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A SIMPLE SELF-CENTERING SHEAR FUSE FOR COST-EFFECTIVE CONTROLLED ROCKING STEEL BRACED FRAMES

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Abstract

The controlled rocking braced frame system has been developed over the last two decades to achieve damage-free and self-centering behaviour for superior seismic performance. The main feature of the system is the reduced overturning moment demand near the structure base and the resulting reduction in column axial loads in the bottom levels and the structure foundations. One drawback of the system, however, is the development of high brace force demands due to the inherent high storey shear stiffness of the system and higher mode response. Large axial loads can also develop in columns at intermediate levels due to the contribution from higher modes. These effects typically increase as the number of storeys is increased due to the growing contribution from the higher modes of vibration. Past numerical and shake table studies have shown that nonlinear bracing members placed in the first level can significantly diminish the storey shear force demand over the structure height. Overturning moments in the upper storeys were also reduced by the presence of such a base shear fuse mechanism. In thee studies, an advanced self-centering brace exhibiting a flag shape hysteretic response was used to create this shear fuse behaviour while preventing the development of residual storey shear displacements at the frame base level.

This article introduces a simpler shear fuse system that can be used to develop similar behaviour without using self-centering braces. In the proposed rocking steel braced frame system, nonlinear storey shear response is permitted to occur over the entire frame height with the objective of reducing and controlling member forces induced by higher modes of vibration. Nonlinear storey shear response is achieved by using a chevron bracing with one nonlinear brace at every level of the frame. The other brace and the beam are designed to remain elastic such that the system can exhibit a positive storey shear stiffness during nonlinear storey shear response up to a target storey shear value. This positive stiffness is expected to mitigate P-Δ effects on response and provide re-centering capacity to the structure. A method is presented to predict member forces to be used in the design of the nonlinear braces and other members of the rocking braced frame.

Response history analysis is performed on three prototype structures with gravity-controlled rocking braced frames to validate the proposed concept and design procedure. Two different designs were considered: braced frames with braces designed to remain elastic and braced frames with nonlinear braces. The results show that the frames with nonlinear braces experienced similar storey drifts when compared to the frames designed to remain fully elastic. The addition of the nonlinear braces was found to be effective in reducing and controlling member forces and floor horizontal accelerations. The results also demonstrated the re-centering capacity of the system.

Keywords: Braced steel frames; Rocking; Self-centering; Friction; Shear Fuse

1. Introduction

Steel controlled rocking braced frame (CRBF) systems have been proposed as a mean of achieving superior seismic performance under strong earthquakes. Contrary to conventional steel braced frames such as concentrically braced frames, buckling restrained braced frames or eccentrically braced frames, CRBFs do not rely on ductile inelastic behaviour to withstand earthquake effects. They are designed such that column uplift will occur during a strong earthquake, thereby limiting the overturning moment at the structure base, as is the case for cantilevered reinforced concrete shear walls. Members of the structure are however proportioned to remain essentially elastic to avoid structural damage and allow the structure to return to its original configuration after the earthquake. As shown in Fig. 1a, the system typically consists in steel braced frames with their columns free to uplift at their bases. In CRBFs vertically decoupled from the gravity frame, self-centering response is generally obtained by using vertical post-tensioned elements. The PT elements also provide elastic lateral stiffness upon rocking, which can mitigate P-delta effects. Gravity loads supported by the rocking frames also contribute to the re-centering capacity of the system. CRBFs part of the gravity framing that solely on gravity loads for self-centering are referred to as gravity-controlled rocking braced frames (G-CRBFs). Energy dissipation in rocking braced frames is typically achieved by means of dedicated ED devices placed at the bases of the uplifting columns. For CRBFs decoupled from the gravity framing, ED elements can also be placed between the rocking columns and the adjacent gravity framing columns.

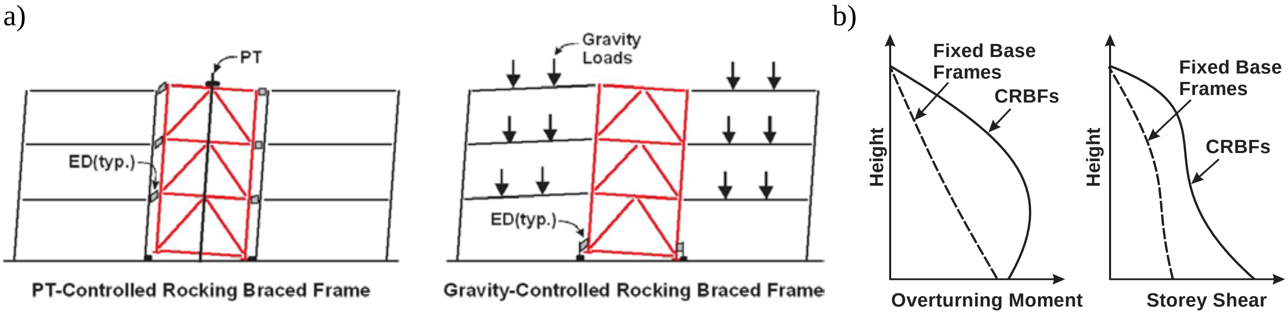

Fig. 1 – a) Rocking steel braced frames; b) Force demand in fixed base and rocking steel braced frames.

In the last two decades, the seismic response of steel rocking braced frames has been studied extensively both numerically and experimentally [1-10] and the system has already been implemented in practice [e.g. 11,12]. These investigations showed that the intended re-centering performance with no structural damage could be achieved in well-proportioned CRBFs. The investigations also show that although the base overturning moment is well controlled by allowing rocking at the structure base, large member forces can still develop in the structure due to higher mode response of the frame upon rocking. Typical force demands in bracing members and columns are schematically illustrated in Fig 1b. As shown, braces and columns obtained from this approach can be very large. In G-CRBFs, higher mode response during column uplift excursions induce additional member forces caused by the vertical masses supported by the frames. Design methodologies have been proposed to predict member forces by combining the forces from first mode response, as limited by the base rocking moment capacity, with the forces generated by higher modes in the column uplifted condition assuming elastic frame response [13,15]. In these methods, response spectrum analysis is generally used to evaluate higher mode effects and although realistic mean seismic force demand estimates can be obtained, the forces are highly variable as the frame elastic in higher modes is sensitive being essentially elastic for a steel frame is that response is unbonded and response from the predictions. Introducing rocking at intermediate levels along the structure height or using braces exhibiting nonlinear response at the structure base have been shown to successfully reduce forces in CRBFs [5,6]. However, as shown in Fig. 1b, the force reductions are localised, and the remaining parts of the frame must still sustain large force demands from higher mode response.

In this article, an alternative approach is examined to attenuate members forces in CRBFs and improve the cost-effectiveness of the system while preserving its inherent advantages. The approach consists in using bracing members exhibiting nonlinear behaviour at every level of the structure to achieve uniform force control. In each braced bay, an elastic brace is paired with a nonlinear brace in each storey to develop a positive storey shear stiffness upon brace nonlinear response and achieve re-centering capacity [16]. The nonlinear braces are designed for forces that include forces from higher mode response reduced by a force modification factor R_{hm}. The concept is applied to 4-, 8- and 12-storey prototype buildings laterally braced by G-RCBFs with brace resistances determining using R_{hm} = 1, 4, and 8. The structures are assumed to be located in Vancouver, British Columbia, Canada, a site exposed to long duration interface subduction earthquakes. The design procedure is first illustrated. Results from nonlinear response history analysis are then examined with focus on peak storey drifts, peak force demands in the braces and columns, and peak floor accelerations, and residual deformations.

2. Structures studied

2.1 Building description

The building studied is assumed to be located on a class C (firm ground) site in the region of Vancouver, British Columbia, Canada. The building plan view is illustrated in Fig. 2. Design gravity loads are also given in the figure. Three different heights are considered: 4-, 8-, and 12-storeys. In all three cases, lateral resistance for the structures is provided by one-bay braced frames located along the perimeter wall. The braced frames acting in the E-W direction are studied in this article. As shown in the figure, a chevron (inverted V) bracing is adopted for all braced frames.

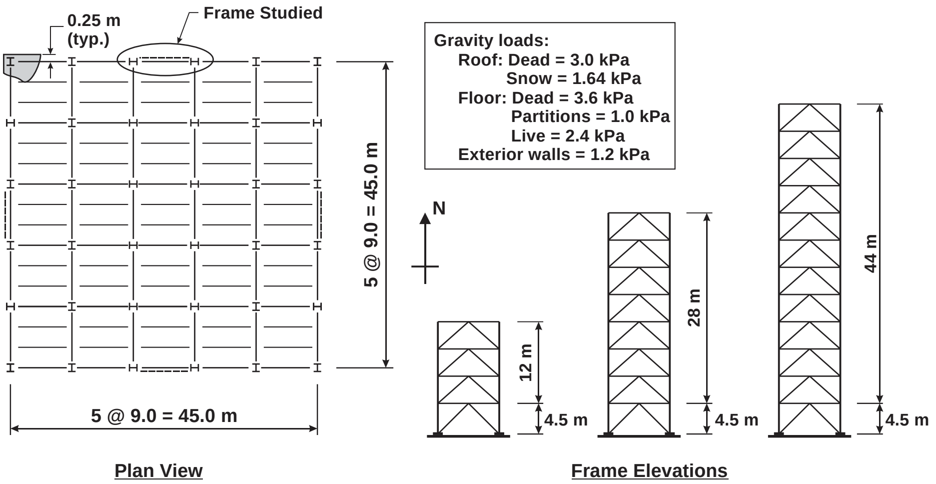

Fig. 2 – a) Structures studied; b) Proposed bracing configuration.

2.2 Design of the rocking braced frames

Design loads and load combinations were determined in accordance with NBC 2015 [17] and the design of the structures was performed following the requirements of the CSA S16-14 steel design standard [18]. The seismic load combination included 100% of the dead load (D), 50% of the floor live load (L) and 25% of the roof snow load (S). The seismic loads were determined from the combination of the first mode response in the fixed base condition and higher mode response in the rocking (uplifted) condition. First, the design base shear V_{NBC} was obtained from:

$$V_{NBC} = \frac{S(T_a)\, M_v\, I_E\, W}{R_d\, R_o} \qquad (1)$$

where S is the design spectrum, T_a is the design period, M_V is a factor that accounts for higher mode effects on base shear, I_E is the importance factor, and W is the seismic weight. For site class C in Vancouver, S is equal to 0.845 for $T \leq 0.2$ s followed by linear segments between S values of 0.845, 0.752, 0.424, 0.256, and 0.081 at periods of 0.2, 0.5, 1.0, 2.0, and 5.0 s, respectively. For braced frames, the period can be obtained from modal analysis, without exceeding a value equal to 0.05 h_n, where h_n is the structure height (in m). For the braced frames studied, the upper limit on T_a controlled, and the values used are given in Table 1. The factors M_V and I_E were both equal to 1.0. The seismic weight includes the dead load with reduced partition loads of 0.5 kPa plus 25% of the roof snow loads, and the resulting values are reported in Table 1. In the NBC, R_d and R_o are respectively the ductility- and overstrength-related force modification factors. G-CRBFs do not possess lateral overstrength and R_o = 1.0 was used. A value of 8.0 was tentatively selected for R_d. The computed values of V_{NBC} are given in Table 1. The building structures have a symmetrical plan arrangement and it was assumed that half of the total lateral load was resisted by each braced frame, thus omitting in-plane accidental eccentricity effects. In Table 1, the values of W and V_{NBC} are given for one braced frame.

Table 1 Properties of the structures studied (per frame)

Properties	**4-storey**	**8-storey**	**12-storey**
W (kN)	17818	36541	55264
T_a (s)	0.825	1.625	2.425
V (kN)	1200	1457	1597
R_D (kN)	848	1806	2767
R_E (kN)	1563	3257	4836
F_s (kN)	715	1451	2069
T_1 (s)	0.92	1.85	2.97
T_2 (T_{2u}) (s)	0.34 (0.34)	0.56 (0.58)	0.84 (0.89)
T_3 (T_{3u}) (s)	0.21 (0.21)	0.32 (0.33)	0.45 (0.47)
T_4 (T_{4u}) (s)	0.16 (0.16)	0.23 (0.23)	0.32 (0.32)

The computed periods in the first four vibrational modes of the structures with fixed base frames, T_1 to T_4, are given in Table 1. For each frame, member forces for design were determined from three responses spectrum analyses (RSA) using the code elastic design spectrum modified as described herein. Multi-mode response spectrum analysis (RSA) was first performed on the fixed base frame using the code design spectrum scaled such that the base shear from the analysis matched the code design base shear V_{NBC} in Table 1 (Fig. 3a). The vertical reaction R_E from this analysis was then used to determine the required frictional resistance for the ED devices at the column bases: $F_s = R_E - R_D$, where R_D is the vertical reaction due to dead loads. Values of R_E, R_D and F_s are given in Table 1. The second RSA was also performed on the fixed base frame considering only the structure first mode properties (Fig. 3b). This time, the design spectrum was adjusted such that the base vertical reaction from the RSA is equal to R_E from the previous multi-mode RSA. The structure model was then modified by introducing a flexible vertical spring at the base of one braced frame column to simulate the conditions upon rocking. The computed second, third and fourth mode periods in this uplifted condition, T_{2u}, T_{u3}, and T_{4u}, are given in Table 1. As shown in Fig. 3b, the design spectrum for this third RSA was truncated at a period slightly longer than T_{2u} to eliminate the contribution from the first mode in the uplifted position. The spectrum was also divided by the higher mode force modification factor, R_{hm}. To achieve elastic frame response, R_{hm} is set equal to 1.0, as typically done for the design of CRBFs. In this study, this third analysis was performed with R_{hm} = 1, 4, and 8, the last two cases being considered to achieve nonlinear response in the rocking frame.

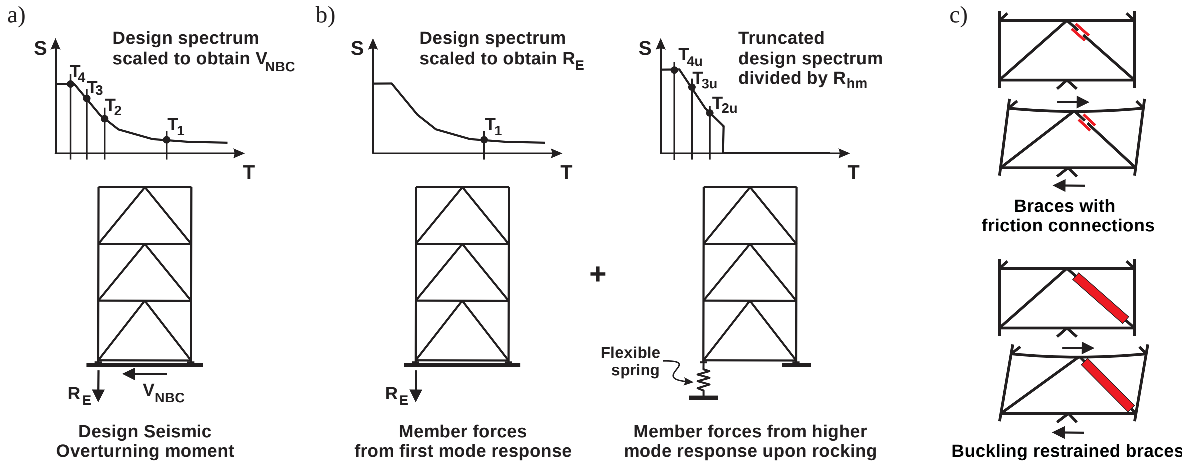

Fig. 3 – a) Structures studied; b) Proposed bracing configuration; c) Proposed nonlinear brace arrangement.

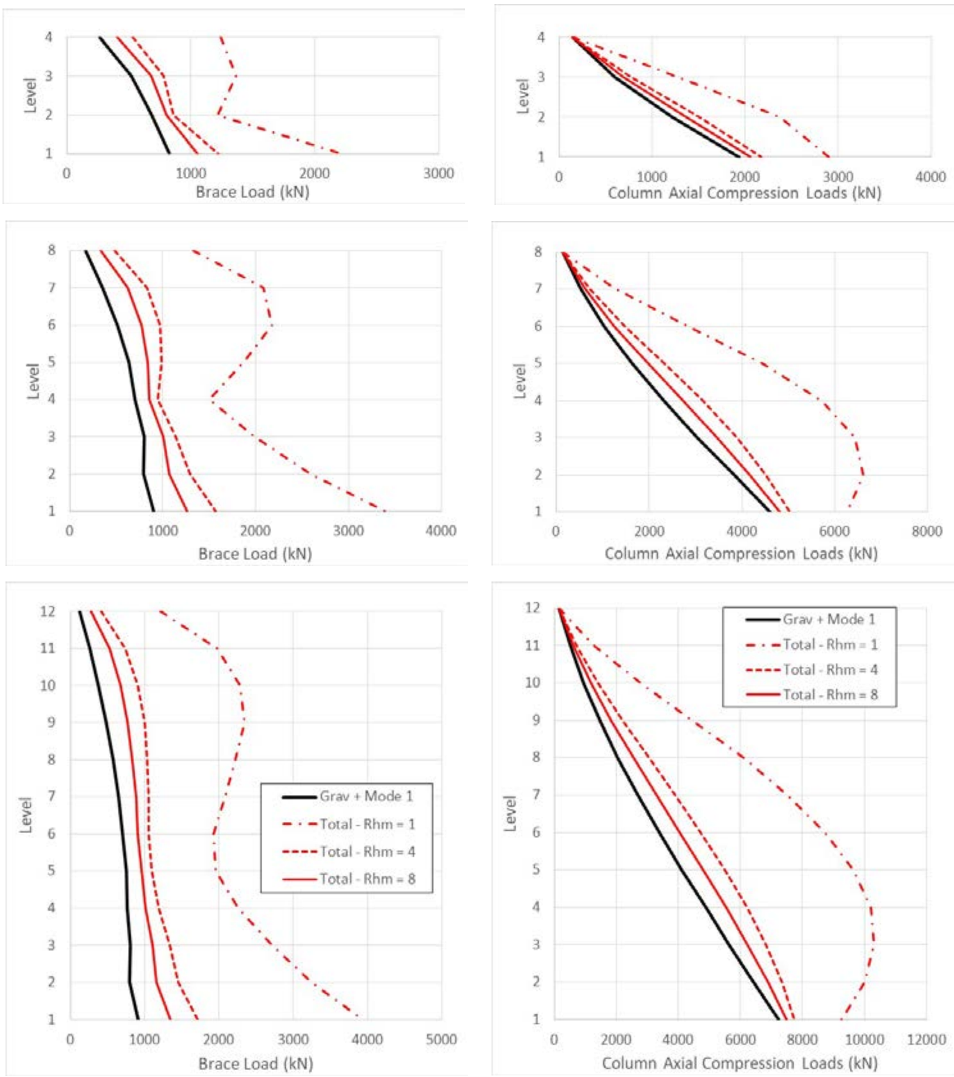

Fig. 4 – Design axial loads in braces and columns.

Member forces from gravity loads are added to those from the two RSA of Fig. 3b to obtain member design loads. Design loads for the braces, reflecting storey shears, and for the columns, reflecting overturning moments, are presented graphically in Fig. 4. As shown, significant reduction in design loads is obtained when using R_{hm} = 4. Adopting R_{hm} = 8 reduces further the design loads at mid-distance between the design loads from gravity loads plus first mode response and the values for R_{hm} = 4.0. The first design was performed to achieve elastic frame response using the design loads obtained with R_{hm} = 1.0 for all members. The braces were ASTM 1085 HSS members and ASTM A992 W shapes were used for the beams and columns.

For the other two designs with R_{hm} = 4 and 8, nonlinear response is intended to develop through storey shear deformations of the rocking frames so that storey shears could be reduced as intended in design. This could be achieved by using conventional bracing members or buckling restrained braces (BRBs) but this would result in structural damage and residual frame deformations after an earthquake, thereby compromising the advantages of using a rocking braced frame. Instead, it is proposed to use only one nonlinear brace per level, as illustrated in Fig. 3c. Braces with friction connections designed to slip at the reduced design loads, or BRB members designed to yield at the reduced design loads, would represent good choices for the nonlinear braces. In this study, friction connections were considered but a similar response would be expected with BRB members. Once slip is triggered in the brace connection, additional storey shear can be developed by the elastic brace acting in series with the beam in flexure. Hence, the beam and the elastic brace at every level can be designed to obtain a positive storey shear stiffness for mitigating P-Δ effects and providing a re-centring capacity such that residual storey shear deformations are minimized.

This behaviour is illustrated in Fig. 5. The storey shear stiffness upon sliding of the brace connection, k'_s, can be obtained from:

$$\Delta_{slip} < \Delta_s < \Delta_{sm}:\ \Delta'_s = V'\left(\frac{L}{2EA_g} + \frac{L}{EA_d\cos^3\theta} + \frac{L^3\tan^2\theta}{48EI_g}\right)\ ;\ k'_s = \frac{V'}{\Delta'_s} \tag{2}$$

where V' and Δ' are the additional storey shear and storey shear drift after initiation of slip. The maximum storey shear drift prior to yielding of the beam under combined flexure and axial load, Δ_m, is given by:

$$\Delta_m = \Delta_{slip} + \frac{V_m - V_{slip}}{k'_s} \tag{3}$$

where:
$$V_m = V_{slip} + \min\left[\frac{M_{pb} - M_{b,grav}}{\tan\theta(L/4)}\ ;\ \frac{P_{yb} - 0.5\,V_{slip} - 0.85\,M_{b,grav}\left(P_{yb}/M_{pb}\right)}{1 + 0.85\,\tan\theta\,(L/4)\left(P_{yb}/M_{pb}\right)}\right] \tag{4}$$

In these expressions, V_{slip} is the storey shear initiating slip in the brace connection, Δ_{slip} is the shear storey drift under V_{slip}, $M_{b,grav}$ is the beam moment due to gravity loads, $P_{y,b}$ and $M_{p,b}$ are the cross-sectional yield strength properties of the beam determined with probable yield strength properties. Other parameters are given in Fig. 5. Eq. 4 was derived from interaction equation for I-shaped member subjected to combined axial load and bending moment [18]. This concept was initially proposed in [16] to improve the stability of tall braced frames. The beam at every level is first selected to develop the minimum required stiffness k'_s and remain elastic until the target shear storey drift Δ_m is reached. Once the beam section is selected, the elastic brace is chosen to resist an axial compression (or tension) load $P_{e,d}$ developing at point B, as given by Eq. 5. Finally, columns are sized to resist axial loads due to gravity loads plus brace induced loads, including column loads P_c due to brace unbalanced forces also given in Eq. 5.

$$P_{e,d} = \frac{V_m - 0.5V_{slip}}{\cos\theta}\ ;\ P_c = \frac{V_m - V_{slip}}{2\,\tan\theta} \tag{5}$$

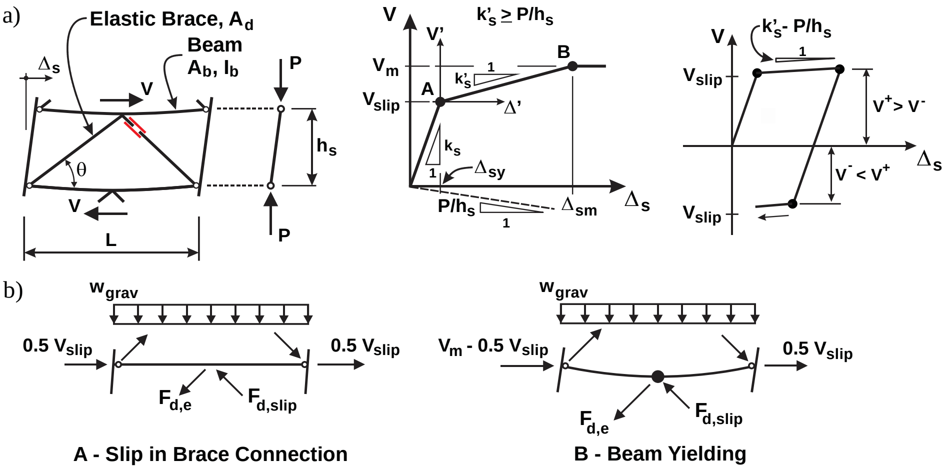

Fig. 5 – a) Proposed chevron bracing shear fuse with an elastic brace and post-yielding stiffness; b) Forces imposed to the beam at Points A and B.

In this study, the beams were designed to develop a stiffness k'_s equal to 1.5 $\Sigma P/h_s$, where ΣP and h_s are respectively the total gravity loads carried by the columns and the storey height at the level under consideration. This is sufficient to overcome P-Δ negative stiffness -$\Sigma P/h_s$ and provide re-centring capacity. The storey drift Δ_m was set equal to 1.2 % h_s. In actual G-CRBFs, the nonlinear braces would be designed for the reduced design loads of Fig. 4, the elastic braces would be designed for P_{ed} from Eq. 5 and the columns would be designed for the reduced loads in Fig. 3 plus the load P_c from Eq. 5. In this study, the braces and columns selected for the elastic design (R_{hm} = 1) were kept unchanged so that the shear and flexural stiffness of the frame remained essentially the same for the three designs. In this way, the influence of introducing nonlinear frame response could then be examined in isolation.

3. Response history analysis

Response history analysis of the structures was performed with the SAP2000 computer program [19] using two-dimensional models of the rocking frames. The G-CRBF members were modelled with elastic frame elements including P-M plastic hinges. At the column bases, nonlinear gap and plastic link elements were used at the interface of the rocking columns and foundations. Slippage in the brace friction connections was reproduced using axial hinges with elastic-plastic behaviour in the brace members. The model included all building columns supporting their gravity loads to properly accounted for P-Δ effects. Vertical masses were assigned to the rocking columns at every building level to include vertical inertia forces induced by the floor structure supported by the columns. Rayleigh damping based on initial stiffness and corresponding to 3% of critical was specified in modes 1 and 2 for the 4-storey building, and in modes 1 and 3 for the 8- and 12-storey. Stiffness-related damping was not assigned, however, to the nonlinear link elements. Prior to applying the seismic ground motions, the models were analysed under specified dead loads. The structures were then subjected to an ensemble of three suites of 5 ground motion records, each suite corresponding to one of the three sources of earthquakes contributing to the hazard in southwest British Columbia: shallow crustal earthquakes, in-slab subduction earthquakes and interface subduction earthquakes. The ground motions were selected and scaled in accordance with the guidelines provided in [20] using the NBC design spectrum as the target. P-Δ effects were included in the response history analyses.

The envelopes of peak storey drifts obtained from the 15 ground motions are presented in Fig. 6 for the three buildings and the designs performed with R_{hm} = 1, 4, and 8. In the figure the seismic design demand (darker line) corresponds to the mean of the largest five values obtained from the 15 ground motion records. As shown, the storey drifts for the elastic design (R_{hm} = 1) lie between 1.0 and 1.5% h_s over the building heights, which is satisfactory performance. When reducing design forces from higher mode response, the drifts remained essentially the same for the 4- and 8-storey buildings and reduced slightly for the 12-storey frames. The reduction is probably a consequence of the increased energy dissipation capacity brought by the nonlinear braces. However, for all structures, allowing nonlinear shear response by using R_{hm} = 4 and 8 resulted in a non-symmetrical storey drift response with positive values (drifts toward the RHS) being generally larger than the negative ones. This phenomenon is attributed to the gravity loads supported by the beams of the braced frames. This load favored downward bending of the beams upon slippage of the brace connections, compared to upward bending, which resulted in larger positive lateral displacements because of the presence of the elastic brace and the frame geometry. This undesired response could be mitigated in actual structures by having two braced frames with nonlinear direction in opposite directions along the same column line.

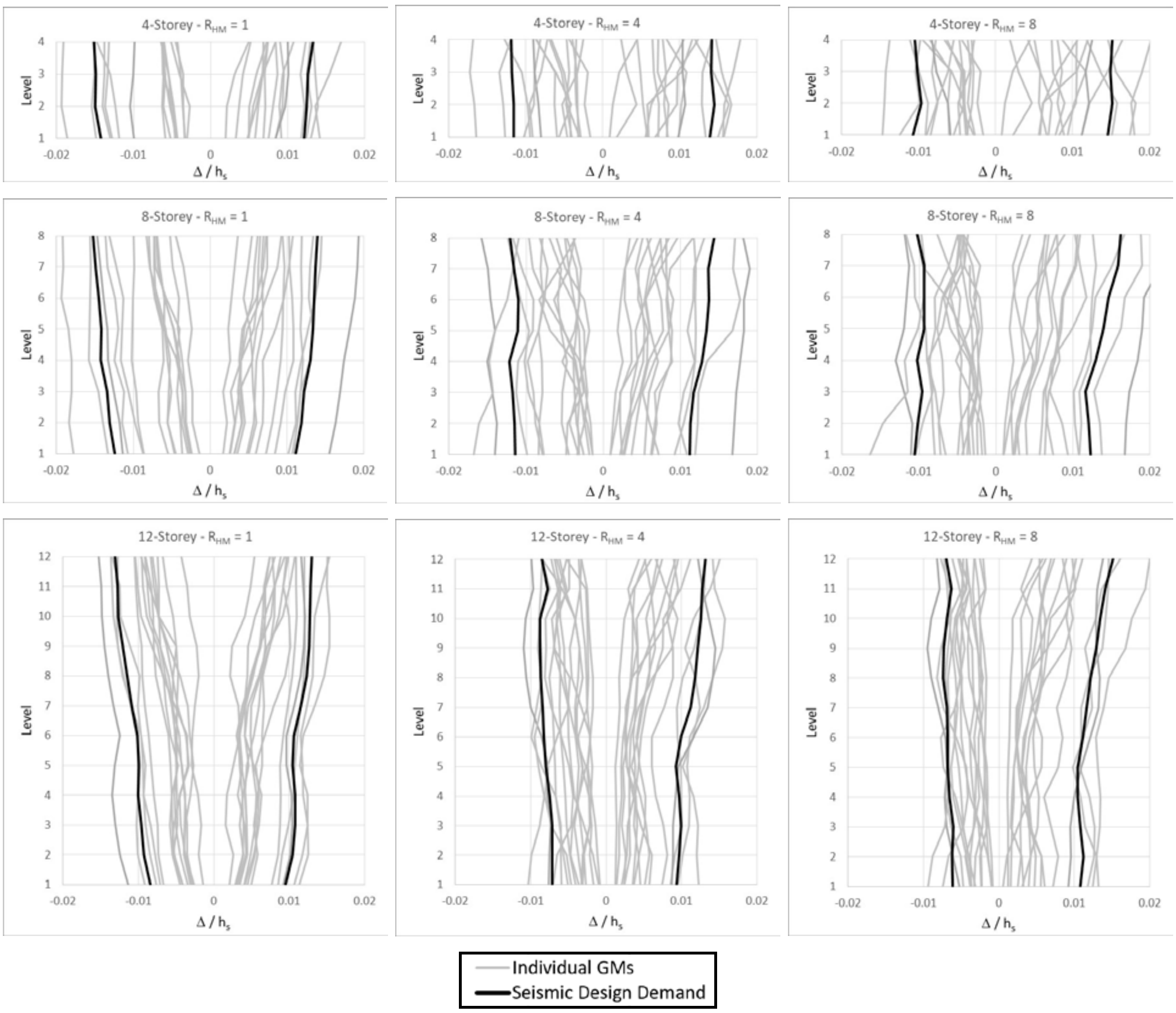

Fig. 6 – Computed peak storey drift demands.

Peak axial load demands in the braces and columns are presented in Fig. 7. The graphs show the peak forces in the LHS (elastic) braces as well as the LHS column, i.e. the column to which the elastic brace is connected. For simplicity and clarity, in this figure and the subsequent ones, only the mean value of the largest 5 results (seismic design demand) are presented for simplicity. For the braces, the axial loads obtained with R_{hm} = 1.0 exceed the values predicted from RSA in design, especially for the 8- and 12-storey buildings. This difference can be partly attributed to the fact that RSA was performed with an 5% damped acceleration spectrum and lower damping (3%) was considered in the analysis. Nevertheless, the differences confirm the difficulty to control member forces in rocking frames that remain fully elastic under ground shaking. The results also show that the use of nonlinear braces can effectively reduce the force demands in the both the braces and the columns, as intended in design. Peak horizontal floor accelerations are reported in Fig. 8. As expected, by controlling the force demands in the structure, floor accelerations also reduce during the ground motions. In addition, higher mode response present in the elastic structure is nearly entirely eliminated by the addition of the nonlinear braces. This effect can be very beneficial for structures that must satisfy strict floor acceleration limits.

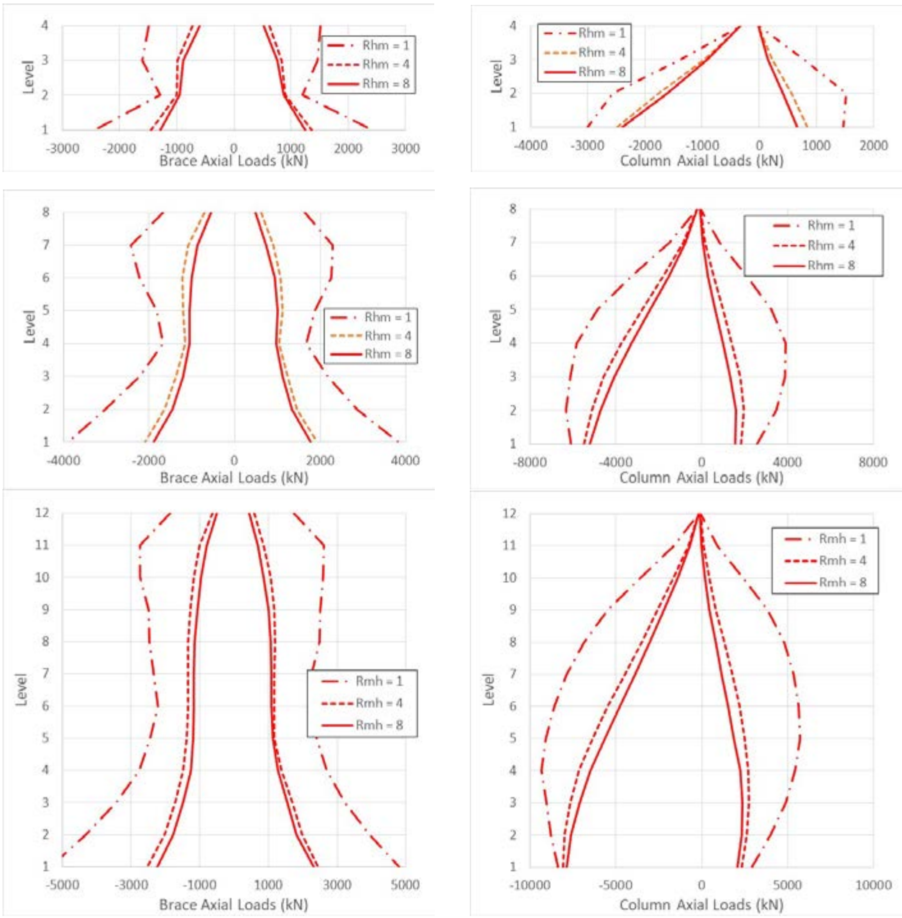

Fig. 7 – Peak force demands in braces and columns

In Fig. 9a, results for the peak column uplift and residual roof drifts are presented. For all three designs, column uplift tends to decrease as the number of storeys is increased, from 121 mm for the 4-storey frame to 79 mm for the 12-storey frame when using R_{hm} = 1.0 in design. This behaviour could be expected as the first mode response responsible for base rocking becomes relatively less important for taller frames. Column uplift also slightly reduced when using nonlinear braces in the frames, likely due to higher energy dissipation capacity of the frames with friction brace connections. The frames designed with R_{hm} = 1 did not experience residual deformations. As anticipated, the frames with nonlinear braces sustained some residual storey drifts, these permanent drifts being larger when using R_{hm} = 8 in design. The values are small, however, being equal to or less than 0.4 % for all cases studied.

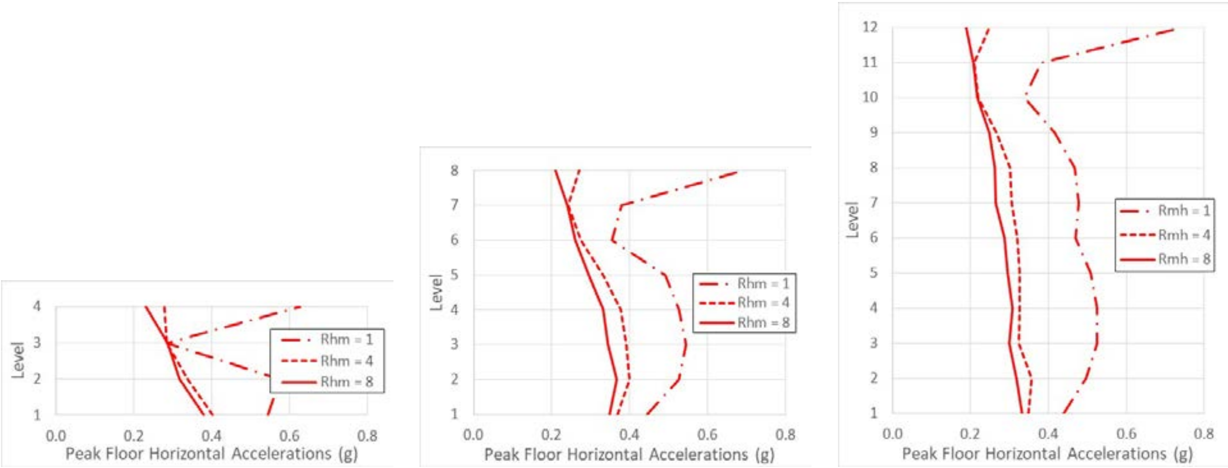

Fig. 8 – Peak floor horizontal accelerations

a) b)

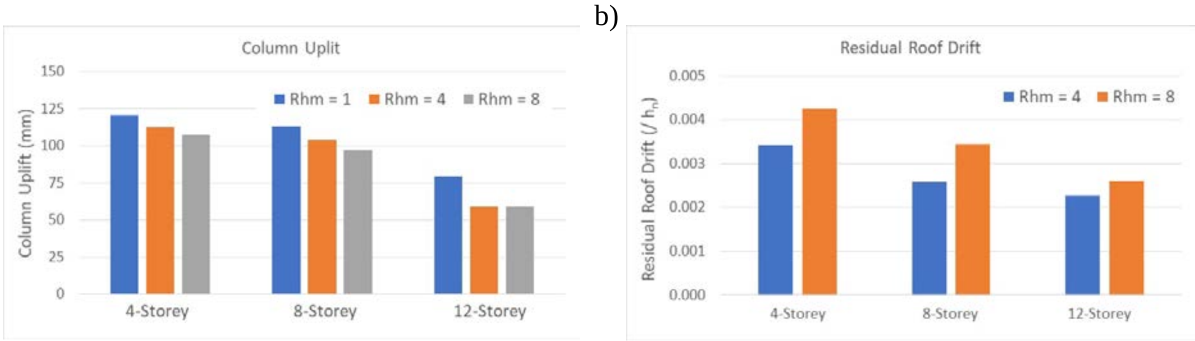

Fig. 9 – a) Peak column uplifts; b) Residual roof drifts.

Figs. 10 and 11 show time histories of the storey drift at the first level of the 12-storey building for two different ground motions. The results are presented for the frames designed with R_{hm} = 4 and 8. For the frame designed with R_{hm} = 4, an additional analysis was performed on a frame that included two nonlinear braces at every level. In the first two cases, the results show that the system possesses re-centering capacity and that very similar response could be reached with the two values of R_{hm}. This desirable response was lost when replacing the elastic braces by nonlinear braces. In Fig. 11, the structure even collapsed due to progressive drifting under the long duration motion from interface subduction earthquake. These results confirm the efficiency of the elastic brace/elastic beam combination for ensuring stable seismic response.

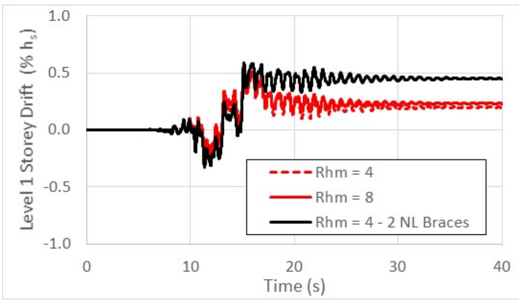

Fig. 10 – Time history of the storey drift at the first level of the 12-storey building under a shallow crustal ground motion

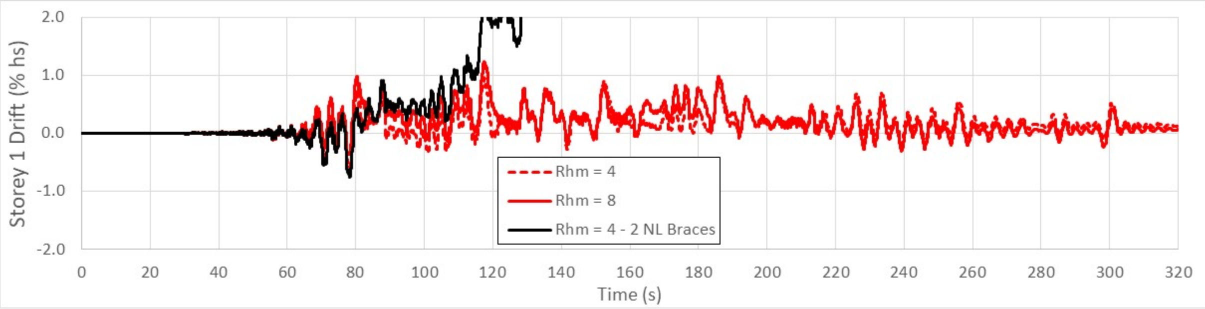

Fig. 11 – Time history of the storey drift at the first level of the 12-storey building under an interface subduction ground motion.

4. Conclusions

This article described a rocking steel braced frame system in which nonlinear storey shear response is permitted to occur along the frame height with the objective of reducing and controlling member forces induced by higher modes of vibration. Nonlinear storey shear response is achieved by using a chevron bracing with one nonlinear brace at every level of the frame. The other brace and the beam are designed to remain elastic such that the system can exhibit a positive storey shear stiffness during nonlinear storey shear response up to a target storey shear value. This positive stiffness is expected to mitigate P-Δ effects on response and provide re-centering capacity to the structure. A method was described to predict the member forces to be used in the design of the nonlinear braces and the other members of the rocking braced frame.

Response history analysis was performed on three prototype G-CRBF structures to validate the proposed concept and design procedure. The structures were designed to remain fully elastic (R_{hm} = 1) or display nonlinear storey shear response (R_{hm} = 4 and 8). The results show that the frames with nonlinear braces sustained experienced similar storey drifts when compared to the frames designed to remain elastic. The addition of the nonlinear braces was found to be effective in reducing and controlling member forces and floor horizontal accelerations. The results also demonstrated the re-centering capacity of the system.

A di-symmetry was observed in the storey drifts due to the gravity loads carried by the braced frame beams and the bracing geometry. It is expected that this shortcoming could be mitigated by using two G-CRBF with nonlinear braces acting in opposite directions. Additional studies are needed to further validate the adequacy of the proposed system and develop complete design provisions for the system.

5. Acknowledgements

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