Steel Innovations Conference 2015
Auckland, New Zealand
3-4 September 2015

CONTROLLED ROCKING STEEL BRACED FRAMES: CONNECTING RESEARCH AND PRACTICE

L. D. A. Wiebe[1]

ABSTRACT

Because of their potential to avoid structural damage during design-level earthquakes, controlled rocking steel braced frames (CRSBFs) are attracting interest from researchers and practicing engineers alike. In a CRSBF, selected columns are permitted to uplift from the foundation, resulting in a nonlinear response that limits peak seismic forces while also avoiding structural damage and residual deformations. This paper connects developments from research and from practice regarding three key issues in the design of CRSBFs: the base rocking joint, the capacity design of the frame, and the connection between the frame and the floor diaphragms. Designing the base rocking joint includes selecting supplemental energy dissipation and post-tensioning, and research is ongoing to develop expressions that will allow a designer to predict how the joint design will affect the peak seismic displacements. Capacity design of the frame must account for the forces that develop due to the higher modes. These forces can be estimated using methods that are based on response spectrum analysis, and they can be mitigated by designing multiple mechanisms. A variety of solutions have been proposed for the critical connections between the frame and the floor diaphragms, although most practical solutions have not yet been validated by large-scale experiments.

Introduction

The Canterbury earthquakes made the public aware of what many engineers already understood: that even well-designed traditional seismic force resisting systems do not necessarily prevent earthquake damage. Even before 2011, interest was already developing in seismic force resisting systems that would minimize structural damage during a major earthquake, and this interest has only increased since that time.

One system that is being promoted for minimizing earthquake-induced structural damage is a controlled rocking steel braced frame (CRSBF). Over the past decade, several buildings have been built in New Zealand with CRSBFs as the seismic force resisting system in at least one direction (e.g. Gledhill et al. 2008, Latham et al. 2013, Tait et al. 2013). At the same time, several major research programs have investigated the behaviour of CRSBFs based on both numerical analysis and large-scale experimental testing. However, CRSBFs do not yet have prescriptive design guidance in any building code worldwide. Therefore, in an effort to make the system more accessible to practicing engineers, Steel Construction New Zealand recently commissioned Aurecon to prepare a Design Guide on CRSBFs (SCNZ 2015).

In some respects, a CRSBF is similar to a conventional concentrically braced frame: both have the same behaviour under low levels of lateral load, as shown in Fig. 1a-b. However, the columns of a CRSBF are detailed to uplift rather than transferring tension to the foundation. Thus, when the base overturning moment is large enough to overcome the column precompression, the frame will rock as shown in Fig. 1c, and this rocking behaviour is the nonlinear mechanism that limits the peak seismic forces for capacity design. The frame is often post-tensioned to the foundation to provide a column precompression that increases the rocking load, and to provide a positive stiffness while rocking. When the lateral load is removed, the system

[1]Assistant Professor, Dept. of Civil Engineering, McMaster University, 1280 Main Street West, JHE 301, Hamilton, ON, Canada, L8P 2K1

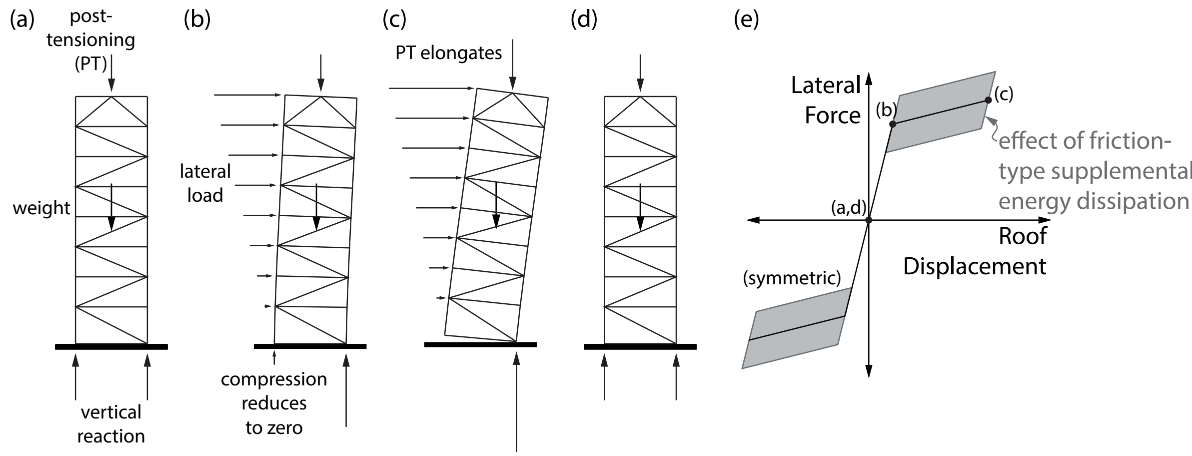

Figure 1 Schematic behaviour of a controlled rocking steel braced frame: (a) at rest; (b) incipient rocking; (c) maximum displacement; (d) complete unloading; (e) resulting hysteretic response

returns to the initial undeformed configuration (Fig. 1d). The behaviour of a symmetric CRSBF is the same in the reverse direction (Fig. 1e). A CRSBF is typically designed to allow individual frames within a building to rock, rather than allowing the entire building to rock as a unit. The frame is normally designed to remain essentially linear elastic, with the nonlinear behaviour limited to the rocking joint.

The rocking response described above is bilinear elastic, so supplemental energy dissipation that is activated by the rocking motion is often provided. This increases the rocking load and also dissipates energy during each rocking cycle, transforming a bilinear elastic hysteresis into a flag-shaped hysteresis, as shown in grey in Fig. 1e. If the energy dissipation is not nearly rigid in the elastic range, the system response in the first cycle (when the energy dissipation starts from zero load) will be different from the response is subsequent cycles (when the energy dissipation starts with a precompression).

Three issues that are critical for the design of a CRSBF are the base rocking joint (including the supplemental energy dissipation and post-tensioning), the frame itself, and the connection between the frame and the floor diaphragms. The purpose of this paper is to highlight research that has been performed on each of these three issues, and to identify how this knowledge can be applied in practice based on CRSBFs that have been built previously and on the recommendations of the Design Guide. The design of the floor diaphragms and of the foundation are also critical for the seismic performance of CRSBFs, but they are not expected to be substantially different from diaphragms and foundations for other seismic force resisting systems, and therefore are not discussed in this paper. Considerations regarding non-structural elements are also outside the scope of this paper because the storey displacements and accelerations of CRSBFs are expected to be similar to those of other seismic force resisting systems.

Key Issue 1: Base Rocking Joint Design

Components of Resistance to Base Rocking

The resistance to rocking at the base rocking joint, M_{rock}, is provided by three components: the gravity load that acts on the frame (M_W), any supplemental energy dissipation (M_{ED}), and any post-tensioning (M_{PT}):

$$M_{rock} = M_W + M_{ED} + M_{PT} \tag{1}$$

Neglecting the elastic deformations of the structure before rocking, the rocking resistance caused by the gravity loads acting on the frame is given by:

$$M_W = W \times d_W \tag{2}$$

where W is the total gravity load carried by the CRSBF and d_W is the horizontal distance from the rocking toe to the centroid of that load. The moment resistance provided by the supplemental energy dissipation is:

$$M_{ED} = ED \times d_{ED} \tag{3}$$

where ED is the linear limit of the energy dissipation device, and d_{ED} is the distance from the rocking toe to the line of action of the energy dissipation force. If there are multiple energy dissipation devices, Eq. 3 can be taken as a summation of individual contributions. Finally, the moment resistance provided by the post-tensioning is:

$$M_{PT} = PT_0 \times d_{PT} \tag{4}$$

where PT_0 is the total prestressing force and d_{PT} is the distance from the rocking toe to the line of action of the resultant vertical prestressing force.

Load-Bearing and Non-Load-Bearing CRSBFs

The connections between the floor diaphragms and the CRSBF can be detailed either to transfer gravity loads to the CRSBF, or to allow the CRSBF to uplift without transferring vertical loads. Both alternatives have been used in practice. Examples of load-bearing CRSBFs include Te Puni Village and Elevate Apartments in Wellington (Gledhill et al. 2008, Tait et al. 2013), and the Orinda City Offices in California (Mar 2010). Kilmore Street Medical Centre is an example of a CRSBF that was designed to be non-load-bearing (Latham et al. 2013). Most testing of CRSBFs for buildings has assumed that they carry no significant gravity load (Roke et al. 2010, Eatherton and Hajjar 2010, Ma et al. 2011, Wiebe et al. 2013a-b). Once a decision has been made regarding whether the CRSBF is to carry tributary gravity loads, M_W in Eq. 1 is fixed.

Defining the Target Rocking Load, Rocking Stiffness, and Energy Dissipation

A key step in designing a CRSBF is determining the load at which the system will be designed to rock, as well as the stiffness and energy dissipation that are associated with rocking. When selecting these parameters, the key objective is to ensure that the seismic displacement demand is less than the displacement capacity. This system-level displacement capacity depends on the component-level displacement capacity of the post-tensioning, supplemental energy dissipation devices, and gravity framing.

To estimate the seismic displacement demand, a CRSBF is often represented by a single-degree-of-freedom (SDOF) system with a flag-shaped hysteresis (e.g. Ma et al. 2011, Wiebe and Christopoulos 2014b). Christopoulos et al. (2002) found that the displacement ductility demand on such a system generally reduces with increasing linear limit, initial period, energy dissipation, and rocking stiffness. For the cases that were considered, it was possible to achieve the same ductility demand as an elastoplastic system with the same strength by adjusting the energy dissipation and nonlinear stiffness. Seo and Sause (2005) found that self-centring systems generally had a higher ductility demand than bilinear or stiffness-degrading systems with the same linear limit, but they also found that it was possible to make the ductility demands similar by adjusting the energy dissipation and nonlinear stiffness when the force reduction factor (R, similar to the structural ductility factor in New Zealand terminology) was no more than six.

Since the displacement capacity of a CRSBF is independent of the displacement at which rocking initiates, Wiebe and Christopoulos (2014b) considered the seismic displacement demand, rather than ductility demand, of SDOF systems with flag-shaped hystereses. They found that systems in California with an initial period of less than about 0.4 s would need to be designed with R of less than five to limit the interstorey drifts to 2.5% based on an assumed period-height relationship, while the rocking load could be almost arbitrarily low for systems with an initial period longer than 0.6 s.

Zhang and Wiebe (2015) compared the peak seismic displacements of SDOF systems with flag-shaped hystereses to those of linear elastic systems with the same initial period. Fig. 2 shows that the median displacements for systems with no rocking stiffness (i.e. the post-tensioning stiffness exactly counters the P-Δ effects) are generally larger than the displacements of the corresponding elastic systems, but the displacement ratio reduces as the initial period increases. The displacement ratio can also be reduced by increasing the rocking load or the energy dissipation. For $\beta = 100\%$ (i.e. maximum energy dissipation while remaining self-centring), the displacement of the rocking system is within 30% of that of the elastic system if $R = 4$ and the initial period is longer than about 0.65 s, while the initial period must be at least 1.5 s for $R = 8$. The displacement ratio is less if the rocking stiffness is very high, while a negative rocking stiffness (due to P-Δ effects) can lead to instability for systems with small linear limits (i.e. R greater than about 10). All of these results assume 5% initial stiffness-proportional damping, while preliminary research shows somewhat larger displacement ratios when tangent stiffness-proportional damping is assumed.

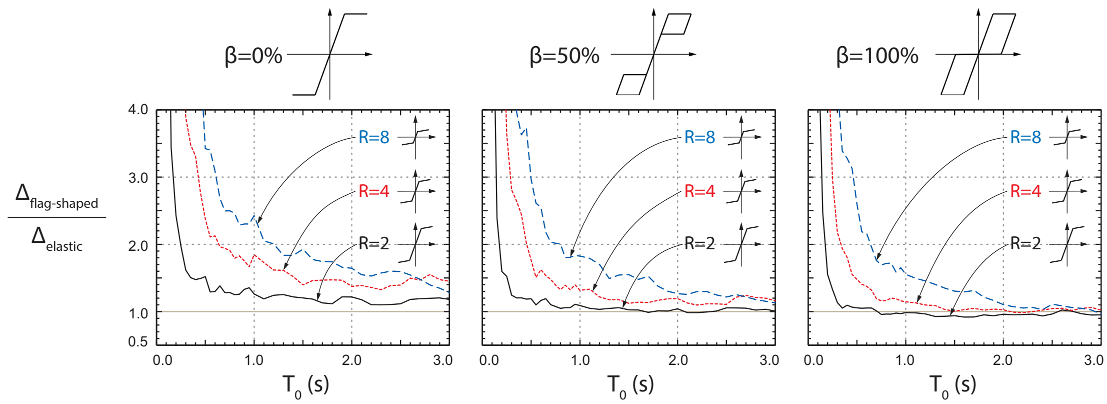

Figure 2 Comparison of displacements of SDOF systems with elastic and flag-shaped hysteretic behaviour for varying periods (T_0), supplemental energy dissipation (β), and linear limits (elastic demand divided by R), assuming 5% initial stiffness proportional damping

In recommending a structural ductility factor, the Design Guide (SCNZ 2015) distinguishes between load-bearing and non-load-bearing systems. For load-bearing systems, the floor slab must bend in order to uplift along with the CRSBF. This may cause damage, so the Design Guide recommends a ductility factor of $\mu = 1$ at the Serviceability Limit State (SLS), which translates to $\mu \leq 4$ at the Ultimate Limit State (ULS) for most of New Zealand. For non-load-bearing systems where the connections are detailed to allow the CRSBF to uplift without transferring vertical load to the floor system, the Design Guide suggests that some limited uplift could be permitted at the SLS level and recommends a ductility factor of $\mu \leq 6$ at the ULS level, consistent with the largest values in New Zealand practice.

Research is ongoing to develop equations that a designer could use to predict the displacement demand on a CRSBF. In the absence such equations, the Design Guide recommends multiplying the elastic displacements of the fixed-base frame under the code-based reduced lateral loads by μ and by an additional factor of 1.3 to account for an increase in displacements relative to linear elastic systems. The results shown in Fig. 2 suggest that this factor is likely to be a reasonable simplification for long-period structures and assuming initial stiffness-proportional damping, although it may underestimate the displacement demand on stiffer structures. The impact of this factor on design is mitigated by recommending that the displacements do not need to be further multiplied by the drift modification factor k_{dm}, which ranges from 1.2 to 1.5, because the interstorey drifts are expected to be essentially uniform over the height.

Energy Dissipation

The energy dissipation associated with impact of a CRSBF on a foundation is unlikely to be large enough to be relied upon for design. For example, free vibration tests of a 30% scale CRSBF without supplemental energy dissipation took about 25 cycles to damp out from an initial roof displacement of approximately 2.5% (Wiebe et al. 2012). The Design Guide (SCNZ 2015) recommends neglecting energy dissipation due to impact for design, and providing enough supplemental energy dissipation that M_{ED} is at least $0.4M_{rock}$.

Many different forms of supplemental energy dissipation have been considered for CRSBFs, both in research and in practice. Ma et al. (2011) used a buckling-restrained brace at the wall base in one set of shake table tests, and yielding steel plates in other tests. Eatherton et al. (2014a) conducted quasi-static cyclic testing on pairs of CRSBFs with similar plates as coupling elements. U-shaped flexural plates have been used in tests of controlled rocking coupled precast concrete walls (Priestley et al. 1999). Yielding base plates have been used without any post-tensioning by Midorikawa et al. (2006). Friction elements have also been considered, either at the base (Wiebe et al. 2013a-b) or as an integral part of the connections between the floor diaphragms and the frame (Roke et al. 2010). The latter case has the potential benefit of increasing the energy dissipation as the earthquake intensity increases. Viscous dampers have been included as part of a shake table test program reported by Tremblay et al. (2008).

In practical applications, Te Puni Village (Gledhill et al. 2008) and Elevate Apartments (Tait et al. 2013) were both designed to dissipate energy using friction plates together with Ringfeder springs. Fig. 3a shows that a Ringfeder spring consists of pairs of nesting inner and outer rings, which are arranged so that compression on the spring causes the inner rings to contract and the outer rings to expand, resulting in a hysteretic shape

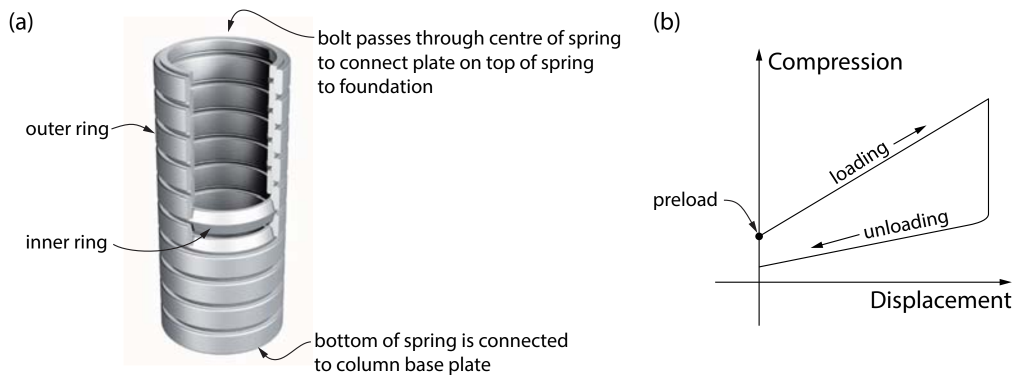

Figure 3 Ringfeder spring: (a) arrangement of rings (rendering courtesy of Lars Jahnel, Ringfeder Power Transmission GMBH); (b) idealized behaviour

that has stiffness because of the change in ring size and energy dissipation because of the friction between the rings (Fig. 3b). The Orinda City Offices used pairs of angles that connected the column bases to the foundation and were designed to yield when the frame rocks (Mar 2010). Kilmore Street Medical Centre (Latham et al. 2013) uses CRSBFs that are coupled with mild steel fuse rods that yield in both tension and compression, and also with lead extrusion dampers that have some velocity dependence.

Post-Tensioning

After the base moment contributions from gravity loads and supplemental energy dissipation have been determined, the remaining component to design is the post-tensioning. For a given target base rocking moment, the designer can select from a range of possibilities between a small post-tensioning area with a high initial stress, and a large post-tensioning area with a low initial stress. The initial stress affects the maximum rotation before the post-tensioning yields or fractures, while the cross-sectional area affects the overall system stiffness during rocking, which may affect the peak displacement demands. Studies of post-tensioning for applications in controlled rocking systems have shown that the anchorage detail may cause the post-tensioning to begin to fracture at strains that are much lower than the material strain capacity: Eatherton et al. (2014b) found that individual strands began to fracture at strains of approximately 1%, but this fracture was not necessarily catastrophic because it did not propagate to adjacent strands. They also emphasized that the strain capacity of the strands is sensitive to the strand and anchorage system.

The Orinda City Offices used post-tensioning strands that passed through a U-shaped duct in the foundation (Mar 2010), while Kilmore Street Medical Centre used 75 mm high strength Macalloy bars (Latham et al. 2013). Meanwhile, the CRSBFs that have been built with Ringfeder springs have not included post-tensioning (Gledhill et al. 2008, Tait et al. 2013), as the springs provide stiffness as the system rocks.

Eatherton et al. (2014b) identified global uplift as an important consideration in the design of a base rocking joint. Global uplift occurs when the post-tensioning and gravity loads are not sufficient to overcome the resistance of the supplemental energy dissipation to the rocking joint closing. This may result in a ratcheting response, where each cycle of loading increases the displacement demand on the energy dissipation, potentially leading to early failure. Global uplift can be avoided by ensuring that the post-tensioning force is greater than the energy dissipation force with appropriate allowance for cyclic strain hardening (Eatherton et al. 2014b). In addition, Wiebe (2013) showed how placing the energy dissipation at the corners of the CRSBF can often avoid global uplift because equilibrium requires the device in compression to be loaded more heavily than the device in tension, ensuring that the compression device will close before the tension device can extend, assuming that the devices have at least as much strength in tension as in compression.

Base Shear Transfer

The rocking joint must also transfer the base shear to the foundation. Wiebe et al. (2013b) observed that the base shear during shake table testing was not always in the direction that would be expected based on the direction of gap opening, because of the influence of the higher modes. Most experimental tests of CRSBFs have used a bumper system at the base (e.g. Roke et al. 2010, Wiebe et al. 2013a, Eatherton et al. 2014a), as shown in Fig. 4. A similar technique was used for Kilmore Street Medical Centre (Latham et al. 2013), whereas Te Puni Village (Gledhill et al. 2008) and Elevate Apartments (Tait et al. 2013) used a plate that was welded to the underside of a beam at the base of the CRSBF and that fit between two plates that extend up from the foundation.

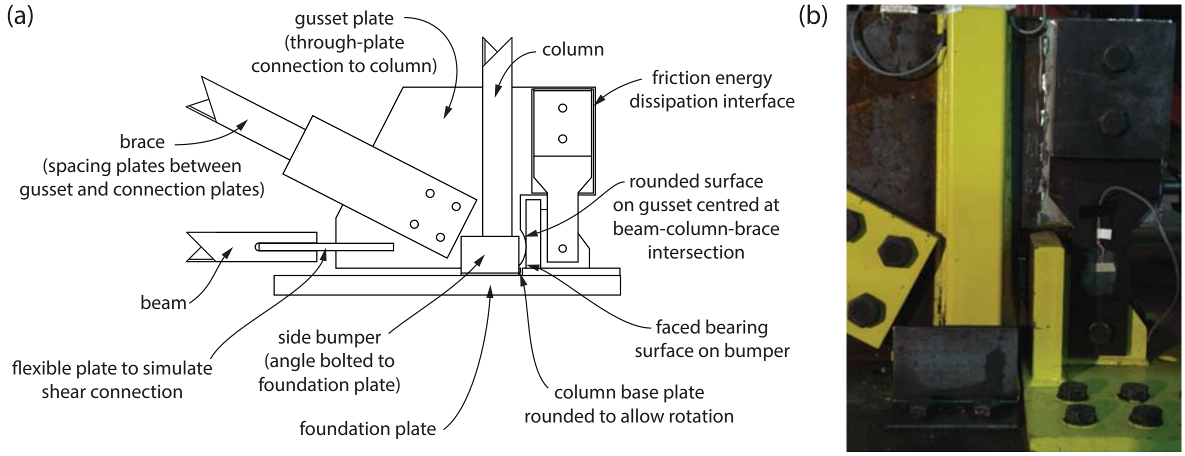

Figure 4 Base shear transfer detail (after Wiebe et al. 2013a): (a) schematic; (b) photograph

Key Issue 2: Capacity Design of Frame for Higher Mode Effects

Causes of Force Demands in Frame Members

Although the rocking joint limits the maximum overturning moment that can develop at the base of a CRSBF, this mechanism does not completely limit the forces that can develop in the members of the frame. Rather, these forces are influenced, and may even by dominated, by the response in the higher modes (e.g. Wiebe and Christopoulos 2014c). This has been demonstrated by shake table testing of an eight-storey CRSBF at 30% scale: for example, when one record was scaled up by 100%, the maximum base overturning moment increased by only 36% because it was limited by the rocking mechanism, but the base shear increased by 86%, with a commensurate increase in the first-storey brace force (Wiebe et al. 2013b). This higher mode demand is similar to what occurs in reinforced concrete shear walls, where the higher modes are not fully cut off by a base rotational hinge. Higher mode effects are particularly important for taller buildings or buildings that have been designed to rock at a relatively low load (Wiebe and Christopoulos 2014c).

An intuitive concern for the design of frame members is the amplification of column forces following impact with the foundation. However, large-scale shake table testing has shown that this does not cause significant forces for CRSBFs that do not carry significant gravity loads other than their own weight (Ma et al. 2011, Wiebe et al. 2013c). This is because very little mass is available to be excited following impact, and the peak column forces instead develop when the post-tensioning and supplemental energy dissipation reach their maximum force because the system is at its maximum displacement. Conversely, the amplification of column forces due to impact is non-negligible for CRSBFs that carry significant gravity load, such as steel bridge piers (Pollino and Bruneau 2008).

Calculation of Capacity Design Forces

Several strategies have been proposed for calculating the design forces on the members of CRSBFs (e.g. Roke et al. 2009, Eatherton and Hajjar 2010, Ma et al. 2011, and Wiebe and Christopoulos 2014a,c). A recent paper by Steele and Wiebe (2014) compared four different capacity design methods for CRSBFs:

(1) A theory-based approach by Wiebe and Christopoulos (2014a,c) that is based on the assumptions that a CRSBF behaves like a cantilever with uniformly distributed mass and elasticity, and that the first mode is limited by the base rocking mechanism while the higher modes act on a system with negligible rotational stiffness at the base. This method cannot be calculated by commercial software models, but can be calculated using a spreadsheet.

(2) A modified response spectrum analysis, in which a model of the fixed-base structure is subjected to an inverted triangular load distribution that is scaled to cause the maximum expected base overturning moment. The resulting frame member forces are referred to as the first-mode forces, R_1. The higher-mode forces (R_n for each mode n) are calculated from an elastic response spectrum analysis of the fixed-base structure. The first set of forces is added to the second to calculate the total actions, R_T:

$$R_T = R_1 + \sqrt{R_2^2 + R_3^2 + \ldots + R_n^2} \tag{5}$$

(3) A modified modal superposition method following Priestley et al. (2007). This method is essentially the same as method (2), except that all actions are combined using the square root of the sum of squares:

$$R_T = \sqrt{R_1^2 + R_2^2 + R_3^2 + \ldots + R_n^2} \tag{6}$$

(4) A proposed alternative method that is similar to method (2), except that the higher-mode contributions are calculated using a model that is modified to account for the effects of column uplift.

Three different heights of CRSBFs were designed, and nonlinear time history analyses were conducted to determine the force demands assuming that the frame members remained elastic. The ground motions were scaled to the maximum considered earthquake (MCE) level, and all methods were calculated using the 84th-percentile spectrum from the suite of ground motions in order to avoid buckling and yielding of elements under most MCE-level ground motions. Fig. 5 shows the peak compressive forces in the columns and braces of the 12-storey CRSBF, which was designed with a very low rocking load (no rocking under code wind loads). Methods (2) and (3) calculate all forces using the fixed-base structure, so they underestimate the column compressions (related to moments) around where the second mode of a fixed-base cantilever has zero moment, while they overestimate the brace forces near the base, where the second-mode shear is larger than for a pinned-base structure (Wiebe and Christopoulos 2014a). Methods (1) and (4) calculate the column compressions more conservatively than intended over the full height, but they are reasonably close to the 84th-percentile results for the braces. The differences would likely have been less if the base rocking joint had been designed to rock at a higher load.

In a different study, Steele and Wiebe (2015) conducted nonlinear time history analyses where the potential buckling or yielding of CRSBF members was modelled. A six-storey CRSBF was designed with a force reduction factor of eight and with capacity design using method (4), but with the higher modes scaled to: (a) zero, (b) the design basis earthquake (DBE); (c) the MCE; and (d) 1.5 times the MCE (approximately the 84th percentile of the MCE ground motion suite). The first two designs both collapsed during one out of seven MCE-level analyses. In contrast, the frames that were designed for higher mode effects at or above the MCE level did not have any member inelasticity during any of the records. The results demonstrated the influence of the higher modes in capacity design, but they also suggested that it may not be necessary to design to completely preclude member inelasticity during very large ground motions.

The SCNZ Design Guide (2015) recommends calculating the capacity design forces using a modified response spectrum analysis approach (method (2) above). Detailed guidance is not given on the issue of column force amplification due to impact for load-bearing systems, as they have a ratio of vertical to horizontal tributary seismic mass that is between the ratio for non-load-bearing systems (nearly zero) and bridge piers (unity).

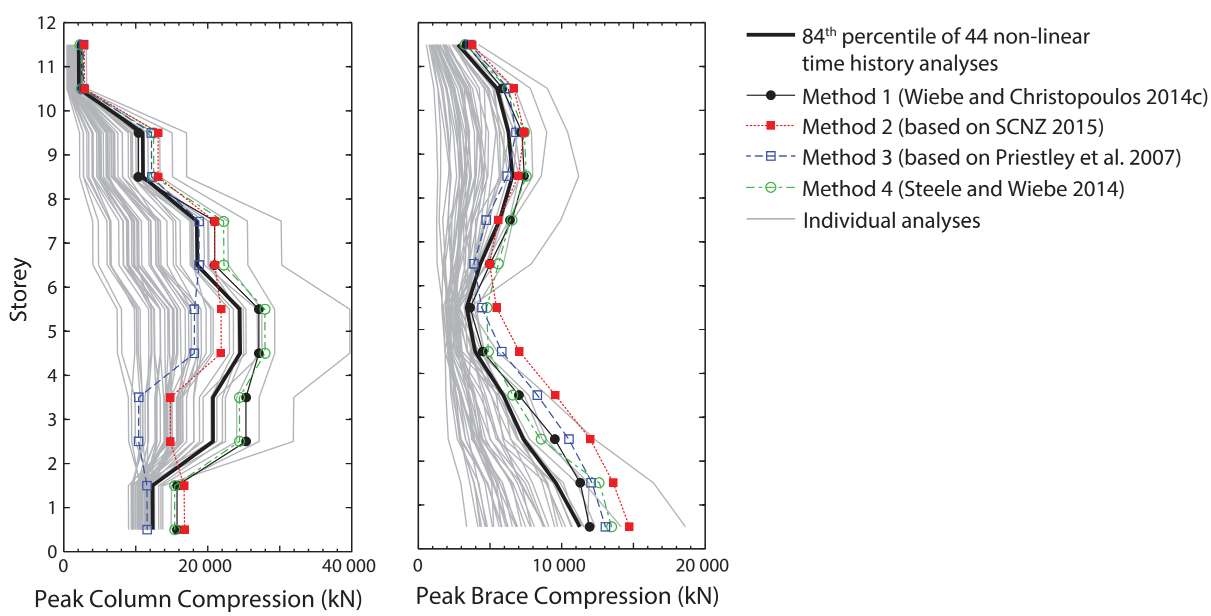

Figure 5 Comparison of capacity design methods that target the 84th-percentile forces to the peak forces from nonlinear time history analyses of a 12-storey CRSBF (after Steele and Wiebe 2014)

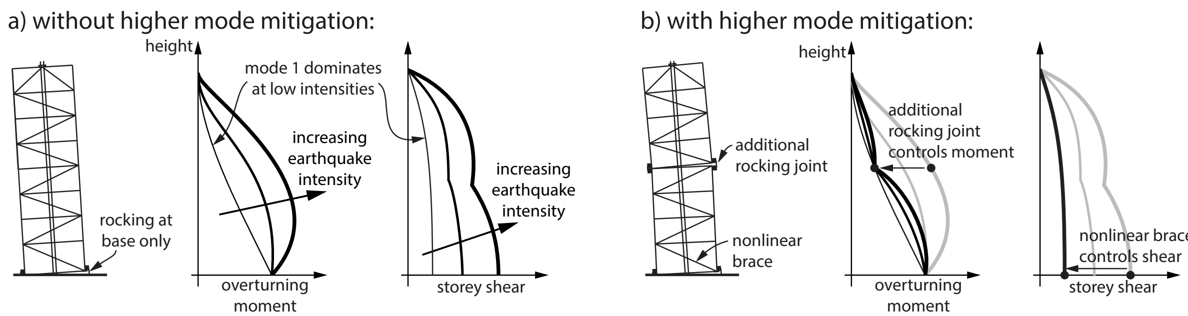

Figure 6 Higher mode mitigation concepts, adapted from Wiebe et al. (2013a)

Mitigation of Higher Mode Effects

If higher mode effects dominate the capacity design forces, they can be mitigated by providing multiple rocking joints or by designing one or more braces to have a nonlinear response, as shown schematically in Fig. 6. In large-scale shake table testing, Wiebe et al. (2013b) showed reductions in the peak base shear of up to 37% by designing for rocking to occur at two locations, and this reduction was increased to 54% by also replacing the first-storey brace with a self-centring energy dissipative brace. A preliminary design approach for higher mode mitigation has been proposed by Wiebe and Christopoulos (2014c).

Key Issue 3: Connections between Frame and Floor Diaphragms

Non-Load-Bearing Systems

Most research has assumed that the CRSBF will be separated from the gravity framing system. Large-scale shake table testing has done this using long pin-ended struts to transfer horizontal forces while allowing vertical uplift (e.g. Ma et al. 2011, Wiebe et al. 2013a), but these details were not developed for use in practice. Large scale testing has confirmed the quasi-static performance of a knuckle detail that transfers horizontal diaphragm forces in bearing from a non-uplifting gravity column to the CRSBF column (Roke et al. 2010). Eatherton and Hajjar (2010) proposed using plates oriented to bend about their weak axis to connect the CRSBF beams with collector beams, but this detail has not been validated in large-scale tests of a CRSBF. Latham et al. (2013) have discussed several design alternatives for the connection detail in a non-load-bearing system, including the option that was selected for the Kilmore Street Medical Centre.

Load-Bearing Systems

Comparatively little research has been done on load-bearing CRSBFs for buildings. Although gravity loads on a CRSBF increase its resistance to rocking and provide a restoring force to avoid global uplift, there are concerns that a CRSBF with significant uplifting mass may experience impact forces, as has been studied for bridge piers where the seismic masses are equal in the vertical and horizontal directions (Pollino and Bruneau 2008). Although this is not expected to be an issue for non-load-bearing systems, as discussed earlier, no guidance is currently available for the intermediate case of load-bearing CRSBFs in buildings.

The effect of the floor system in restraining uplift is also a potential concern that should be considered in design. Henry et al. (2012) examined this issue for controlled rocking precast concrete walls, and found that although the damage to the floor slab was likely to be reparable after loading to 2% lateral drift, the increase in force on the wall due to slab restraint was significant (up to 50%) when a rigid cast-in-place connection was used. To avoid this issue in practice, the joint between the CRSBF column and the adjacent floor beam has sometimes been designed to transfer shear and axial forces while allowing free rotation (e.g. Mar 2010).

Conclusions

This paper summarized academic research and design practice related to three key issues in the design of controlled rocking steel braced frames (CRSBFs): the base rocking joint, the capacity design of the frame, and the connection between the frame and the floor diaphragms. The resistance to rocking at the base is provided by gravity loads, which depend on how the connection to the floor diaphragms is detailed, together with energy dissipation and post-tensioning. The rocking joint can be designed using a variety of different supplemental energy dissipation technologies, and there are also multiple design alternatives for the post-tensioning and base shear transfer. Although the displacements of CRSBFs are generally larger than the displacements of elastic systems with the same initial period, the displacements can often be made similar by selecting appropriate levels of rocking load and energy dissipation. Research is ongoing to develop design-level expressions for estimating the peak displacements of a CRSBF.

The capacity design of the frame is likely to be heavily influenced by the higher modes. Design tools are available to estimate the design forces using modified response spectrum analysis techniques, but the designer must select the intensity of response to consider because the rocking behaviour does not fully limit the frame forces. Excessive design forces can be reduced by providing multiple mechanisms.

Several different floor-frame connection details have been used in practice, including some details that are intended to transfer lateral seismic loads to the frame without transferring vertical loads, and other details that are intended to transfer vertical loads while allowing rotation between the frame and the floor slab. Research is currently underway to verify the performance of these details. The implementation of CRSBFs in recent years, particularly in New Zealand, suggest that both research and practice will continue to support the ongoing advancement of this steel innovation.

Acknowledgments

The perspectives presented here were shaped through conversations with many New Zealand practicing engineers and academics, first during a visit to New Zealand in 2013 that was funded by the Canadian Seismic Research Network and facilitated by SCNZ (particularly Alistair Fussell and Kevin Cowie), and then during the initial drafting of the Design Guide while the author was in residence with Aurecon (supported by Geoff Sidwell, Sean Gledhill, and John Finnegan) and engaged in discussions with Professors Greg MacRae and Charles Clifton. Figures in this paper were developed with the assistance of Taylor Steele and Changxuan Zhang, both graduate students at McMaster University.

References

Christopoulos, C., Filiatrault, A., and B. Folz, 2002. Seismic response of self-centring hysteretic SDOF systems. *Earthquake Engineering and Structural Dynamics*, 31(5), 1131-1150.

Eatherton, M. R., and J. F. Hajjar, 2010. Large-scale cyclic and hybrid simulation testing and development of a controlled-rocking steel building system with replaceable fuses. *Report No. NSEL-025*, Newmark Structural Engineering Laboratory Report Series, University of Illinois at Urbana-Champaign, Urbana, USA.

Eatherton, M. R., Ma, X., Krawinkler, H., Deierlein, G. G., and J. F. Hajjar, 2014a. Quasi-static behavior of controlled rocking steel frames. *Journal of Structural Engineering*, 140(11), 04014083.

Eatherton, M. R., Ma, X., Krawinkler, H., Mar, D., Billington, S., Hajjar, J.F., and G. G. Deierlein, 2014b. Design concepts for controlled rocking of self-centering steel-braced frames. *Journal of Structural Engineering*, 140(11), 04014082.

Gledhill, S. M., Sidwell, G. K., and D. K. Bell, 2008. The damage avoidance design of tall steel frame buildings – Fairlie Terrace student accommodation project, Victoria University of Wellington. *Proc. 2008 NZSEE Conference,* Wairakei, NZ.

Henry, R. S., Ingham, J. M., and S. Sritharan, 2012. Wall-to-floor interaction in concrete buildings with rocking wall systems. *Proc. 2012 NZSEE Conference,* Christchurch, NZ.

Latham, D. A., Reay, A. M., and S. Pampanin, 2013. Kilmore Street Medical Centre: Application of a post-tensioned steel rocking system. *Proc. Steel Innovations Conference 2013,* Christchurch, NZ.

Ma, X., Krawinkler, H., and G.G. Deierlein, 2011. Seismic design and behavior of self-centering braced frame with controlled rocking and energy dissipating fuses. *Report No. 174*, The John A. Blume Earthquake Engineering Center, Stanford University, Stanford, USA.

Mar, D, 2010. Design examples using mode shaping spines for frame and wall buildings. *Proc. 9th US and 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada.

Midorikawa, M., Azuhata, T., Ishihara, T., and A. Wada, 2006. Shaking table tests on seismic response of steel braced frames with column uplift. *Earthquake Engineering and Structural Dynamics*, 35(14), 1767-1785.

Pollino, M. and M. Bruneau, 2008. Dynamic seismic response of controlled rocking bridge steel-truss piers. *Engineering Structures*, 30(6), 1667-1676.

Priestley, M. J. N., Sritharan, S., Conley, J. R., and S. Pampanin, 1999. Preliminary results and conclusions from the PRESSS five-story precast concrete test building. *PCI Journal*, 44(6), 42-67.

Priestley, M. J. N., Calvi, G. M., and M. J. Kowalsky, 2007. *Displacement-based seismic design of structures*. IUSS Press, Pavia, Italy.

Roke, D., Sause, R., Ricles, J.M., and N. Gonner, 2009. Damage-free seismic resistant self-centering steel concentrically-braced frames. *Proc. STESSA 2009*, Philadelphia, USA.

Roke, D., Sause, R., Ricles, J.M., and N. B. Chancellor, 2010. Damage-free seismic-resistant self-centering concentrically-braced frames. *ATLSS Report No. 10-09*, Lehigh University, Bethlehem, USA.

Seo, C.-Y., and R. Sause, 2005. Ductility demands on self-centering systems under earthquake loading. *ACI Structural Journal*, 102(2), 275-285.

Steel Construction New Zealand (SCNZ), 2015. *Design guide for controlled rocking steel braced frames.* Draft dated 22 April 2015, publication pending by SCNZ, Manukau City, NZ.

Steele, T., and L. Wiebe, 2014. A practical approach to capacity design of controlled rocking steel braced frames. *Proc. CSCE 2014 4th International Structural Specialty Conference,* Halifax, Canada.

Steele, T., and L. Wiebe, 2015. Influence of member inelasticity on the performance of controlled rocking steel braced frames. *Proc. STESSA 2015,* Shanghai, China.

Tait, J. D., Sidwell, G. K., and J. F. Finnegan, 2013. Case study – Elevate Apartments – A rocking 15 storey apartment building. *Proc. Steel Innovations Conference 2013,* Christchurch, NZ.

Tremblay, R., Poirier, L.-P., Bouaanani, N., Leclerc, M., Fronteddu, L., and S. Rivest, 2008. Seismic response of viscously damped controlled rocking braced steel frames for building structures. *Proc. 14th World Conference on Earthquake Engineering*, Beijing, China.

Wiebe, L., Christopoulos, C., Tremblay, R., and M. Leclerc, 2012. Modelling inherent damping for rocking systems: results of large-scale shake table testing. *Proc. 15th World Conference on Earthquake Engineering*, Lisbon, Portugal.

Wiebe, L. D. A., 2013. Design of controlled rocking steel frames to limit higher mode effects. *PhD Thesis*, University of Toronto, Canada.

Wiebe, L., Christopoulos, C., Tremblay, R., and M. Leclerc, 2013a. Mechanisms to limit higher mode effects in a controlled rocking steel frame. 1: Concept, modelling, and low-amplitude shake table testing. *Earthquake Engineering and Structural Dynamics*, 42(7), 1053-1068.

Wiebe, L., Christopoulos, C., Tremblay, R., and M. Leclerc, 2013b. Mechanisms to limit higher mode effects in a controlled rocking steel frame. 2: Large-amplitude shake table testing. *Earthquake Engineering and Structural Dynamics*, 42(7), 1069-1086.

Wiebe L., and C. Christopoulos, 2014a. A cantilever beam analogy for quantifying higher mode effects in multistorey buildings. *Earthquake Engineering and Structural Dynamics*, available online ahead of print at DOI 10.1002/eqe.2549.

Wiebe, L., and C. Christopoulos, 2014b. Performance-based seismic design of controlled rocking steel braced frames. I: Methodological framework and design of base rocking joint. *Journal of Structural Engineering*, 10.1061/ (ASCE)ST.1943-541X.0001202, 04014226.

Wiebe, L., and C. Christopoulos, 2014c. Performance-based seismic design of controlled rocking steel braced frames. II: Design of capacity-protected elements. *Journal of Structural Engineering*, 10.1061/(ASCE)ST.1943-541X.0001201, 04014227.

Zhang, C., and L. Wiebe, 2015. Parametric study of displacements in self-centering single-degree-of-freedom systems. *Proc. 11th Canadian Conference on Earthquake Engineering,* Victoria, Canada.