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DESIGN AND ANALYSIS OF CONTROLLED ROCKING STEEL BRACED FRAMES FOR SEISMIC HAZARD IN CANADA

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Abstract: Controlled rocking steel braced frames (CRSBFs) have been proposed as a low-damage seismic force resisting system with self-centering capabilities. The force-limiting mechanism is rocking at the base, where energy dissipation may be provided to limit peak displacements, and post-tensioning provides self-centering, a positive post-uplift stiffness, and displacement capacity after rocking. All of the frame members are therefore capacity-protected elements, and are ideally designed to remain elastic. The performance of these systems has been evaluated extensively for regions of high seismicity in the western United States, but little insight is available on the benefits of constructing buildings in the high-risk areas of Canada using CRSBFs. This paper presents the design of three example CRSBFs for six-storey structures located in Vancouver, Ottawa, and Montreal. The energy dissipation and post-tensioning components are designed to resist the overturning moment calculated using the equivalent lateral-force procedure. A simple dynamic procedure is used to estimate the contribution of the higher modes to the capacity-design forces for the frame members. The frames are designed for lower lateral forces than traditional lateral force resisting systems, so little post-tensioning and energy dissipation is required. The nonlinear time history analyses using ground motions selected to match the uniform hazard spectra show that the frame members in the CRSBFs remain damage-free during the ground motions, and that the median interstorey drifts are much less than 2.5%. Procedures for estimating the peak interstorey drifts and floor accelerations are proposed and shown to be accurate compared to the analysis results.

1 INTRODUCTION

To mitigate the economic risks associated with earthquake damage to buildings in these areas, controlled rocking steel braced frames (CRSBFs) have been developed as a low-damage seismic force resisting system. CRSBFs mitigate structural damage during earthquakes through a controlled rocking mechanism, where energy dissipation can be provided at the base of the frame, and pre-stressed tendons pull the frame back to its centred position after rocking. The result is a flag-shaped hysteresis as shown in Figure 1, for which the residual drifts of the system after an earthquake are zero, and the energy dissipation does not result from structural damage.

CRSBFs have been studied extensively for areas of high seismicity in the Western United States, with researchers developing design procedures for the base rocking joint (Roke et al. 2010, Eatherton and Hajjar 2010, Ma et al. 2011, Wiebe and Christopoulos 2014a) and the capacity-protected frame members (Roke et al. 2010, Eatherton and Hajjar 2010, Ma et al. 2011, Wiebe and Christopoulos 2014b, Steele and Wiebe 2016). However, little insight is available on the design and analysis of buildings using CRSBFs in high-risk

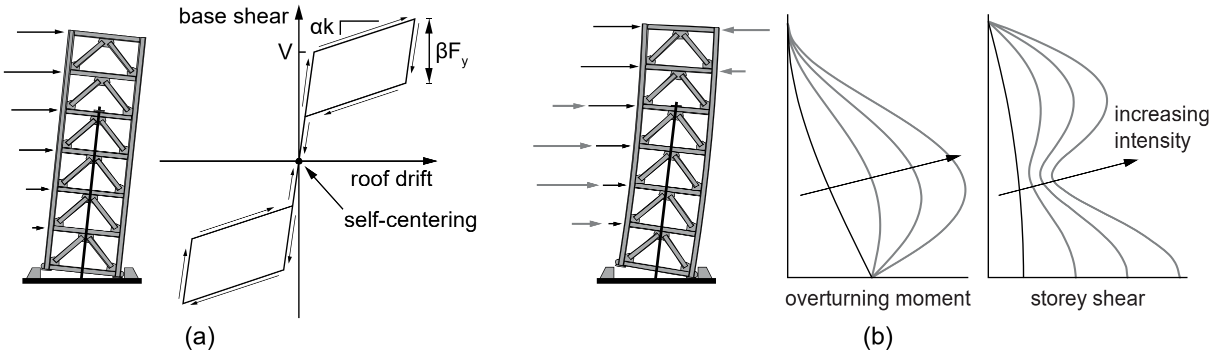

Figure 1: Characteristic design-level behaviour of controlled rocking steel braced frames (a) self-centering behaviour, (b) development of higher-mode forces

regions of Canada, and whether or not these procedures will be adequate when designing for Eastern or Western Canada seismicity.

This paper presents the design and analysis of three CRSBFs as the primary lateral force resisting system for steel framed buildings in Canada. The CRSBFs are designed for six-storey buildings in located in Ottawa, Montreal, and Vancouver. The base rocking joints are designed to resist the overturning moments that develop under the seismic loads that are calculated in accordance with the National Building Code of Canada (NBCC) (NRC 2015). The frame members are capacity-designed to remain elastic up to twice the design level using the dynamic procedure developed by Steele and Wiebe (2016). The frames are modelled and subjected to 30 ground motions selected and scaled to represent the seismic hazard for each site at the first-mode period of the structures. The peak interstorey drifts, residual drifts, and peak floor accelerations are evaluated for each frame design, and procedures for estimating the peak interstorey drifts and peak floor accelerations are presented and evaluated.

2 DESIGN OF EXAMPLE CONTROLLED ROCKING STEEL BRACED FRAMES

Example prototype structures were designed for Class D (V_{30} = 250 m/s) sites in Ottawa, Montreal, and Vancouver. Figure 2(a) shows the NBCC 2% in 50 year elastic design spectra for the three sites, and Figure 2(b) shows the floor plan for the example prototype buildings which had a 5 bay by 5 bay floor plan, with 9 m wide bays. The first storey height was 4.5 m, and the remaining 5 stories were all 3.75 m in height. The seismic weight of each floor and the roof were 9110 kN and 6840 kN, respectively. Each of the buildings was designed with four frames in each direction.

2.1 Design of the base rocking joint for NBCC specified seismic forces

The base shear for each frame was calculated in accordance with the NBCC 2015 (NRC 2015) using Equation [1]:

[1] $$V = S(T_a)M_vI_EW / R_dR_o$$

divided by the number of frames in each direction, where M_v is an adjustment factor for the higher modes, I_E is the importance factor for earthquakes, W is the seismic weight of the building, and the product R_dR_o is the seismic force reduction factor related to ductility and overstrength. The factors I_E and M_v were both taken to be 1.0 for all three buildings. The value of R_dR_o was taken as 8 for all three buildings; although it has been suggested that larger force reduction factors can be used to design the base rocking joint for CRSBFs, the NBCC currently does not permit an R_dR_o value as large as 8 for even the most ductile systems. The largest value of R_dR_o permitted in the NBCC is 6.8 for ductile moment resisting frames, but it has been shown that using an equivalent value of 12 was safe against collapse when designing CRSBFs in accordance with ASCE 7 (Steele and Wiebe 2017). It is not preferred to specify values of R_d and R_o individually for CRSBFs because both values depend heavily on the base rocking joint design parameters

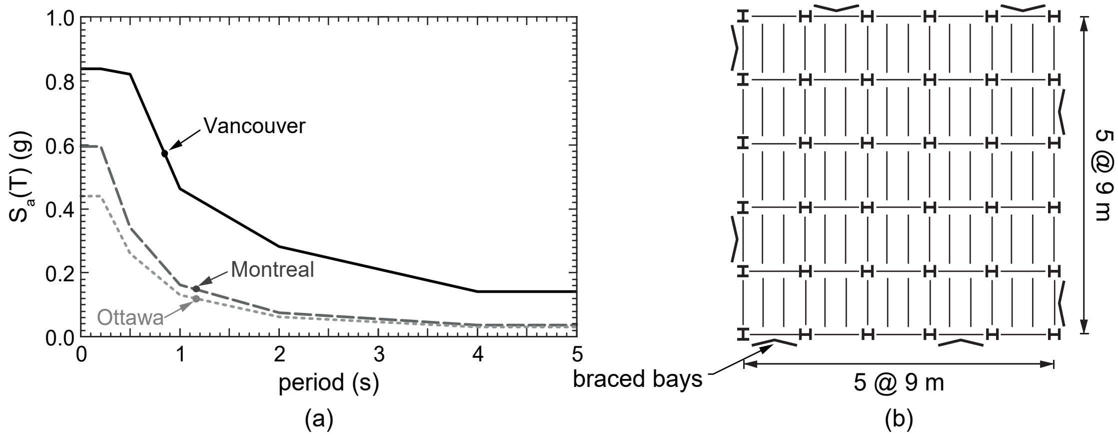

Figure 2: (a) NBCC 2015 elastic design spectra and (b) prototype building floor plan.

that are selected by the designers rather than the material properties of steel. For example, R_o can be calculated as the ratio of the maximum base overturning moment to the base overturning moment used to design the energy dissipation and post-tensioning:

[2] $R_o = M_{b,max}/M_b$

The ductility factor, R_d, is also not dictated by material properties for CRSBFs, as long as the post-tensioning is designed not to yield at the design level displacements. Therefore, the product of R_dR_o is simply specified such that the probability of collapse and design-level displacements are within reasonable limits. The effective value of R_d can be calculated back from the specified value of R_dR_o and the calculated value of R_o. The fundamental period was initially estimated as recommended in the NBCC 2015 as T_a = 0.025h (where h is in metres), and the period of the structural model was used to iteratively refine the base rocking joint design. However, the codified upper limit for T_a (i.e. 0.05h) was used if the modelled period exceeded this value. The equivalent lateral forces were distributed along the height of the buildings, and the resulting base overturning moment was computed. The recommendations made by Wiebe and Christopoulos (2014a) and Steele and Wiebe (2016) were used to proportion the energy dissipation and post-tensioning for the given base overturning moment. Table 1 shows the parameters used to design the base rocking

Table 1: Base rocking joint design parameters

	Ottawa	**Montreal**	**Vancouver**
Design fundamental period, $\mathbf{T_a}$	1.16 s	1.16 s	0.84 s
Modelled first-mode period, $\mathbf{T_1}$	1.19 s	1.17 s	0.84 s
Force reduction factor, $\mathbf{R_dR_o}$	8	8	8
Design base shear, **V**	193 kN	241 kN	948 kN
Base overturning moment, $\mathbf{M_b}$	3250 kN-m	4070 kN-m	15 635 kN-m
Energy dissipation ratio, $\boldsymbol{\beta}$	0.50	0.80	0.85
Energy dissipation activation load, **ED**	100 kN	200 kN	830 kN
Post-tensioning prestress ratio, $\boldsymbol{\eta}$	0.35	0.35	0.35
Number of post-tensioning strands, $\mathbf{N_s}$	7	7	23
Initial post-tensioning force, **PT**	638 kN	638 kN	2096 kN
Equivalent spring stiffness, $\mathbf{k_{PT}}$	12 130 kN/m	12 130 kN/m	39 867 kN/m
Ultimate post-tensioning force, $\mathbf{PT_u}$	1823 kN	1823 kN	5989 kN
Maximum base overturning moment, $\mathbf{M_{b,max}}$	8254 kN-m	9095 kN-m	31 376 kN-m
Overstrength factor, $\mathbf{R_o}$	2.54	2.24	2.01

joint. The energy dissipation ratios (defined in Figure 1(a) as the ratio of the height of the flag to the linear limit, V) were selected to be 0.85 for Vancouver, 0.80 for Montreal, and 0.5 for Ottawa, which resulted in energy dissipation activation loads of 830 kN, 200 kN, and 100 kN, respectively. Choosing energy dissipation ratios that are close to but less than 1.0 provides higher hysteretic energy dissipation while ensuring that the frame will be fully self-centering. An energy dissipation ratio of 0.85 could also have been used for the frames in Ottawa and Montreal, but this would have resulted in too few post-tensioning strands to provide a positive post-uplift stiffness. High-strength post-tensioning strands with a diameter of 15 mm and a cross-sectional area of 140 mm^2 were specified, with a yield stress of 1680 MPa, ultimate stress of 1860 MPa, and an elastic modulus of 195 GPa (DSI 2006). The post-tensioning was specified to be anchored at the top of the fourth storey in the middle of each frame. To prevent post-tensioning yielding at interstorey drifts up to the design-level limit of 2.5%, a post-tensioning prestress ratio, η (defined as the ratio of the prestress to the ultimate stress), of 0.35 was selected. This resulted in 23 post-tensioning strands for the frame in Vancouver, and 7 strands for the frames in Ottawa and Montreal. The post-uplift stiffness was checked to be positive using the recommendations in Steele and Wiebe (2016); a positive post-uplift stiffness is recommended to control the peak displacements from rocking (Wiebe and Christopoulos 2014a). For all cases, the seismic loads governed the design of the base rocking joint.

2.2 Design of the capacity-protected frame members

Because the force-limiting mechanism is rocking only at the base, the frame is free to experience the full forces that develop under the higher modes of vibration. This means that for the frame members in CRSBFs to remain elastic, they should be designed to carry the maximum expected forces from both the maximum first-mode response and the forces from the higher modes expected at the design level. Several researchers have proposed methods to calculate the capacity design forces for CRSBFs to account for the additional forces from the higher modes (Roke et al. 2010, Eatherton and Hajjar 2010, Ma et al. 2011, Wiebe and Christopoulos 2014b, Steele and Wiebe 2016). The dynamic procedure proposed by Steele and Wiebe (2016) was shown to be a practical and accurate method that can be readily implemented in commonly used analysis software; the method uses a simple elastic model of the frame with boundary conditions that reflect the expected conditions when the frame rocks. In the dynamic procedure for capacity design, the higher mode forces are accounted for using a modified response spectrum analysis; the first mode is considered separately from the response spectrum analysis by explicitly using the peak energy dissipation and posttensioning forces and their corresponding overstrength lateral forces. The equivalent lateral forces that were used to calculated the base overturning moment were amplified by R_o, and applied to the structural model at the frame centre nodes, where the gravity system would connect to the CRSBF. The initial post-tensioning force and the maximum energy dissipation forces were applied at their respective locations, and the additional post-tensioning force up to the maximum force, $PT_{u,}$ developed in the equivalent linear springs of stiffness k_{PT} as the frame rotates about the pin at the base.

The spectral intensity used to calculate the higher-mode forces was twice the NBCC elastic design spectrum for each of the three frames to account for the variability of the spectral intensity around the mean. The first six lateral modes were considered. The frame members were all designed as axial members in accordance with CSA S16-14 (CSA 2014a). The members were generally governed by global flexural buckling limit states, but the horizontal members at the third storey act as tension ties to resist the force imposed on them from the post-tensioning and were therefore governed by tensile limit states.

3 NUMERICAL MODELLING OF THE EXAMPLE FRAMES

Figure 4 shows the numerical model of the frames developed in the earthquake simulation program OpenSees (PEER 2016). Rocking at the base of the frame was modelled using compression-only gap elements in the vertical and horizontal directions at the base both frame columns. Because the centre-line dimensions were used to model the frame, the stiffness of the gap elements was defined to be representative of the stub of column between the base and the centreline of the ground-level beam. The frame members were all modelled using multiple force-based beam-column elements with fiber sections to capture inelastic buckling when the member capacities were exceeded. The fiber sections included a linear

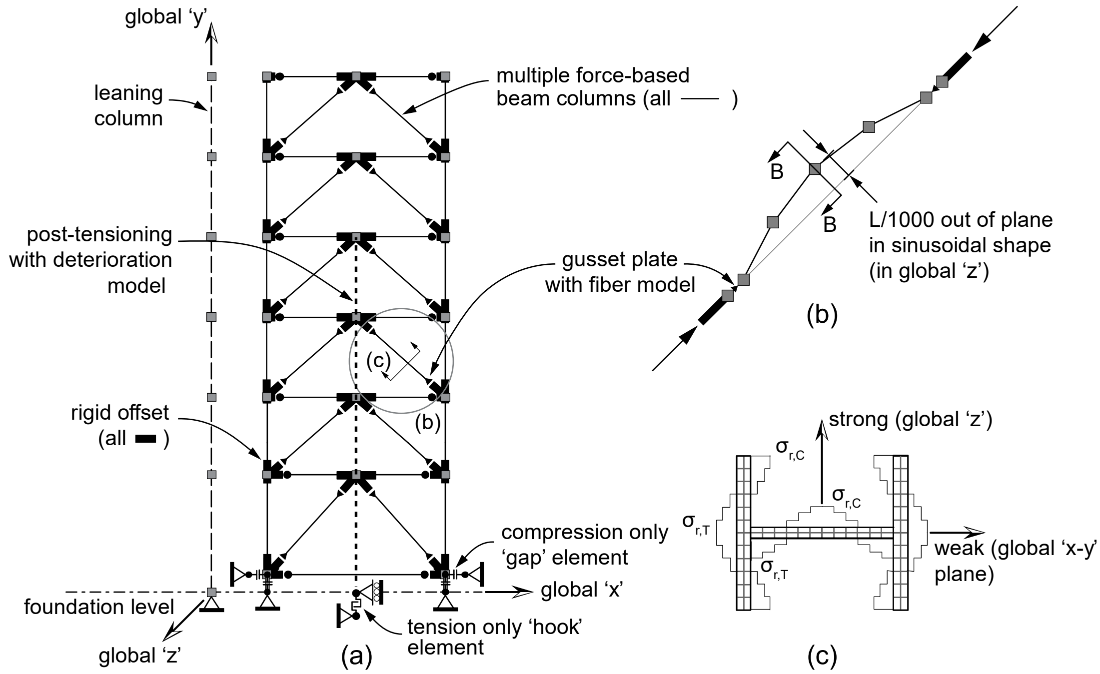

Figure 4: OpenSees model for six-storey frames (a) global frame model, (b) member buckling model, (c) fiber section with residual stresses model.

residual stress model to capture buckling at an appropriate value. Rigid offsets were defined using the frame geometry and included as elastic beam-column elements. The post-tensioning was modelled using a corotational truss element with an initial stress and multi-linear material model to capture the yield point, initial wire fracture, and gradual wire fracture of the high-strength steel strands as observed in laboratory tests (Ma et al. 2011). A tension-only hook element was included at the base of the post-tensioning to capture elastic buckling of the strands at negligible compressive loads. The frictional energy dissipation interfaces were modelled using truss elements and an elastic-perfectly plastic material model with yield force equal to the specified activation force. Other sources of energy dissipation not explicitly modelled were included using mass-proportional and stiffness-proportional Rayleigh damping; the Rayleigh damping coefficients were calculated assuming a damping ratio of 5%, using the second-mode frequency and the frequency corresponding to the secant stiffness of the frame while rocking. The secant stiffness was calculated as the initial stiffness of the frame reduced by R_dR_o, under the assumption that the displacements would be the same as those in the equivalent elastic system; using these frequencies to calculate the Rayleigh damping coefficients resulted in a damping ratio of roughly 3% in the elastic first mode. The seismic weight and P-Delta effects from the gravity system were modelled using a leaning column with an axial stiffness representative of the gravity columns tributary to the frame, and a negligible flexural stiffness.

4 GROUND MOTION SELECTION AND SCALING

To evaluate the performance of the CRSBFs in this study during expected design-level events, the synthetic ground motions developed for Eastern and Western Canada by Atkinson (2009) were used for the nonlinear time history analyses. For the buildings in Ottawa and Montreal, 30 ground motions were selected to represent earthquakes in Eastern Canada at a site of Class D with a magnitude of 7.0 and a distance of 15-100 km away from the source. The ground motions shown in Figure 5 for Eastern Canada were collectively scaled by a factor of 1.34 such that the median of the ground motions matched the design spectrum for the site in Montreal at the first-mode period of 1.16 s. For Ottawa, the same ground motions were scaled by a factor of 1.12 such that the median of the ground motions matched the design spectrum at the same 1.16 s period.

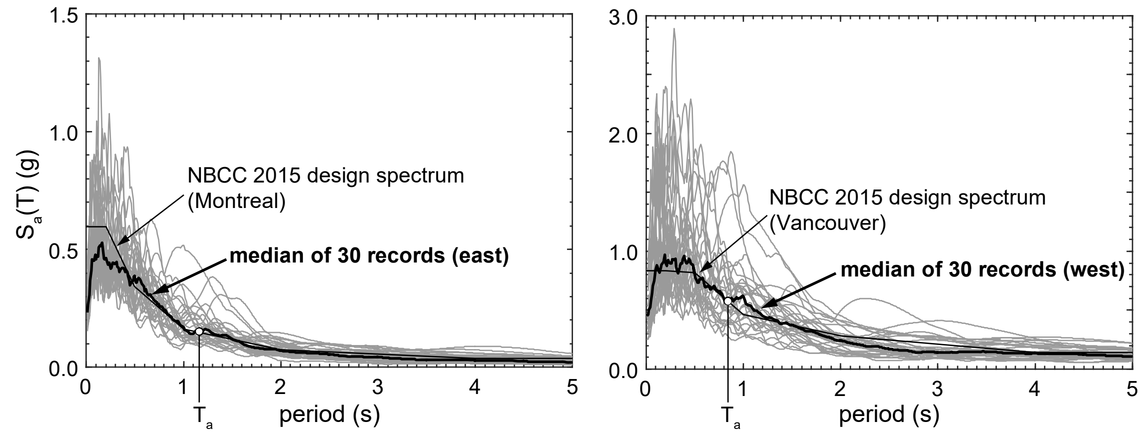

Figure 5: Ground motion response spectra for ground motion suites representing (a) seismicity in the east (Montreal), and (b) seismicity in the west (Vancouver).

For the building in Vancouver, 30 ground motions were selected to represent earthquakes in Western Canada at a site of Class D with a magnitude of 7.5 and a distance of 15-25 km away from the source. These ground motions were selected because the median spectrum matched the uniform hazard spectrum at the first-mode period of 0.84 s with a scaling factor near unity.

5 SEISMIC PERFORMANCE ASSESSMENT

The three frames were all assessed through nonlinear time history analysis based on the peak interstorey drift ratios, residual drift ratio, and peak floor accelerations, as discussed in the following subsections.

5.1 Interstorey drift ratios

Figure 6 shows the peak interstorey drifts for each of the three six-storey structures. The vertical uniformity of the peak interstorey drifts indicates both that the first-mode rocking response dominates the displacement response in CRSBFs, and that the frame members generally remained elastic during the ground motions. For the two frames that were designed and analysed for eastern-Canada seismicity, the peak interstorey drifts were well below the 2.5% limit; the peak interstorey drifts were very close to or less than 1% for all but one of the ground motions. This is because the spectral accelerations in the long-period range is low relative to the first-mode periods used to design the base rocking joint in the Eastern-Canada regions, and indicates that a larger product of R_dR_o could be used to design the base rocking joints for these regions. However, this would have to be verified through a collapse risk assessment before adopting such a value for code-compliant design.

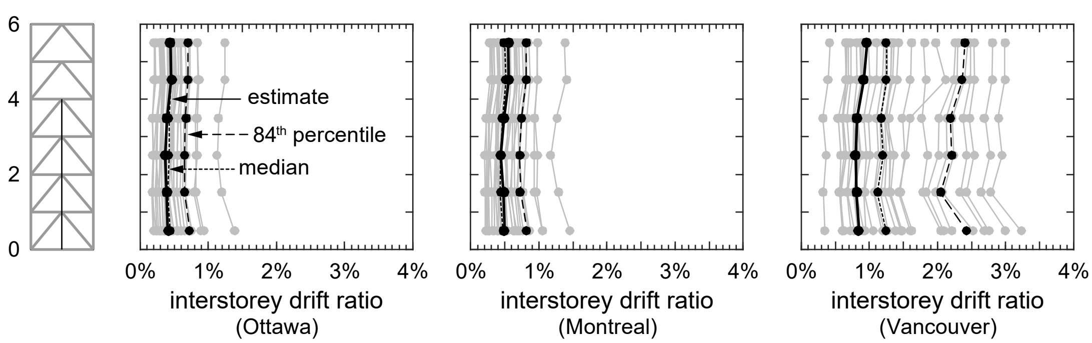

Figure 6: Peak interstorey drift ratios for the buildings from nonlinear time history analyses and estimates of the median from the dynamic procedure.

For the frame designed for the Western Canadian seismicity in Vancouver, the peak interstorey drifts were less than 2.5% on average, but the interstorey drifts were as large as 3.3% for the more extreme ground motions. The interstorey drifts were larger for the building in Vancouver than for those in Ottawa and Montreal, which was expected given the increased seismic intensity both at the first-mode period of the building and also in the long-period range of the spectrum. However, even the 84th percentile peak interstorey drift was less than 2.5%, and the median interstorey drifts were already considered acceptable.

Designers require a method to check that the peak interstorey drifts in buildings will be limited to less than 2.5% without completing any nonlinear analysis. For seismic design in general, it is often permissible to check the peak interstorey drifts as the elastic interstorey drifts multiplied by R_dR_o/I_E under the equal displacement assumption. For CRSBFs, the first-mode interstorey drifts are generally constant along the height of the building, and so the interstorey drifts can be estimated as $\Delta_{roof,y}/H \times R_dR_o/I_E$, where $\Delta_{roof,y}$ is the roof displacement at incipient rocking, and H is the building height. In this study, using this approximation underestimated the median peak interstorey drifts by up to 29% for the building in Ottawa, 23% for the building in Montreal, and 42% for the building in Vancouver. To reduce this error, the dynamic procedure was used to estimate the interstorey drifts including the higher-mode contribution. The interstorey drifts for each mode were calculated using the dynamic procedure, and the total response was calculated using Equation 3:

[3] $$ID_T = ID_1 + [ID_2^2 + ID_3^2 + \ldots + ID_N]^{1/2}$$

where ID_1 is calculated as $\Delta_{roof,y}/H \times R_dR_o$, and ID_2 through ID_N are the interstorey drifts at each storey level in modes 2 through N. This formulation was justified by applying the assumption that the rocking response will generally remain constant while the higher modes oscillate (Wiebe and Christopoulos, 2015). To evaluate the accuracy of this approach compared to the nonlinear time history analysis results, the interstorey drifts from the higher modes were estimated using the median response spectrum of the 30 ground motions that were selected for each site. Including the higher modes, this new approximation underestimated the interstorey drifts by no more than 13% for the building in Ottawa with a 2% difference in the peak value, and consistently overestimated the interstorey drifts by no more than 10% for the building in Montreal with a 4% difference in the peak value. However, the interstorey drifts were still underestimated for the building in Vancouver by up to 34%. Because the estimate includes the higher modes, this suggests that the equal displacement assumption is not an effective approximation for the nonlinear displacements in the first-mode response for the Vancouver site.

5.2 Residual drifts

Figure 7 shows the residual drifts after the 30 ground motions for each of the three six-storey structures. For the frames in Montreal and Ottawa, the residual drifts are practically zero for all of the records. This demonstrates that neither of the two frames rocked excessively during the records to yield the post-tensioning to an extent where the residual force in the post-tensioning would be unable to re-center the

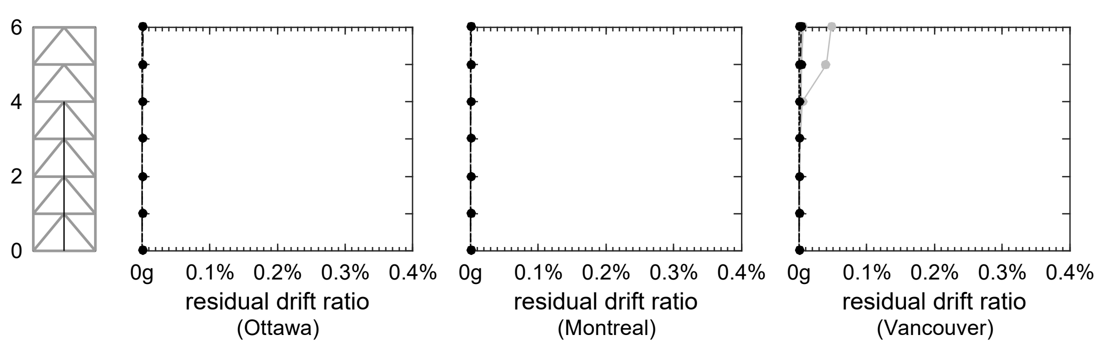

Figure 7: Residual drift ratios for the buildings from nonlinear time history analyses.

frame after rocking. This also indicates that the frame members remained elastic during all of the records, as intended.

The residual drifts were also generally zero for the frame in Vancouver, with the exception of a single record for which there was a residual drift of up to 0.05% in the top two stories. This residual drift was caused by buckling of the braces in the top two stories, but the braced did not buckle to below 90% of their initial compressive resistance, and the residual drift is still much less than 0.2% for this particularly extreme record. Thus, the structure was still within construction out-of-plumb tolerances according to CSA-S16 (CSA 2014a).

5.3 Floor accelerations

Figure 8 shows the peak floor accelerations for each of the three six-storey structures. From the shape of the peak floor accelerations, the second-mode response is seen to dominate the acceleration response. The peak floor accelerations were all less than 0.5g for the building in Ottawa, with median values no more than 0.22 g at the roof level. The peak floor accelerations were roughly 30% higher for the building in Montreal then for the building in Ottawa, which was expected, as the seismicity in Montreal is roughly 30% higher than that in Ottawa on average. The floor accelerations in Vancouver were much larger, with peak floor accelerations over 1.5 g during one extreme event, and as large as 0.67 g on the median. While there are no explicit limits on acceleration values according to the NBCC 2015, the peak floor accelerations must be known to design appropriate anchorage for operational and functional components in the buildings (CSA 2014b), and larger accelerations will require more robust anchorage.

To estimate the peak floor accelerations in buildings with controlled rocking systems, Wiebe and Christopoulos (2015) presented a set of closed-form equations for the first three lateral modes and the truncated rigid contribution of the higher modes; however, this formulation is limited to systems with uniform and continuous mass and stiffness, assuming the kinematic response is dominated by shear behaviour. This approach also does not require a numerical model of the frame, but as designers implementing the dynamic procedure proposed by Steele and Wiebe (2016) already have the simple elastic model and associated modal properties for the structure, a discrete formulation for floor accelerations may prove practical. However, modal response spectrum analysis is typically only capable of calculating relative floor accelerations in structures, which do not include the peak ground acceleration. The formulation for including the peak ground accelerations in the peak floor accelerations by modal response spectrum analysis proposed by Pozzi and Der Kiureghian (2012) was adopted and applied to the same elastic rocking model developed by Steele and Wiebe (2016) and used for the capacity design in this study. Applying the same assumption that the forces generally remain constant while the higher modes oscillate (Wiebe and Christopoulos, 2015), the accelerations were calculated using Equation 4:

[4] $$a_T = a_1 + [a_2^2 + a_3^2 + \ldots + a_N^2 + a_r^2]^{1/2}$$

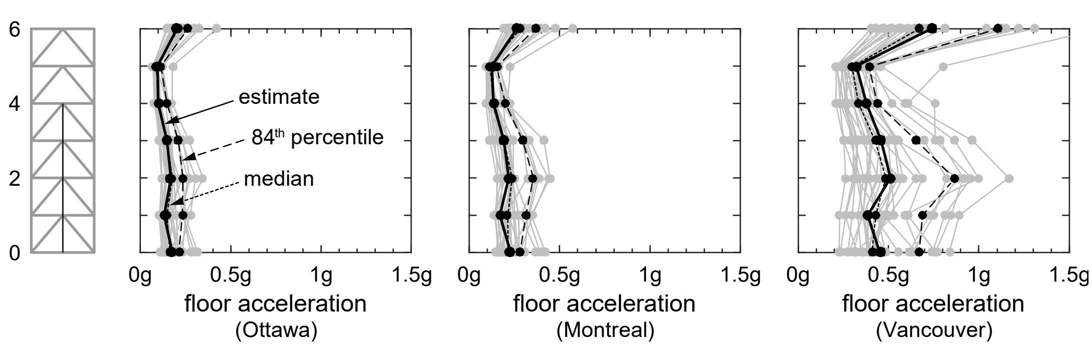

Figure 8: Peak floor accelerations for the buildings from nonlinear time history analyses and estimates of the median from the dynamic procedure.

where a_1 through a_N are the accelerations at each storey level in modes 1 through N, and a_r is the approximate residual acceleration from the truncated rigid contribution of the higher modes (Pozzi and Der Kiureghian 2012), calculated using Equation 5:

[5] $a_r = \iota - \Sigma\varphi_N\Gamma_N \times PGA$

where φ_N and Γ_N are the mode shape and participation factor, respectively, for mode N, and ι is the influence vector of ones in the horizontal degrees of freedom.

Again, to evaluate the accuracy of this approach compared to the nonlinear time history analysis results, the accelerations were estimated using the median response spectrum of the 30 ground for each site. The estimated floor accelerations are also shown in Figure 8. Both the shape and the values of the median floor accelerations were similar to the estimates, with the estimated values consistently within 10% of the median. The estimates had errors as small as 2% for the building in Ottawa, and the largest error was a 20% underestimate of the floor acceleration at the first storey of the building in Montreal. This approach is similar to using the set of equations proposed by Wiebe and Christopoulos (2015), but is more general in its ability to account for non-uniform mass and stiffness through a discrete numerical model.

6 CONCLUSIONS

This paper presents the design and assessment of controlled rocking steel braced frames as seismically resilient lateral-force resisting systems for buildings in Canada. The CRSBFs designed for buildings in Ottawa, Montreal, and Vancouver were all able to limit the median peak interstorey drifts to much less than 2.5%. The interstorey drifts were less than 2.5% for all of the ground motions in Ottawa and Montreal, indicating that larger values of R_dR_o could potentially be used to design CRSBFs in Eastern Canada, although this should be supported by a collapse risk assessment. For the frame in Vancouver, the 84th-percentile interstorey drifts were less than 2.5%, but were roughly twice as large as the interstorey drifts for the building in Montreal. Attempts to estimate the median interstorey drifts using the equal displacement assumption were not accurate for any of the three frames. The dynamic procedure previously proposed for capacity design of CRSBFs, which uses an equivalent elastic-rocking model to account for the dynamic properties of CRSBFs while they rock, was extended here to estimate the higher mode contribution to the interstorey drifts and provided accurate estimates of the interstorey drifts for the buildings in Ottawa and Montreal, but this approach underestimated the interstorey drifts for the building in Vancouver by as much as 32%.

The frame members all remained elastic during all of the ground motions for the buildings in Ottawa and Montreal, and the residual drifts were practically zero for all of the ground motions. This suggests that designing the frame members for the higher-mode forces at twice the design level is sufficient but perhaps too conservative for CRSBFs in these regions. One ground motion caused the braces in the upper storeys to buckle for the frame in Vancouver, but the residual drifts for the building were still less than 0.05%. The residual drifts were practically zero for the remaining ground motions.

The accelerations in the three buildings were generally dominated by the higher modes, and reached median values of 0.22 g, 0.27 g, and 0.67 g for the frames in Ottawa, Montreal, and Vancouver, respectively. To estimate the peak floor accelerations accurately without nonlinear time history analysis, the dynamic procedure was also extended to estimate median peak floor accelerations in the three buildings, and was shown to be generally conservative but accurate to within 10% on average.

This study was limited to three six-storey buildings in regions of moderate to high seismicity in Canada, all for Class D sites. The design approaches are intended to be general, and do not rely on any empirical calibration, but their application should be evaluated for buildings of different heights and different site classes.

Acknowledgements

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