Canadian Conference - Pacific Conference on Earthquake Engineering 2023
Vancouver, British Columbia
June 25th – June 30th, 2023

Design of Brace Intersected Beams in Inverted-V and Two-Storey X Steel Concentrically Braced Frames

Robert Tremblay[1*] and Hiroyuki Inamasu[2]

[1]Professor, Dept. of Civil, Geological and Mining Engineering, Polytechnique Montréal, Montreal, Canada
[2]Post-Doctoral Researcher, Dept. of Civil, Geological and Mining Engineering, Polytechnique Montréal, Montreal, Canada
[*]robert.tremblay@polymtl.ca (Corresponding Author)

ABSTRACT

Steel concentrically braced frames (CBFs) often adopt inverted-V (IV) (chevron) and two-storey X (2X) bracing configurations. Although the beams in the IV-CBFs are required to remain essentially elastic after buckling of the bracing members for tall buildings, recent studies in the U.S. suggested that limited inelastic response of the beams in the IV-CBFs could result in an acceptable response. As for 2X-CBFs, when performing a capacity design check for the beams intersected by four bracing members at the beam mid-span, it is often assumed that all braces simultaneously reach their probable axial resistances, which results in very small force demand on the beams. However, there is likely a delay in the inelastic responses developing in two adjacent storeys of CBFs, which can lead to much greater force demand on the beams. In this article, alternative beam design approaches for IV-CBFs and 2X-CBFs are explored. In the alternative design approaches, the beam force demand is reduced for the IV-CBFs and increased for the 2X-CBFs. This study includes nonlinear response history analyses (NRHAs) under design-level intensities performed on 4- and 8-storey IV- and 2X-CBFs located in Vancouver, BC and designed in accordance with both the current and alternative approaches. The simulation results suggest that: 1) allowing limited inelastic response of the beams in IV-CBFs may result in a significant soft-storey response while reducing the tensile brace ductility demand and increasing the compressive brace ductility demand, and 2) 2X-CBFs designed with the current Canadian seismic standards with the above-mentioned common assumption, unexpectedly, tend to keep the brace-intersected beams elastic, thus there is no significant benefit to increasing the beam design forces. The findings of this study need to be further confirmed by the NRHAs with the numerical models that are capable of simulating accurately the brace fracture response.

Keywords: Steel concentrically braced frame (CBFs), Force demands on beams, Brace ductility demands, Soft-storey response, Beam yielding.

INTRODUCTION

Steel concentrically braced frames (CBFs) are commonly used in steel building structures in Canada as lateral resisting systems. Among the various possible bracing configurations, the inverted-V (IV) (chevron) and two-storey X (2X) configurations represent the most popular ones for multi-storey building applications in view of their higher structural efficiency and architectural appearance. The inverted-V arrangement also allows door openings in the braced frames.

Current provisions in the CSA S16-19 steel design standard [1] require that the beams in IV-CBFs taller than 4 storeys be designed to remain essentially elastic after buckling of the bracing members, which imposes a significant cost penalty to the system. This requirement aims at mitigating the risk of soft-storey response in tall IV-CBFs. For 4-storey and lower IV-CBFs, limited inelastic response is however permitted, provided that the beams have minimum flexural resistance and are properly detailed to maintain their flexural resistances upon yielding. Recent studies conducted in the U.S. [2–5] have indicated that this approach could result in an acceptable response for taller CBFs. In particular, the approach leads to reduced inelastic tensile deformations demand on the bracing members, which reduced the potential for low-cycle brace fracture. This new approach has been included in the upcoming AISC 341-22 Seismic Provisions in the U.S. [6]. This article describes a preliminary study that was performed to evaluate the possibility of allowing a similar design approach in Canada. The study includes nonlinear response history analyses with design-level intensities performed on 4- and 8-storey structures designed in accordance with both the current and new methods. The structures were assumed to be located on a soft site in Vancouver, BC.

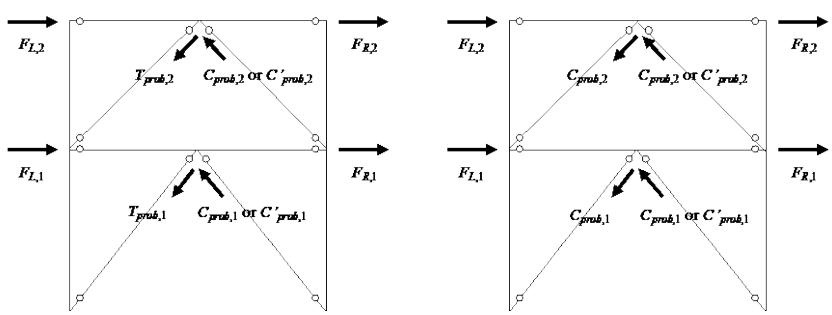
For the two-storey X bracing configuration, current seismic provisions in CSA S16-19 do not provide any guidance for the design of the beams intersected by the bracing members at mid-span. When performing a capacity design check for these beams, the force demand on the beams is generally low because it is assumed in the verification that all four braces connecting to the beams simultaneously reach their probable axial resistances. In reality, there is likely a delay in the inelastic responses developing in two adjacent storeys of a braced frame, which can lead to much greater force demand on the beams [7–9]. Some studies in the U.S. reported that the beams of 2X-CBFs designed without considering such a delay in the storey responses result in the yielding of the brace-intersected beams and larger brace ductility demands compared to the 2X-CBFs with properly capacity-designed beams [7–9]. This article also presents a preliminary study that was performed to evaluate the axial and flexural load demands that can be imposed on the beams. The study was performed on the same frame structures used for the study of the IV-CBFs, except that the 2X bracing configuration was used. The beams were designed assuming different brace force scenarios. Nonlinear response history analyses (NRHAs) of the frames were then conducted to validate the assumed brace force scenarios.

ALTERNATIVE BEAM DESIGN APPROACHES

An alternative design approach for beams of IV-CBFs

In the current CSA S16-19 standard [1], the beams of IV-CBFs are required to be capacity-designed. This achieved by determining beam forces by assuming that the probable resistances in all the bracing members under the first mode deformed shape as shown in Fig. 1a. Two conditions need to be considered for the axial load in the compressive braces: i) probable compressive resistance at first buckling (C_{prob}), and ii) probable post-buckling compressive resistance (C'_{prob}). The difference between the tensile and compressive brace resistance results in an unbalanced downward vertical force at the beam mid-span, which induces flexural demand on the beam. The axial load demand of the beam is due to the sum of the horizontal components of the brace forces.

Recent studies in the U.S. [2–5] suggested that IV-CBFs with beams designed with smaller force demand can exhibit similar or better seismic performance than IV-CBFs with capacity-designed beams, especially because beams designed for reduced forces are expected to yield under seismic excitations, which reduces inelastic tensile deformations on the bracing members and, thereby, reduces the potential of low-cycle brace fracture. In their study, the beams were designed using the brace loading condition shown in Fig. 1b. In this article, this brace loading condition is explored as an alternative design approach in the Canadian design context.



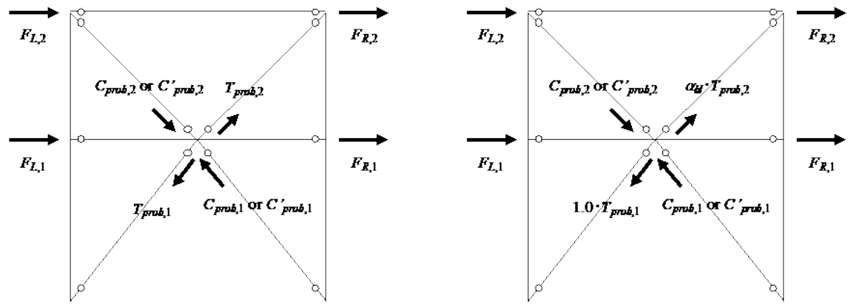
(a) (b)

Figure 1. Brace forces on the beam in IV-CBFs considered in design: (a) current S16-19 approach; (b) alternative approach.

An alternative design approach for brace-intersected beams of 2X-CBFs

Although the current CSA S16-19 standard [1] does not provide any guideline for the design of 2X-CBF beams, beams intersected by four braces at their mid-span are often designed assuming that all four braces reach their probable tensile and compressive resistances simultaneously, as shown in Fig. 2a. Two possible loading conditions are considered for the compressive braces, as explained for IV-CBF beam design, i.e., at first buckling and in the post-buckling range.

In this article, as an alternative design approach, loading conditions shown in Fig. 2b are explored. In this approach, the probable tensile brace forces in the even-numbered storeys are multiplied by the brace force reduction factor, α_{bl}. This factor is introduced to account for the possible delay between the storey responses in the adjacent stories that form 2X bracing. The factor is applied only to the tensile brace forces and not to the compressive brace forces in order to apply higher force demand on the beam. This alternative approach with the factor α_{bl} = 1.0 reduces to the common approach shown in Fig. 2a. As the α_{bl} factor gets smaller, the force demand on the beam increases, which leads to larger beam sections.



(a) *(b)*

Figure 2. Brace forces on the intersected beam in 2X-CBFs considered in design: (a) current S16-19 approach; and (b) alternative approach.

FRAME DESIGN AND NUMERICAL MODELS

Frame design

The seismic performance of CBFs with the beams designed with the alternative beam design approaches described in the previous section was investigated by comparing it with the seismic performance of CBFs with beams designed with the current conventional approaches. For this purpose, moderately ductile (Type MD) CBFs were designed for the office building shown in Fig. 3. The building is assumed to be located in Vancouver, BC, which falls in Seismic Category 4. The design spectrum of the selected site is shown in Fig. 4. The first storey height is 4.8 m while the height of the other storeys is 4.0 m. The CBFs in the EW direction were designed and analyzed in this study. A total of eight different Type MD CBFs (four IV-CBFs and four 2X-CBF), as summarized in Table 1, were designed. Two different building heights were considered: 4 and 8 storeys. The number of braced frames along the EW direction is 4 for the 4-storey buildings and 6 for the 8-storey buildings. For each building height and bracing configuration, the current and the alternative beam design approaches were applied. For the design of the beams in 2X-CBF with the alternative design approach, the factor α_{bl} was set equal to 0.5.

Other than the force demand on the beams, the frames were designed in accordance with the seismic provisions of the CSA S16-19 standard [1]. For all buildings, the seismic loads were determined using overstrength-related and ductility-related force modification factors R_o = 1.3 and R_d = 3.0, respectively. The dynamic (multimode response spectrum) analysis method was used for all buildings. Accidental torsion was neglected in design and each braced frame was assigned a portion of the total earthquake load for the building divided by the number of braced frames in the E-W direction. In this study, notional loads and seismic load amplification factors for P-delta effects were ignored in the design. Wind loads were also ignored in the design.

In each storey, the seismic storey shear demand is fully resisted by a pair of compression and tension acting braces. The brace sections were selected so that their factored compressive resistance for out-of-plane buckling exceeded the factored compression load from seismic and concomitant gravity loads. The brace effective length was taken equal to 0.95 times the length between the "$2t_g$" plastic hinges in the gusset plates. The factor of 0.95 accounts for the rotational restraint due to the flexural stiffness of the gusset plates. Both brace local and global slenderness limits were also satisfied. However, the lower limit on global brace slenderness was relaxed from 70 to 60 in this study. ASTM A1085 square hollow structural sections (HSS) with F_y = 345 MPa were adopted for all braces investigated in this study.

Wide flange sections, ASTM A992 Gr. 50 (i.e., F_y = 345 MPa), were adopted for beams and columns of the braced frames. Both of them were oriented such that flexural demand developed about their strong axis when the braced frame is subjected to in-plane seismic loads. The columns were assumed to be continuous for two storeys at minimum. As explained above, two different brace force scenarios were considered for beam design. In both cases, the beams were selected to satisfy the P-M interaction equation for cross-sectional strength:

$$\frac{C_f}{C_r} + \frac{0.85 U_{1x} M_{fx}}{M_{rx}} \leq 1.0 \tag{1}$$

$$C_r = \phi A F_y \tag{2}$$

$$M_{rx} = \phi Z_x F_y \tag{3}$$

, where C_f and M_{fx} are the axial load and bending moments about strong axis induced by the brace axial forces plus gravity loads, C_r and M_{rx} are the factored axial and flexural resistances of the member, U_{1x} is a factor that accounts for member second-order effects, A and Z_x are the cross-sectional area and the plastic section modulus about the strong axis of the member, F_y is the nominal steel yield stress of the member, and ϕ is the resistance factor. Out-of-plane instability of the beams, including lateral-torsional buckling, was considered to be prevented by proper lateral bracing. In-plane buckling of the beams about strong axis was omitted as the global slenderness ratios of the beams were all very small (i.e., less than 20 with the half-span length). The resistance factor $\phi = 0.9$ was considered in the beam design. While the beams were sized based on the assumed loading scenarios explained above for each design approach, the columns in all frames and the no-brace-intersected beams in the 2X-CBFs were capacity designed using the loading conditions assumed in current CSA S16-19, i.e., assuming the loading conditions shown in Fig. 1a and Fig. 2a. Both beam and column profiles were selected from class 1 or 2 sections. The deflection limit of the beam under gravity loading was also verified.

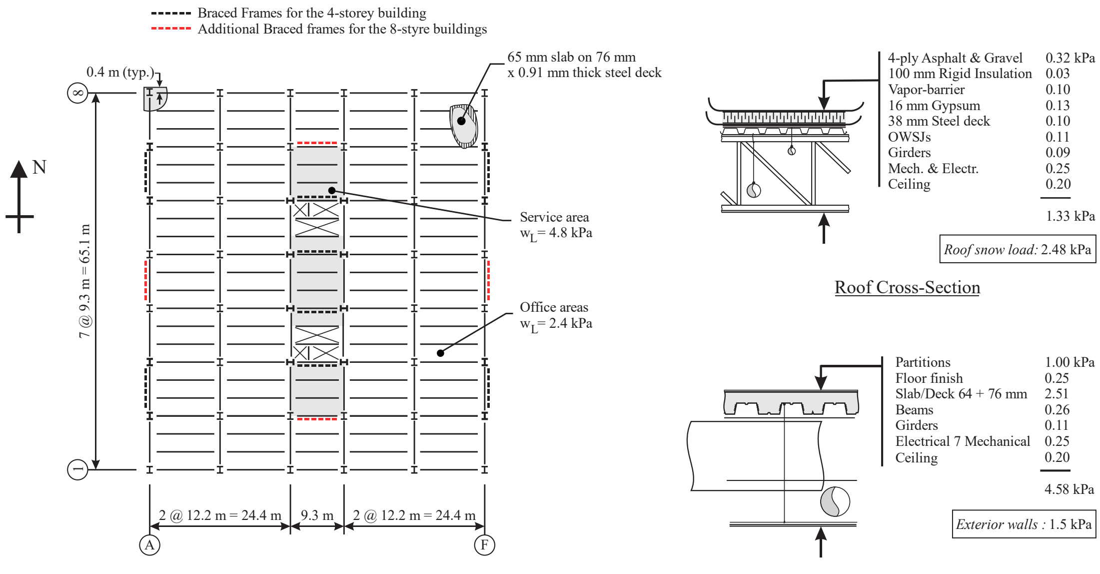


Figure 3. Structure plan view and design gravity loads of the buildings studied.

Table 1. Investigated braced frames.

ID	Num. of storey	Brace config.	Beam design approach
4-IV-T	4	Inverted-V	Current (T_{prob} and C_{prob} / C'_{prob})
4-IV-C	4	Inverted-V	Alternative (C_{prob} and C_{prob} / C'_{prob})
8-IV-T	8	Inverted-V	Current (T_{prob} and C_{prob} / C'_{prob})
8-IV-C	8	Inverted-V	Alternative (C_{prob} and C_{prob} / C'_{prob})
4-2X-100	4	Two-storey X	Current ($\alpha_{bl} = 1.0$)
4-2X-50	4	Two-storey X	Alternative ($\alpha_{bl} = 0.5$)
8-2X-100	8	Two-storey X	Current ($\alpha_{bl} = 1.0$)
8-2X-50	8	Two-storey X	Alternative ($\alpha_{bl} = 0.5$)

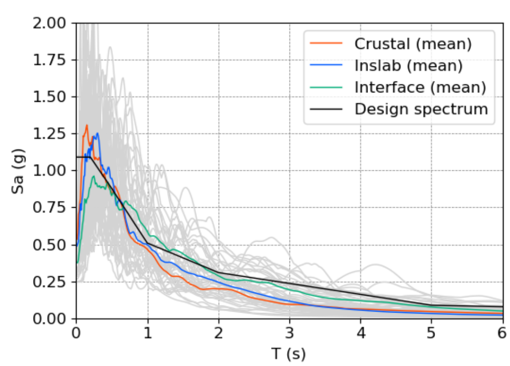


Figure 4. Design spectrum and response spectra of the scaled ground motion records considered in this study.

Slotted welded connections were adopted for the brace-to-gusset connections, and the gusset plates were detailed to accommodate the brace out-of-plane end rotations upon brace buckling by means of a plastic hinge with a length equal to two times the gusset plate thickness, i.e., an $2t_g$ clearance with t_g being the gusset plate thickness. Beam-to-column connections were simple shear tab connections. Therefore, beams were considered as pin-ended in design. At locations where braces connect to beams at beam-to-column connections, the connections consisted of a beam stub and gusset plates shop-welded to the columns, with a shear tab connection between the beam stub and the beam, as illustrated in Fig. 5. Column splices were located 1500 mm above the floor beam centerline such that the flexural demand on the splice connections is minimized. Column bases were assumed to be pinned in design, which is a common assumption for steel CBFs, and an exposed base plate connection detail was used at the column bases.

For each bracing configuration and building height, the same profiles were used for the braces and the columns. Only the beam sizes varied depending on the beam design approach that was used. Member sections are given in Tables 2 and 3. Note that the beam depth was limited to W760 and W610 for the IV- and 2X-CBFs, respectively, to reflect common practice for each bracing configuration. The beam tonnage was reduced by 34 % and 28 % for the 4-storey and 8-storey IV-CBFs, respectively, when adopting the alternative beam design approach. Conversely, the beam tonnage was increased by 88 % and 119 % for the 4-storey and 8-storey 2X-CBFs when adopting the alternative beam design approach. The fundamental periods of the braced frames are equal to 0.79 s (4-storey IV-CBFs), 1.34 s (8-storey IV-CBFs), 0.76 s (4-storey 2X-CBFs), and 1.31 s (8-storey 2X-CBFs).

Table 2. Beam profiles selected for each IV-CBF.

Storey	4-storey		8-storey	
	Current	**Alternative**	**Current**	**Alternative**
8	-	-	W760x196	W610x92
7	-	-	W760x257	W760x147
6	-	-	W760x314	W760x257
5	-	-	W760x350	W760x257
4	W760x220	W760x134	W760x350	W760x257
3	W760x314	W760x257	W760x434	W760x284
2	W760x434	W760x284	W760x434	W760x284
1	W760x582	W760x434	W760x531	W760x314

Table 3. Beam profiles selected for each 2X-CBF.

Storey	4-storey		8-storey	
	Current	**Alternative**	**Current**	**Alternative**
8	-	-	W360x51	W360x51
7	-	-	W610x92	W610x217
6	-	-	W460x74	W460x74
5	-	-	W610x140	W610x341
4	W360x51	W360x51	W610x82	W610x82
3	W610x195	W610x341	W410x38.8	W610x285
2	W610x92	W610x92	W610x92	W610x92
1	W610x140	W610x415	W610x140	W610x415

Numerical models

For the purpose of evaluating the seismic response of the CBFs with the beams designed with the alternative design approaches, nonlinear response history analyses (NRHA) were conducted using the Open System for Earthquake Engineering Simulation, OpenSees v. 3.2.0 [10]. Comprehensive numerical models were developed to conduct these analyses. The developed numerical model is illustrated in Fig. 5 for both braced frame configurations. The numerical model included one of the braced frames along the E-W direction, together with two leaning columns to account for global P-delta effects due to gravity loads carried by the portion of the building laterally stabilized by the braced frame studied. Gravity columns of the building structures were not included in the models. Although the braced frames studied are planar, three-dimensional models were used to reproduce the out-of-plane buckling response of the bracing members and columns.

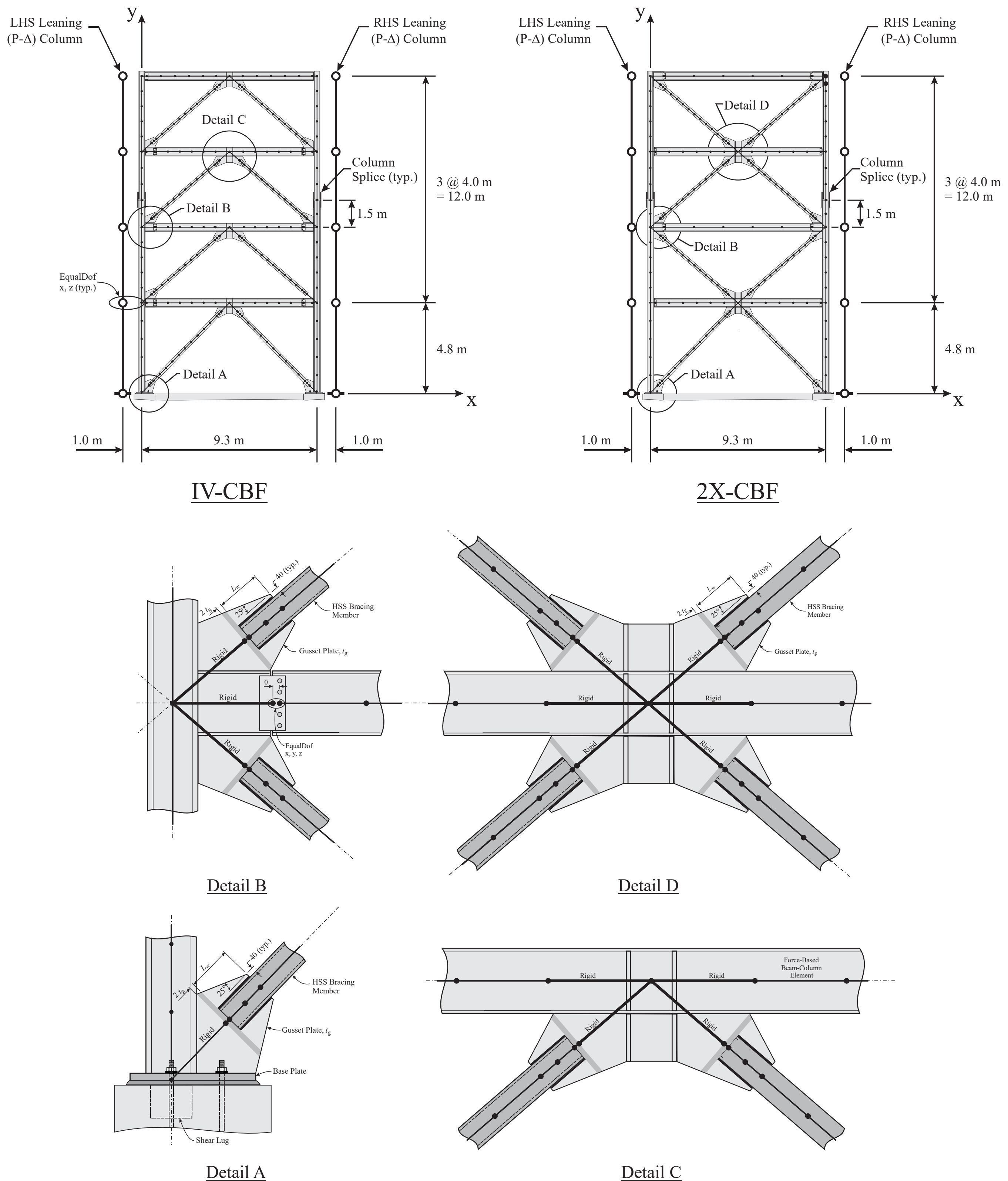


Figure 5. Braced frame numerical model.

All of the structural members such as braces, beams, and columns were modelled using the force-based nonlinear beam-column elements with the Gauss-Lobatto integration scheme and 5 integration points. Each brace member was divided into 10 elements. Round corners of the square hollow sections (SHS) of the braces were explicitly modelled in the cross-section fibres. In the SHS, 5 fibres for the through-thickness direction, 6 fibres along the width direction in the flat wall parts, and 4 fibres along the corners were considered. No residual stresses nor fracture material model, which simulates the low-cycle fatigue response of the braces, were considered in the models for this study. Geometric global imperfections with an amplitude of 0.3 % of the brace length were included in the braces for out-of-plane direction in order to trigger brace buckling. Beam and column members were divided into 5 elements. 10 fibres for the through-thickness direction of the flanges, 5 fibres for the through-thickness directions of the web, and 20 fibres for both the web depth and the flange width directions were applied. Residual stresses based on [11] were considered in the wide flange sections for the beams and columns. Geometric global imperfections with an amplitude of 0.1 % of the member length were included in the columns for both in-plane and out-of-plane directions. Out-of-plane translational degrees of freedom was restrained for the beam nodes to be consistent with the design assumption. The Menegotto-pinto steel material model (i.e., Steel02 material model in OpenSees) with calibrated parameters was adopted for all structural members. Note that the probable yield stress, $F_{ye} = R_yF_y$, was used in the numerical simulations. Specifically, F_{ye} = 460 MPa was considered for bracing members, and F_{ye} = 385 MPa for the beams and columns, in accordance with the provisions of CSA S16-19. The gusset plates were modelled using force-based nonlinear beam-column elements with fibre discretization to better represent the interaction between axial and flexural responses. The "2 t_g" hinge zone was modelled using 15 fibres across the width and 10 fibres across the gusset plate thickness. At the beam-column connections, rigid offsets were considered as shown in Fig. 5. At the shear-tab connections, no flexural resistance was assigned (i.e., pinned connection). Column bases were assumed to be fixed to represent the fixity restraint provided by the anchor rods. Splice connections were also assumed to be continuous for in-plane and out-of-plane flexure. For performing NRHA, 2% Rayleigh damping was applied in the first two modes for the 4-storey frames and in the first and third modes for the 8-storey frames. The damping stiffness was obtained using a mass-proportional factor and a committed stiffness-proportional factor. Horizontal masses were assigned at the nodes of the P-delta columns. In each NRHA, gravity loads computed from the loading combination of 1.0 times the dead loads, 0.5 times the floor live load, and 0.25 times the roof snow load were first applied to the frame; NRHA was then performed using the scaled ground motion records.

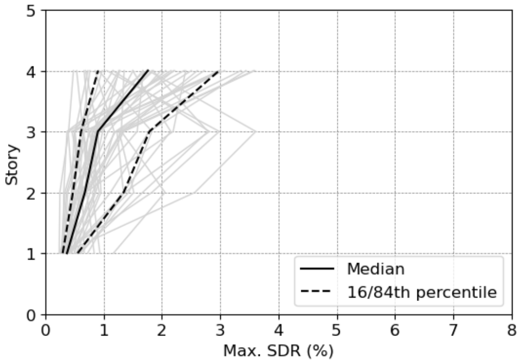
Ground motion record selection

An ensemble of site-representative ground motion records was created from the online strong motion database such as [12,13] for the design location, Vancouver. The ground motions were selected based on the magnitude-distance scenarios dominating the seismic hazard. A total of 44 ground motion records were selected: 11 from crustal earthquakes, and 33 from subduction-zone earthquakes. For the latter group, 11 were from deep in-slab earthquakes and 22 were from interface subduction earthquakes. The selected ground motion records were linearly scaled such that the mean values of the records from crustal and in-slab earthquakes and those from interface earthquakes match the design spectrum, which corresponds to 2 % in 50 years uniform hazard, in the period range of 0-1.2 sec and 1.0-4.0 sec, respectively, as shown in Fig. 4. The scale factors applied to the original records are within 1.2-2.2 (mean: 1.65), 1.8-3.0 (mean: 2.42), and 0.6-3.0 (mean: 1.39) for the record sets obtained from crustal, in-slab, and interface earthquakes.

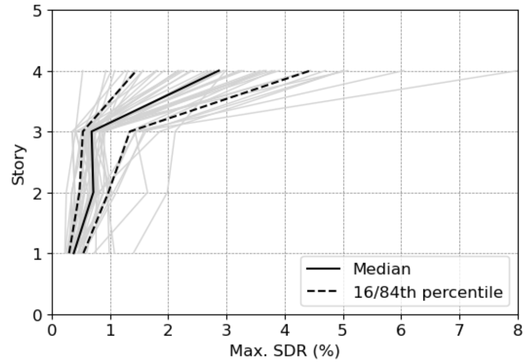
Nonlinear response history analysis results of IV-CBF frames

Structural responses of the four investigated IV-CBF models under NRHAs using 44 scaled ground motion records are summarized in this section. The maximum storey drift ratios (SDRs) of each storey are plotted in Fig. 6. In all frames, SDR tended to concentrate in the top storey, which had the largest or the second largest demand-to-capacity ratio for brace forces. For both building heights, the maximum SDR over the frame was larger when the beams were designed with the alternative design approach. For the 8-storey frame with the alternative beam design approach, a significant level of SDR concentration was observed. This may be attributed to the difference in the lateral storey resistances. In the alternative beam design approach, the storey shear resistance that can be mobilized from the braces is reduced compared to the current beam design approach because the beams are not designed to withstand brace forces larger than the brace compressive resistances. However, this result is not consistent with prior studies conducted in the U.S. [2,3] in which maximum SDR response was comparable between the current and the alternative beam design approaches and the SDR distribution along the frame height was more uniform when adopting the alternative design approach. It should be noted here that brace fracture due to the low-cycle fatigue was not simulated in this study. Moreover, unlike the common practice in the U.S., the beam-column connections in the braced frames are not rigid connections, even with the presence of the gusset plates (see Fig. 5). The resistances of the gravity frame columns and shear tab connections were also neglected in this study. Taking such phenomena into account in the NRHA might change the SDR responses observed in this study, which should be examined further in future investigation.

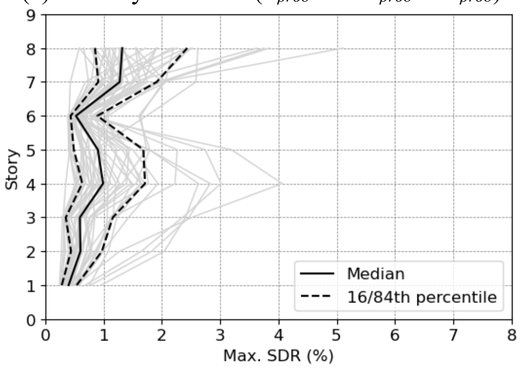
The maximum and minimum vertical displacement values of the beam mid-span are summarized in Fig. 7. Positive values correspond to upward displacements. The beam deflections were limited in the frames with the beams designed with the current approach, regardless of the building height. On the other hand, the beams designed with the alternative beam design approach yielded and experienced a considerable amount of vertical downward deflections. The deflections were the largest for the roof beam where the SDR response was also the largest.



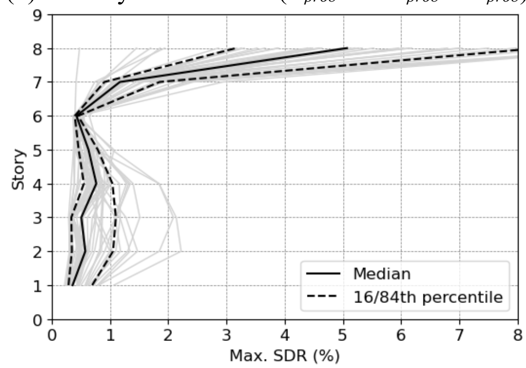
(a) 4-storey / Current (T_{prob} and C_{prob} / C'_{prob})



(b) 4-storey / Alternative (C_{prob} and C_{prob} / C'_{prob})

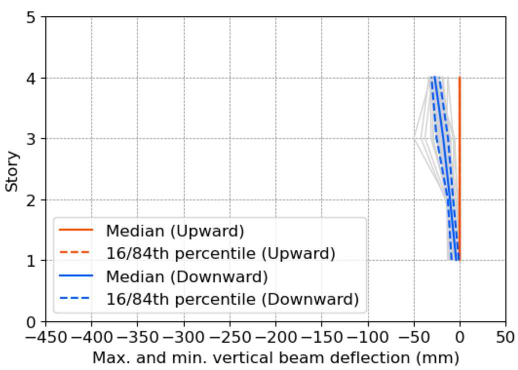


(c) 8-storey / Current (T_{prob} and C_{prob} / C'_{prob})

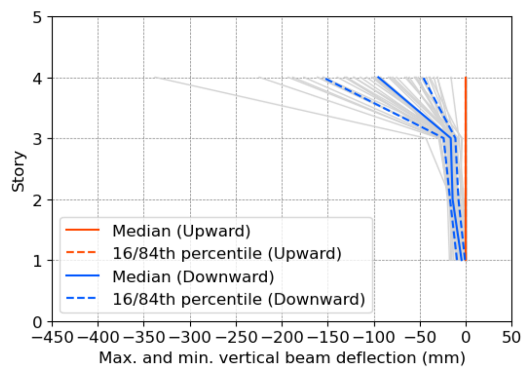


(d) 8-storey / Alternative (C_{prob} and C_{prob} / C'_{prob})

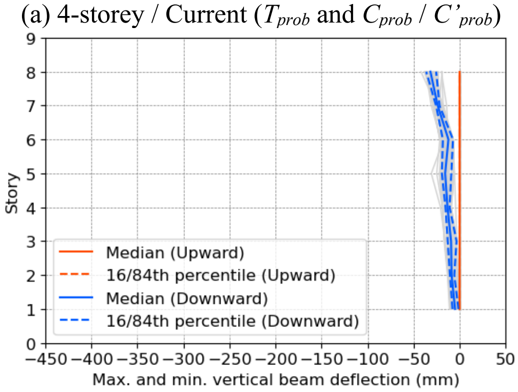
Figure 6. Maximum storey drift ratio (SDR) of each storey of the IV-CBFs.



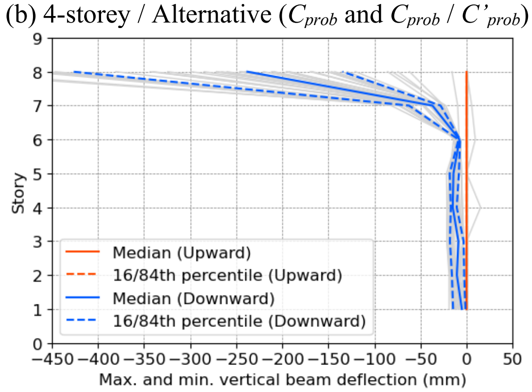
(a) 4-storey / Current (T_{prob} and C_{prob} / C'_{prob})



(b) 4-storey / Alternative (C_{prob} and C_{prob} / C'_{prob})



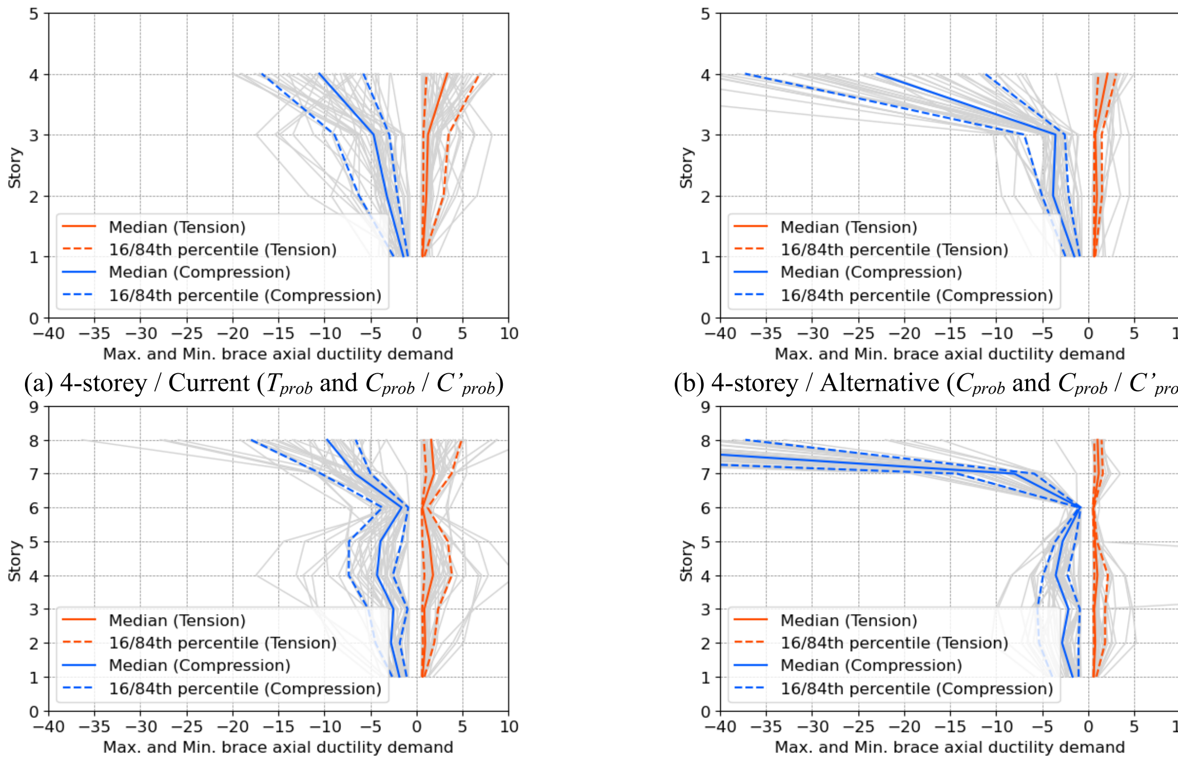
(c) 8-storey / Current (T_{prob} and C_{prob} / C'_{prob})



(d) 8-storey / Alternative (C_{prob} and C_{prob} / C'_{prob})

Figure 7. Maximum and minimum vertical displacement of beam mid-span of the IV-CBFs.

The maximum and minimum brace ductility levels reached in the analyses were also investigated. Those are summarized in Fig. 8. The tensile ductility is defined as the maximum brace axial deformation divided by the brace yield axial deformation based on expected brace yield strength. The compressive ductility is the minimum brace axial deformation divided by the brace axial deformation at brace buckling. Median tensile ductility levels were reduced by 2 to 37 % for the 4-storey frames and by 16 to 40 % for the 8-storey frames when the beams were designed with the alternative beam design approach. However, on the contrary, the compressive ductility levels were significantly increased when using the alternative beam design approach. Since the influence of the tensile and compressive ductility levels on the fracture potential of the brace members are not same, it is not easy to conclude which design approach is superior for reducing the potential of brace fracture. Fracture potential needs to be further assessed using models that can predict brace fracture accurately under arbitrary loading histories. In the alternative design frames, tensile ductility levels reached values greater than 1.0, even if the beams were designed for the loading conditions shown in Fig. 1b. This behaviour may be because the beams were able to withstand the probable brace tensile forces higher than C_{prob} due to the overstrength of the beams resulting from the discrete choice of W-profiles, the use of the ϕ factor in beam design, the use of the expected yield strength of the beams in the models, and strain hardening in the beams.



(a) 4-storey / Current (T_{prob} and C_{prob} / C'_{prob}) (b) 4-storey / Alternative (C_{prob} and C_{prob} / C'_{prob})

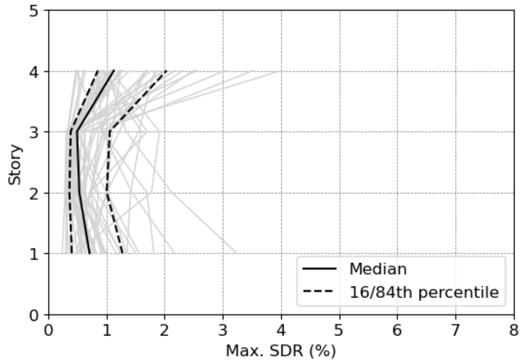
(c) 8-storey / Current (T_{prob} and C_{prob} / C'_{prob}) (d) 8-storey / Alternative (C_{prob} and C_{prob} / C'_{prob})

Figure 8. Maximum and minimum brace ductility demands of each storey of the IV-CBFs.

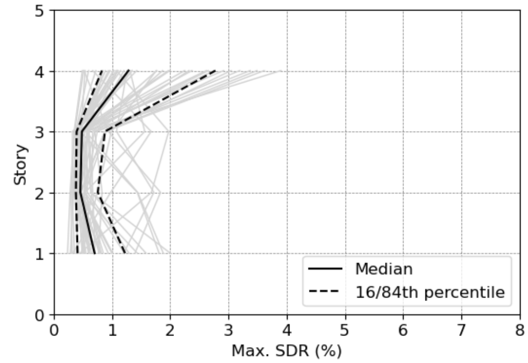
Nonlinear response history analysis results of 2X-CBF frames

Structural responses of the four investigated 2X-CBF models under NRHAs using 44 scaled ground motion records are summarized in this section. The maximum SDR distributions are compared in Fig. 9. Between the current and the alternative design approaches, similar SDR responses were observed. Referring to the median values, the maximum SDR values for the 4- and 8-storey frames were slightly larger when the alternative beam design was used. The dispersion of the maximum SDR over the frame appears also higher when the alternative beam design approach was applied. However, the difference is not significant.

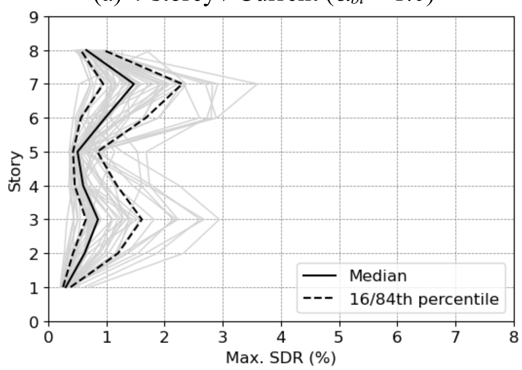
The maximum and minimum brace ductility levels are shown in Fig. 10. The ductility definitions are as described for the IV-CBFs. The trends are similar to those observed for the maximum SDR distributions. While prior studies highlighted that the brace ductility demands are higher when using the current beam design approach compared to frames in which the beams were conservatively designed to remain elastic [7], such a trend was not observed in this study.



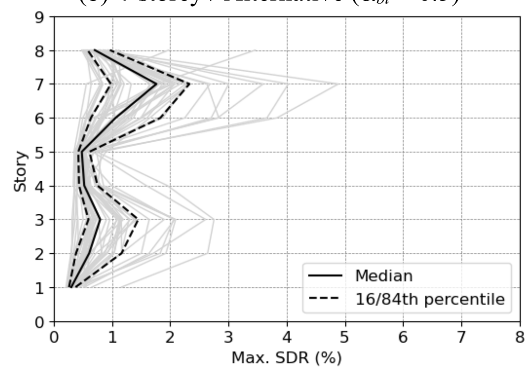
(a) 4-storey / Current (α_{bl} = 1.0)



(b) 4-storey / Alternative (α_{bl} = 0.5)

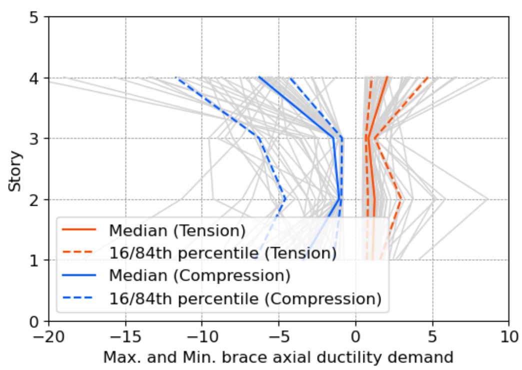


(c) 8-storey / Current (α_{bl} = 1.0)

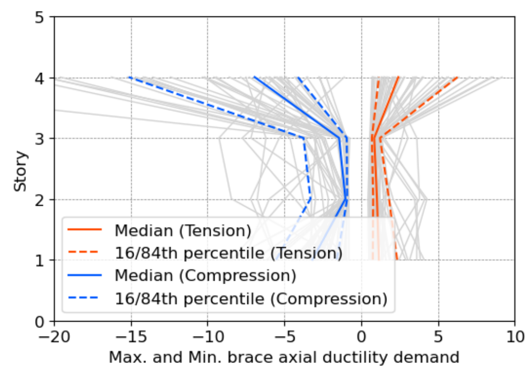


(d) 8-storey / Alternative (α_{bl} = 0.5)

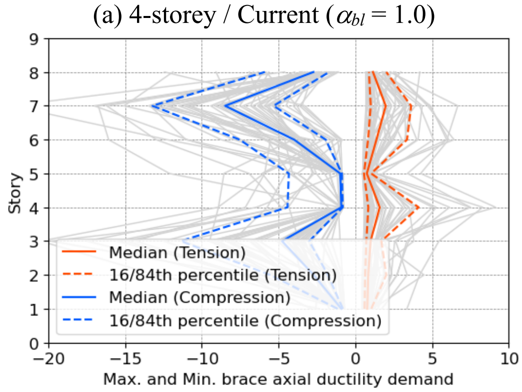
Figure 9. Maximum storey drift ratio (SDR) of each storey of the 2X-CBFs.



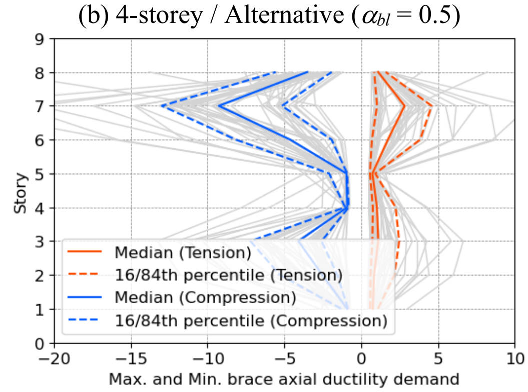
(a) 4-storey / Current (α_{bl} = 1.0)



(b) 4-storey / Alternative (α_{bl} = 0.5)



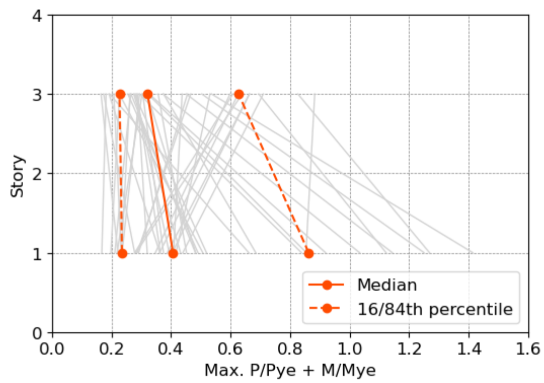
(c) 8-storey / Current (α_{bl} = 1.0)



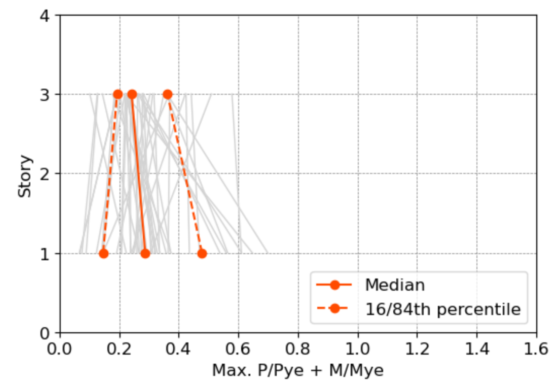
(d) 8-storey / Alternative (α_{bl} = 0.5)

Figure 10. Maximum and minimum brace ductility demands of each storey of the 2X-CBFs.

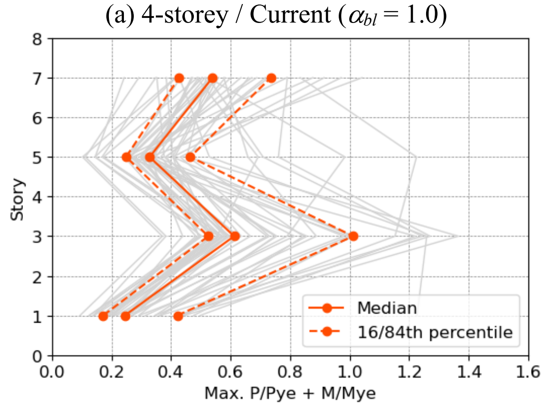
The axial load–bending moment (P-M) interaction of the beam mid-span, where the beam force demand is largest, was also investigated. The maximum values of the P-M interaction equation for each brace-intersected beam are plotted in Fig. 11. The absolute sum of P/P_{ye} and M/M_{ye} was taken as the P-M interaction in order to evaluate if the beams yielded during the seismic motions. Here, P and M are the axial load and strong-axis moment sustained by the beams at the interfaces between the rigid portion of the beam at its mid-span and the left- and right-hand side beam portions, and P_{ye} and M_{ye} are the axial yield strength and the yield moment calculated with the probable steel yield stress of the beams (385 MPa). An interaction equal to 1.0 corresponds to initiation of yielding of the beam section. When using the current beam design approach (i.e., α_{bl} = 1.0), surprisingly, the beams remained elastic under the majority of the seismic motions, with the interaction exceeding 1.0 in only a few cases. This finding is not consistent with prior studies [7–9] where CBFs were designed in accordance with U.S. standards. The ϕ factor and material overstrength (i.e., the ratio between the probable and nominal yield stresses) considered in the Canadian design standard [1] might have been large enough to prevent beam yielding in the 2X-CBFs. Findings of this study suggest that the phase difference of adjacent storey responses may not be a great concern for beam design in 2X-CBFs. In the alternative beam design approach, the median P-M interaction values were less than for the current beam design approach, and none of the values were greater than 1.0, indicating that the loading conditions assumed in Fig. 2b, with α_{bl} = 0.5, are effective in preventing beam yielding. The fact that beam yielding did not occur when adopting either of the two design approaches probably explains why similar responses were observed for the maximum SDR and brace ductility levels. While the results suggest that the current beam design approach for the 2X-CBFs might be adequate, they need to be confirmed in future investigations for other building configurations and other seismic conditions. Also, although the beam design approach did not seem to affect significantly the brace ductility levels, brace fracture should be incorporated into models used to validate the findings of this study.



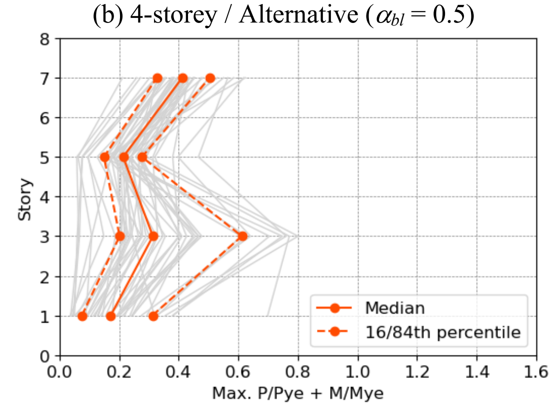
(a) 4-storey / Current (α_{bl} = 1.0)



(b) 4-storey / Alternative (α_{bl} = 0.5)



(c) 8-storey / Current (α_{bl} = 1.0)



(d) 8-storey / Alternative (α_{bl} = 0.5)

Figure 11. Maximum values of P-M interaction equation for the brace intersected beams of the 2X-CBFs.

CONCLUSIONS

This article explored alternative beam design approaches for inverted-V (IV) and 2-storey-X (2X) steel concentrically braced frames (CBFs) recently investigated in the U.S. when applied in the Canadian seismic design context. In the alternative beam design approach for IV-CBFs, the probable tensile brace force was replaced by a tensile force equal to the brace probable compressive resistance in the calculation of the design beam forces for the purpose of reducing the size of the beams. For the

2X-CBFs, in the alternative beam design approach, the probable brace tensile resistance in the even-numbered storeys was multiplied by a reduction factor, α_{bl}, which increased the beam force demand in order to ensure that the beams will remain elastic. In this study, 4- and 8-storey IV- and 2X-CBFs located in Vancouver, BC, were designed using the current and the alternative beam design approaches. Nonlinear response history analyses (NRHA) that do not consider brace fracture response were performed under design-level ground motion records. The findings of this study are summarized as follows:

- For the IV-CBFs, higher maximum storey drift ratios (SDR) over the frame height were observed when the beams were designed with the alternative design approach, both for the 4- and 8-storey frames. In the 8-storey frames, a significant level of SDR concentration was observed when the alternative beam design approach was adopted. This result is not consistent with the recent studies conducted in the U.S. [2,3]. With the alternative beam design approach, median peak tensile brace ductility levels were reduced by up to 40 %. However, median peak compressive brace ductility levels were increased by more than 400 %. A thorough fracture potential assessment under such non-symmetric ductility demands is needed before final conclusions can be stated on the possibility of adopting the alternative beam design approach in Canada.
- In 2X-CBFs with beams designed I accordance with the current approach, i.e., by assuming that all four braces intersecting at the beam mid-span reach their probable resistances simultaneously, unexpectedly, brace-intersected beams remained elastic under most of the applied design-level ground motions. This can be attributed to the difference between factored beam resistances used in design, with ϕ = 0.9 and nominal yield stress, compared to the expected beam resistance with probable yield stress in the beam models. As expected, the alternative beam design for the 2X-CBFs ensured elastic response for the beams. The maximum SDR response and the brace ductility levels were similar regardless of the beam design approach as the beams exhibited elastic response in both cases. While these results suggest that the current beam design approach for the 2X-CBFs is adequate, further investigation is needed to confirm the findings of this study for other building configurations and seismic conditions, whit brace fracture explicitly taken into account in the seismic analyses.

ACKNOWLEDGMENTS

Funding for this research was provided by the Natural Sciences and Engineering Research Council (NSERC) of Canada.

REFERENCES

[1] CSA (Canadian Standards Association) (2019). S16:19 Design of steel structures.
[2] Asada, H., Sen, A.D., Li, T., Berman, J.W., Lehman, D.E., Roeder, C.W. (2020). "Seismic performance of chevron-configured special concentrically braced frames with yielding beams". *Earthquake Engineering & Structural Dynamics*, 49(15), 1619–1639.
[3] Tan, Q., Lehman, D.E., Roeder, C.W., Berman, J.W., Sen, A.D., Wu, B. (2021). "Design-parameter study on seismic performance of chevron-configured SCBFs with yielding beams". *Journal of Constructional Steel Research*, 179, 106561.
[4] Roeder, C.W. Sen, A.D., Terpstra, C., Ibarra, S.M., Liu, R., Lehman, D.E., Berman, J.W. (2019). "Effect of beam yielding on chevron braced frames". *Journal of Constructional Steel Research*, 159, 428–441.
[5] Roeder, C.W., Sen, A.D., Asada, H., Ibarra, S.M., Lehman, D.E., Berman, J.W., Tsai, K.-C., Tsai, C.-Y., Wu, A.-C., Wang, K.-J., Liu, R. (2020). "Inelastic behavior and seismic design of multistory chevron-braced frames with yielding beams". *Journal of Constructional Steel Research*, 167, 105817.
[6] AISC (American Institute of Steel Construction) (2022). Seismic provisions for structural steel buildings: Public review ballot. ANSI/AISC 341-22.
[7] Shen, J., Wen, R., Akbas, B., Doran, B., Uckan, E. (2014). "Seismic demand on brace-intersected beams in two-story X-braced frames". *Engineering Structures*, 76, 295–312.
[8] Shen, J., Wen, R., Akbas, B. (2015). "Mechanisms in two-story X-braced frames". *Journal of Constructional Steel Research*, 106, 258–277.
[9] Momenzadeh, S., Shen, J. (2018). "Seismic demand on columns in special concentrically braced frames". *Engineering Structures*, 168, 93–107.
[10] McKenna, F. (1997). "Object oriented finite element programming frameworks for analysis, algorithms and parallel computing", Ph.D. Thesis, University of California, Berkeley, USA.
[11] Galambos, T.V., Ketter, R.L. (1959). "Columns under combined bending and thrust", *Journal of the Engineering Mechanics Division*, 85, 1–30.
[12] Ancheta, T.D., Darragh, R.B., Stewart, J.P., Seyhan, E., Silva, W.J., Chiou, B.S.-J., Wooddell, K.E., Graves, R.W., Kottke, A.R., Boore, D.M., Kishida, T., Donahue, J.L. (2014). "NGA-West2 Database". *Earthquake Spectra*, 30, 989–1005.
[13] Mazzoni, S., Kishida, T., Contreras, V., Ahdi, S.K., Kwak, D.Y., Bozorgnia, Y., Stewart, J.P. (2021). "NGA-Sub Flatfile: R211022". *The B. John Garrick Institute for the Risk Sciences*.