inelastic-frame model, except that the members were all included as single elastic beam-column elements so that member yielding and buckling would not be simulated.

To model the response of steel, fiber sections were used with the Giuffrè-Menegotto-Pinto (Steel02) material with a yield stress of 345 MPa, a post-yield stiffness ratio of 0.5%, and default isotropic and kinematic hardening parameters, combined with an initial stress material to account...
for the residual stresses in hot-rolled sections. The maximum tensile and compressive residual stresses were assumed to be equal to the product of the residual stress ratio, $\gamma_r$, and yield stress with a simplified linear approximation between peaks. A value of $\gamma_r = 0.4$ was used based on calibration of a component model with the compression member buckling curve in AISC 360-16 [13].

PT fracture was modeled using the model proposed by Ma et al. [3], with a maximum strain of 1.3% before the first wire fractures, and a gradual decrease in the PT force until complete fracture at a strain of 4.8%. Elastic buckling of the tendons at negligible compressive loads was modeled using gap elements in series with the PT elements. The energy dissipating frictional interfaces were modeled using truss elements with the elastic-perfectly plastic constitutive relationship.

A leaning column was included to model P-Delta effects. The connection details between the gravity framing and the CRSBF were not modeled explicitly; rather, only the horizontal degrees-of-freedom of the leaning column nodes and the center frame nodes are constrained to transfer the lateral forces while allowing the frame to uplift freely relative to the gravity system.

Inherent damping was modeled using 5% Rayleigh damping proportional to the mass and stiffness matrices, where the stiffness-proportional portion came from 20% of the initial stiffness matrix, and 80% of the committed tangent stiffness matrix to limit the negative numerical damping forces generated by the geometric effects of the buckled members. Stiffness proportional damping was not applied to the gap elements, the PT elements, or the ED elements. The Rayleigh damping coefficients were calculated using the second-mode natural frequency determined from modal analysis and an estimate of the equivalent design-level secant frequency that was calculated using

![Schematic of numerical model used for the nonlinear time history analyses](image-url)
the first-mode natural frequency, $\omega_1$, and the expected ductility of the building [16]. The resulting equivalent design-level secant period was approximately 10 s for all three example CRSBFs. This damping model was selected for the analyses in this study to avoid the artificially high damping forces that can develop using only initial stiffness-proportional damping when the members develop a negative stiffness, while also preventing the energy that can be introduced into the system when members experience a negative tangent stiffness due to geometric effects while buckling. This damping model was also selected to avoid over-damping either the first-mode rocking response or the second-mode response that was expected to dominate the contribution of the higher modes to the force demand in the frame members.

**Ground Motion Selection**

The ground motions in this study were selected to match conditional spectra (both the conditional mean spectrum and variability around it) based on the first-mode period and the site conditions using the ground motion selection tools provided by Baker et al. [17]. A suite of 30 ground motions was selected for each of the three frames to represent the spectral intensity at the first-mode period at the MCE-level. The three different designs had first-mode periods of 2.54 s, 2.80 s, and 3.48 s. The ground motions were selected in this way, rather than using a uniform hazard spectrum, to avoid conservative bias in the long-period range when the frames rock, and also to avoid the conservative bias in the higher modes that dominate the force demand and initiate frame member buckling. Because of the range of fundamental periods for the designs, different conditional spectra were used for each frame.

The ground motions were selected and scaled to match both the target mean and the target standard deviation based on the ground motion prediction equations using the software tools provided by Baker et al. [17] at 20 logarithmically-spaced points between 0.1 s and 10 s. The selected ground motions have a magnitude greater than 6.0 and a closest distance from the fault between 3 and 50 km, and none of the ground motions was scaled by more than a factor of 10. Fig.

![Conditionally-selected ground motions used for the nonlinear time history analyses](image_url)
4 shows the conditional spectra that were used to select the ground motions, and response spectra for the 30 ground motion records that were selected for the frame designed using $\gamma_{HM} = 1.5$.

**Results of Nonlinear Time History Analyses**

This section presents the results of the nonlinear time history analyses for the three CRSBFs subjected to their respective ground motion suites. The analyses were completed using both the elastic and inelastic frame models for all of the frames. Only the inelastic-frame model could simulate the buckling and yielding of the frame members, whereas only the elastic-frame model could capture the total elastic force demand on the frame members. For all records during which the peak frame member forces did not exceed the nominal compressive resistance, the peak frame member force and interstory drift time histories were identical using both the elastic-frame and inelastic-frame model. A record was considered to have caused collapse when the peak interstory drift exceeded 10%. Past this point, even if the frames maintained dynamic stability, the gravity system was assumed to have lost its ability to carry the gravity loads.

The frame that was designed for only the first-mode pushover collapsed during 23 of the 30 ground motions that were selected for the analysis at the MCE level. Thus, it is clear that omitting the higher-mode forces from the capacity design forces for CRSBFs is not advisable. All of the simulated collapses were due to excessive buckling and yielding of the frame members. When the elastic-frame model was used, collapse was not simulated during any of the 30 ground motion records. This also demonstrates that improving the base rocking joint design alone would not be a sufficient collapse prevention strategy because of the vulnerability of the frame members to excessive buckling and yielding.

For the frame designed using $\gamma_{HM} = 1.0$, Fig. 5 shows the peak compressive forces that developed in the braces and vertical struts, and the peak interstory drifts, during the analysis for the Gulf of Aqaba ground motion scaled by a factor of 5.54. This record demonstrates the ability of moderate buckling and yielding of the frame members to cap the peak member forces during extreme ground motions. The forces experienced in the braces were reduced by a factor of up to

![Figure 5](image-url)  
**Figure 5.** Peak frame member compressive forces and peak interstory drifts for the frame designed using $\gamma_{HM} = 1.0$ for both elastic and inelastic frame models during the Gulf of Aqaba ground motion scaled by factor of 5.54.
2.0, and the peak compression in the vertical struts was reduced by a factor of up to 1.6. However, this same buckling of the third story braces also led to a concentration of interstory drift at the third level, and the peak interstory drift at this level was just outside the assumed MCE limit of 2.5%. Accepting moderate damage in the braces at any level due to buckling for the force reduction experienced through the rest of the frame members may be considered reasonable, given that this was an extreme record in the ground motion suite.

Fig 6. shows aggregate plots of the peak frame member forces that developed during all of the ground motions in the MCE-level record suites, both for the frame designed for $\gamma_{HM} = 1.0$ and for the frame designed for $\gamma_{HM} = 1.5$. For the frame designed for $\gamma_{HM} = 1.0$, the greatest reduction in the frame member forces was in the braces, where the peak compression forces were reduced by factors of up to 3.2. The peak compression forces in the vertical struts were reduced by factors of up to 2.3. However, the buckling of the braces in the lower stories was excessive enough to cause collapse during two extreme records, for which the elastic force demand in the braces was more than twice their nominal compressive resistance. With the exception of these extreme records, the frame members only buckled during 8 of the 30 records, and the peak interstory drifts were less than the assumed MCE-level limit of 2.5% for 60% of the records. Therefore, these results generally show that permitting the frame members to buckle and yield reduced the peak frame member forces without excessive median peak interstory drifts. The median peak frame member forces were not significantly reduced relative to the demands in the elastic frame model,

Figure 6. Peak frame member compression forces and peak interstory drifts for the two frames designed using $\gamma_{HM} = 1.0$ and $\gamma_{HM} = 1.5$ for both elastic and inelastic frame models.
because buckling was not initiated in the frame members during the majority of the ground motions below the median intensity. This is because the ground motions were conditioned on the MCE-level spectral intensity at the first-mode period, resulting in a median spectral intensity in the higher-mode periods closer to the DBE-level spectral intensity of 1.0 g rather than the MCE-level intensity of 1.5 g.

For the frame designed for $\gamma_{HM} = 1.5$, the greatest reduction in the frame member forces was also in the braces, where the peak compression was reduced by factors of up to 2.7. The forces in the columns were reduced by factors of up to 2.0. For this frame, while one extreme record caused buckling of the first-story braces that was significant enough to reach an interstory drift of nearly 6%, the additional compressive resistance of the frame members compared to the less conservative frame designs was sufficient to prevent collapse of the building during all 30 ground motions, and the frame members buckled notably (i.e. to a post-buckling strength less than 90% of the initial compressive resistance) during only four out of 30 records. As was the case for the frame designed for $\gamma_{HM} = 1.0$, the interstory drifts were well within the assumed MCE-level limit of 2.5% on average, and the interstory drift limit was exceeded during only five out of 30 records.

**Conclusions**

This study evaluated the effectiveness of permitting limited buckling and yielding of frame members to cap the peak frame member forces in CRSBFs during ground motions scaled to the MCE level, while still limiting damage. The results demonstrated that frame member buckling and yielding yields catastrophic results if the higher-mode forces are omitted completely from the capacity design forces. The frame designed for only the forces associated with the first-mode pushover response collapsed during 80% of the MCE-level ground motions, so this approach is not adequate for the design of CRSBFs. Including the higher-mode forces at even the DBE level (i.e. using $\gamma_{HM} = 1.0$) significantly reduced the number of records that caused collapse, and sufficiently capped the peak frame member forces throughout the CRSBF to the design envelope. However, this force reduction also results in reduced frame member strengths once they buckle under the dynamic forces, but the members usually retained a post-buckling strength of at least 50% of their initial compressive resistance. The buckling of the frame members still quickly led to collapse during two extreme ground motions when the peak elastic force demand exceeded the capacity by approximately 2.5 times or more, and both of the simulated collapses were due to excessive buckling of the braces in the lower stories; none of the simulated collapses were due to over-rotation of the frame. Including the higher-mode forces at the MCE level (i.e. using $\gamma_{HM} = 1.5$) resulted in a CRSBF that did not collapse during any of the 30 considered MCE-level ground motions, and reduced the number of records that caused frame member buckling from ten to four. The peak frame member forces were still reduced by up to 2.5 times during the extreme records, but the median peak frame member forces were still similar for both the inelastic-frame and the elastic-frame models. Given the number of records that caused buckling of the frame members and even collapse for the less conservative frame designs, it does not seem advisable to design CRSBFs for less that the MCE-level higher-mode forces.

This study was limited to a single 12-story building without mass, geometric, or stiffness irregularities, where the CRSBFs were designed using high-strength PT strands and frictional ED interfaces. While the comparisons between designs are not expected to depend on the assumptions made during this study, the numerical results are expected to depend on the gravity loads in the
structure, the structure geometry, the assumed inherent damping, the behavior of PT, and the type and behavior of ED used.

References


2. Eatherton MR, Hajjar JF. Large-scale cyclic and hybrid simulation testing and development of a controlled rocking steel building system with replaceable fuses. Report NSEL-025, Newmark Structural Engineering Laboratory, Urbana IL, USA 2010.


