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# Seismic design and behaviour of chevron steel braced frames

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## ABSTRACT

This paper presents the results of nonlinear dynamic analyses performed to examine the seismic performance of chevron braced frames for steel buildings. The results indicate that the maximum loads that develop in the tension braces and the columns can be predicted by a simple static analysis. The post-buckling strength of the braces and the flexural capacity of the beams both have a significant impact on the inelastic response of the structures. Chevron braces with the beams designed to remain elastic exhibited a more uniform inelastic demand over the building height and lower interstorey drifts. Such a system could classify under the Ductile Braced Frames category (R=3.0).

## INTRODUCTION

Inverted V, chevron braced frames are commonly used to resist seismic lateral loads in steel building structures. Under severe earthquake ground motions, the compression braces are expected to buckle and the beams are bent downward due to the combined action of the gravity loading and the tension acting braces. Unless the beams are designed to carry this net vertical load, a plastic hinge eventually forms at their mid-span before the tension braces develop their yield tensile capacity (Fig. 1). Such chevron braced frames exhibit a severely pinched hysteretic lateral response and, hence, can classify only under the Braced Frames with Nominal Ductility (R = 2.0) category, as defined in the CSA-S16.1 Standard (CSA, 1994). Past studies (Khatib, 1988; Remennikov and Walpole, 1998) have shown that the seismic behaviour of chevron braced frames can be improved when stronger beams are used.

In this paper, the behaviour of typical multi-storey chevron braced frames is examined through nonlinear dynamic analyses. A model is proposed to predict the maximum forces that will develop in the braces, the beams, and the columns of the braced frames. The effects of the building height and location are examined. The influence of the post-buckling strength of the braces and of the flexural strength of the beams is also investigated. For the latter, the performance of chevron braced frames with the beams designed to remain elastic has been studied when using an R factor equal to 3.0 in design.

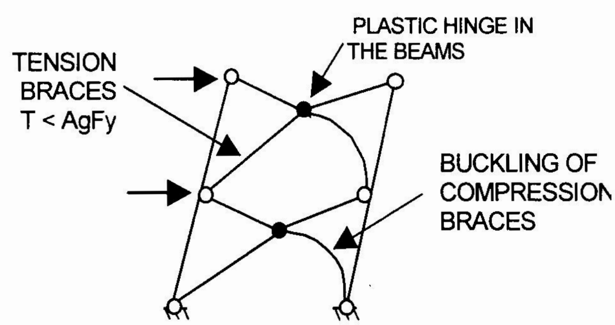


Figure 1 Inelastic response of chevron braced frames.

## DESCRIPTION AND DESIGN OF THE BUILDINGS

Buildings of two different sizes (900 $m^2$ and 1500 $m^2$) and four different heights (2, 4, 8 and 12 storeys) were investigated in this study. Two different sites were also considered: Vancouver, B.C. and Montreal, Qc. As shown in Table 1, only the small 2, 4, and 8 storey buildings were examined for Montreal. The storey heights are 3.8 m. For all buildings, one of the two bracing bents resisting the seismic loads in the east-west direction was studied (Fig. 2).

The building structures were designed according to the 1995 National Building Code of Canada (NBCC) (NRCC, 1995) and the S16.1-94 Standard. The following gravity loads were considered in the calculations: roof dead load of 1.2 kPa, floor dead load of 3.7 kPa, roof snow load of 1.48 kPa (Vancouver ) and 2.32 kPa (Montreal), and an occupancy floor live load of 2.4 kPa. The weight of the walls was assumed to be 1.2 kPa.

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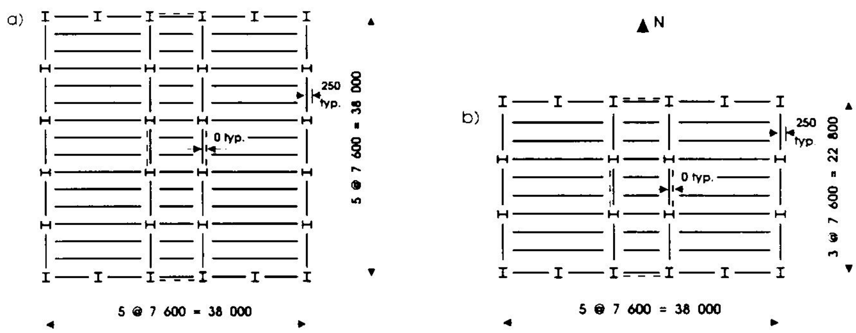


Figure 2 Plan view of the buildings: a) large size building (1 500 $m^2$); b) small size building (900 $m^2$).

The braced frames were designed for the seismic load, V:

$$V = vSIFW\left(\frac{U}{R}\right) \tag{1}$$

where v is the velocity ratio for the site (0.21 m/s for Vancouver, and 0.097 m/s for Montreal), S is the seismic response factor, I is the importance factor, F is the foundation factor, W is the seismic weight of the structure, U is a calibration factor (U = 0.6), and R is the force modification factor. The value of S varies with the fundamental period of the structure and the seismic zones for the sites. The fundamental periods prescribed in the NBCC provisions were equal to 0.25 s, 0.49 s, 0.98 s and 1.47 s for the 2, 4, 8, and 12 storey buildings, respectively. For all structures, the periods obtained from free vibration analysis exceeded these values and the S factor was reduced accordingly. The importance factor and foundation factor were taken as 1.0 (structure of normal importance on stiff soil). The seismic weight, W, included the floor and roof dead loads, the weight of the exterior walls, and 25% of the roof snow load. An R factor of 2.0 was used for the chevron braced frames with nominal ductility. Other values of R (1.0 and 3.0) were also considered in the study, as described later in the paper. The seismic load was then distributed over the height of the structures according to the NBCC static procedure. Concentrated lateral forces equal to 6.9 % and 10.4 % of the base shear had to be applied at the top of the 8 and 12 storey buildings, respectively. The computed fundamental periods of the buildings as designed are given in Table 1.

Table 1 Computed fundamental periods (s).

| Building height (size) | Montreal | | Vancouver | | |
|---|---|---|---|---|---|
| | R = 1.0 | R = 2.0 | R = 1.0 | R = 2.0 | R = 3.0 |
| 2 (900 $m^2$) | 0.39 | 0.39 | 0.33 | 0.38 | - |
| 4 (900 $m^2$) | 0.78 | 0.81 | - | - | - |
| 8 (900 $m^2$) | 1.61 | 1.78 | 1.26 | 1.59 | |
| 2 (1500 $m^2$) | - | - | 0.39 | 0.45 | - |
| 4 (1500 $m^2$) | - | - | 0.70 | 0.86 | - |
| 8 (1500 $m^2$) | - | - | 1.33 | 1.75 | 1.77 |
| 12 (1500 $m^2$) | - | - | 1.78 | 2.77 | - |

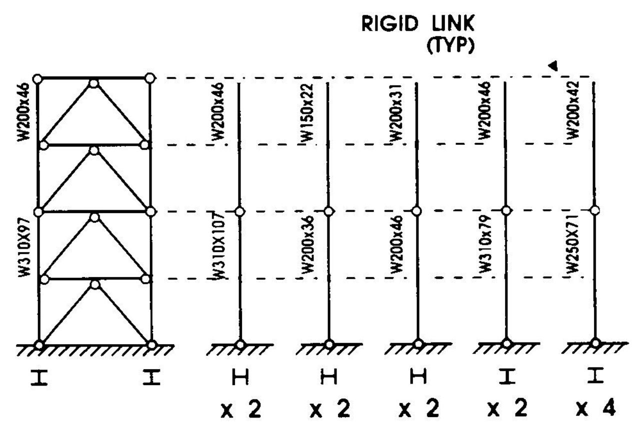


Figure 3 Typical analytical model.

## ANALYTICAL MODEL AND EARTHQUAKE RECORDS

The seismic performance of the buildings was obtained from nonlinear time history dynamic analyses performed using the Drain-2D computer program (Powell and Kanaan, 1973). The model (Fig. 3) included the bracing bent studied as well as the gravity columns that are stabilised by this bracing bent. The bracing members were modelled using the inelastic brace buckling element developed by Jain and Goel (1978). The post-buckling resistance of the braces, $C'_u$, was taken as prescribed in the S16.1 Standard:

$$C'_u = \frac{C_u}{1 + 0.35\lambda} \tag{2}$$

where $C_u$ and $\lambda$ are the ultimate compressive strength and the nondimensional slenderness of the braces, respectively. A Newmark constant acceleration integration scheme with a time step of 0.001 s was used throughout the study. P-$\Delta$ effects were considered in the calculations with 100% of the dead load and 50% of the live load applied to the structure. Rayleigh damping equal to 5% of critical damping in the first two modes was adopted in all analyses.

For each site, the structures were subjected to an ensemble of 10 ground motion time histories. For Vancouver, nine accelerograms were chosen from historical events that occurred along the western coast of North America and one record was a simulated time history generated by Atkinson and Beresnev (1998). For Montreal, the ensemble included six historical and four artificially generated ground motions. The eastern records were scaled to the peak ground acceleration for Montreal (0.18g). Other records for the Montreal and Vancouver regions were scaled to match the velocity ratio, v, of the site. Figure 4 shows the mean and mean plus one standard deviation 5% damping acceleration response spectra for each ensemble of normalised accelerograms. A description of the ground motions is given in Robert (1998).

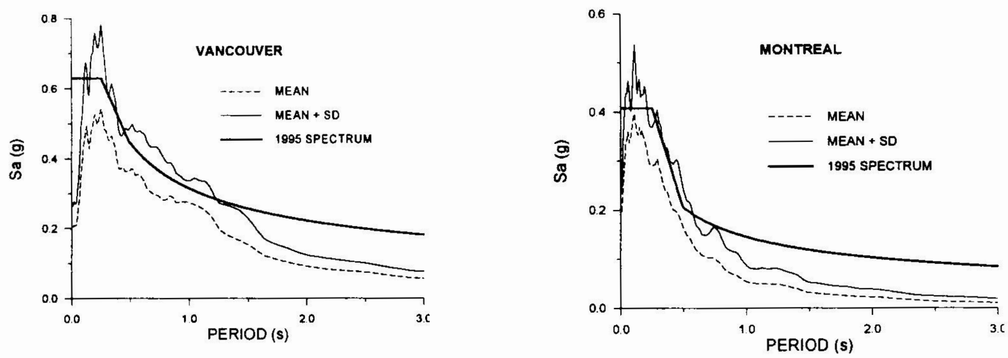


Figure 4 Design spectra and computed 5% damping acceleration response spectra of earthquake ground motions.

## CHEVRON BRACED FRAMES WITH NOMINAL DUCTILITY

Chevron braced frames were designed according to the S16.1 provisions for the Braced Frames with Nominal Ductility category. For these structures, the braces are first sized for the factored gravity and seismic loads. For buildings located in velocity-related zone 4 and higher, the braces must also meet the slenderness and width-to-thickness limits prescribed for braced frames in the Ductile Braced Frames category. Brace connections, beams and columns are then designed to carry the gravity loads together with the maximum forces likely to develop in the bracing members under strong ground motion. Beams must also be Class 1 sections, be continuous between the columns, and have adequate flexural resistance to carry their tributary loads without the support provided by the braces.

In this study, the bracing members were selected assuming a effective length factor of 0.8. For the Vancouver site, the special requirements for Ductile Braced Frames were applied. In absence of specific guidelines, the maximum axial loads used in the design of the beams and the columns were respectively taken equal to:

$$C_{beam_i} = \left(A_g F_y + C_u\right)_i \frac{\cos\theta_i}{2} \tag{3}$$

$$C_{column_i} = C_{W_i} + \sum_{x=i+1}^{n} \left(C_u \sin\theta\right)_x \tag{4}$$

In (3), it is conservatively assumed that both the tension and compression braces below the beam under consideration can reach simultaneously their tensile ($A_g F_y$) and compressive ($C_u$) capacity. In (4), $C_W$ is the axial load due to gravity loads at the level under consideration and n is the total number of floors. This equation assumes that all braces above level i can buckle at the same time, which is also conservative. It does not account, however, for the possibility that differences between tension and compression brace loads in a same storey produce net vertical loads at mid-span of the beams.

After the design was completed, a simple collapse mechanism of the braced frames was examined to better predict the maximum forces likely to develop in the braces, the beams, and the columns during a severe earthquake. Nonlinear dynamic analyses were then used to validate the prediction of this static model. This process was carried out for braced frames designed with an R factor of 2.0 and 1.0. The second series was studied to validate the model when oversized bracing members are used for meeting design criteria such as drift requirements, out-of-plane bending, etc.

The collapse mechanism is developed assuming the frame laterally deforms in its first mode, as shown in Fig.1 for a two-storey building. Figure 5 shows the brace forces acting in the same two-storey frame example. In this model, it is also assumed that all braces reach their maximum loads ($T_{max}$ and $C_u$) simultaneously. However, the maximum load in the tension braces, $T_{max}$, can be lower than the brace yield load. With this model, members forces can be determined once the sizes of the braces and the beams are known. The force $T_{max}$ is first determined based on the capacity of the beam. This force would be used for the design of the brace connections. Thereafter, the beams and column axial loads can be evaluated.

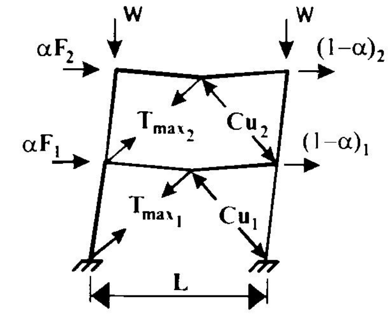

Figure 5 Maximum brace forces.

At any given level, the compressive load and the bending moment in the beam are respectively equal to:

$$C_{beam_i} = (T_{max}\cos\theta)_{i+1} + \alpha F_i \quad , \text{with: } F_i = (T_{max} + C_u)_i \cos\theta_i - (T_{max} + C_u)_{i-1}\cos\theta_{i-1} \tag{5}$$

$$M_{beam_i} = (T_{max} - C_u)_i \sin\theta_i \frac{L}{4} + M_{W_i} \tag{6}$$

In (5), $\alpha$ is the fraction of the lateral loads which is applied to the tension brace side of the bracing bent (= 0.5 in this study). In (6), $M_W$ is the bending moment due to the gravity loads acting on the beam (not shown in Fig. 5). Starting at the top level, the value of $C_{beam}$ and $M_{beam}$ can be obtained as a function of $T_{max}$ in the brace under the roof beam. This brace force can then be obtained from the beam-column interaction equation for Class 1 sections as applied to the top floor beam:

$$\left(\frac{C}{AF_y} + 0.85\frac{M}{ZF_y}\right)_{beam} = 1.0 \tag{7}$$

In this equation, A and Z are the cross section area and the plastic modulus of the beam, respectively. The same procedure can then be used to determine $T_{max}$ at the lower levels. Thereafter, the maximum column loads, $C_{max}$ can be determined by vertical equilibrium, including the net vertical force imposed at mid-span of the beams by the bracing members:

$$C_{max_i} = C_{W_i} + \sum_{x=i+1}^{n}(C_u \sin\theta)_x + \sum_{x=i+1}^{n}(T_{max} - C_u)_x \frac{\sin\theta_x}{2} \tag{8}$$

Figure 6 shows the mean plus one sigma (M+SD) value of the peak axial loads computed in the braces, $T_{dyn}$, and in the columns, $C_{dyn}$ for the 4, 8, and 12-storey, 1 500 m$^2$ buildings located in Vancouver. These forces are normalised with respect to $T_{max}$ and $C_{max}$, respectively. The gravity induced column loads, $C_W$, was subtracted from both $C_{max}$ and $C_{dyn}$ before computing the ratio of these two column loads. A normalised brace or column load equal to 1.0 indicates that the prediction of the maximum seismic induced forces is adequate.

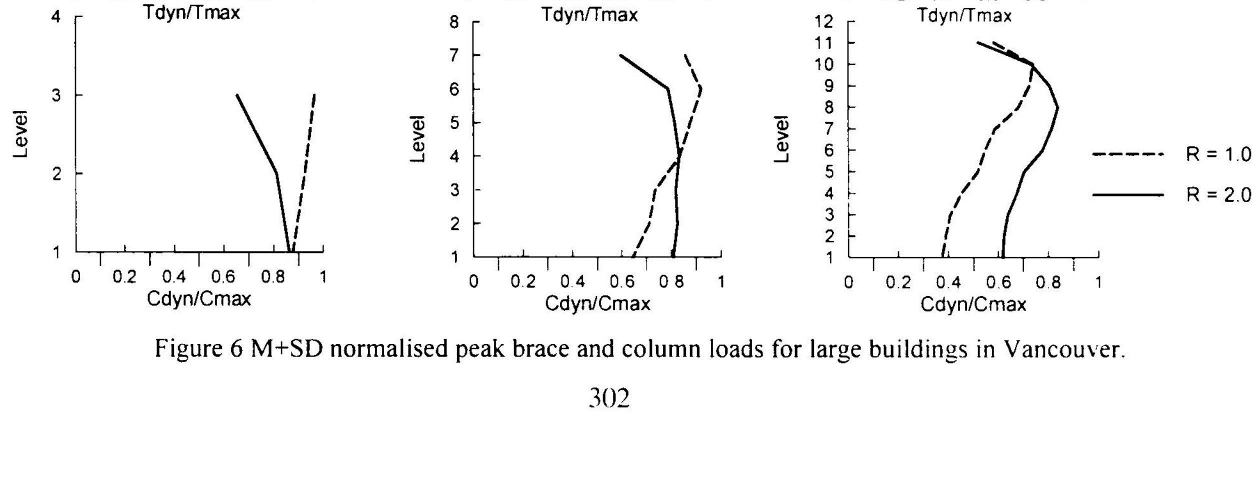

Figure 6 M+SD normalised peak brace and column loads for large buildings in Vancouver.

The static model predicts very well the maximum brace and column loads for the 4-storey buildings, regardless of the brace sizes. Excellent correlation has also been obtained for the 2-storey structures. For taller buildings, the contribution of the higher modes becomes more important and the prediction are more conservative, especially at the bottom floors of the structures or when oversized braces are used. For the 8-storey buildings, however, the method is still considered adequate to predict the behaviour of frames with minimum and oversized braces. Similar results were obtained for the small size buildings in Vancouver, which indicates that the building size has no effects on the relative maximum member forces.

The influence of the site is examined in Fig. 7 for the small 8-storey building with minimum brace sizes (R = 2.0). The M+SD normalised peak brace and column loads for Montreal are significantly lower than predicted by the static model. This can be attributed to the fact that the structures in Montreal are more flexible than in Vancouver (seismic loads are lower in Montreal), and that the NBCC design spectra for Montreal is very conservative for structures with long periods, as shown in Fig. 4.

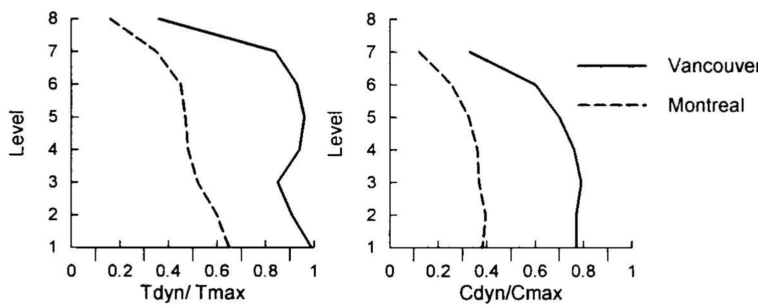


Figure 7 Influence of the site on the peak brace and column loads.

In the static model, the braces were assumed to maintain their compressive strength $C_u$. This is consistent with current S16.1 provisions for braced frames with nominal ductility which do not require to account for the degradation of the compressive resistance of the braces. The conservatism observed in the prediction of the maximum tension brace load is partly due to this assumption because some reduction in the brace compressive strength was included in the analytical model (Eq. 2). On the other hand, the degradation of the compression strength of braces can be more severe than given by Eq. (2). For instance, AISC (1997) suggests to use 30% of the factored compressive strength for the post-buckling capacity, $C'_u$, of bracing members. Such a low capacity can lead to higher inelastic deformations in the braces and, thereby, larger inelastic storey drifts in the structure. The 8-storey braced frame located in Vancouver was re-analysed with $C'_u$ equal to 0.3 $C_u$ for all braces. In Fig. 8, the computed M+SD peak interstorey drifts are compared to those obtained with the previous analytical model. Significant concentration of inelastic demand is observed in the frame with the lower brace post-buckling compressive strength. This suggests that a building height limitation could be appropriate for multi-storey chevron braced frames with nominal ductility designed according to current S16.1 specifications.

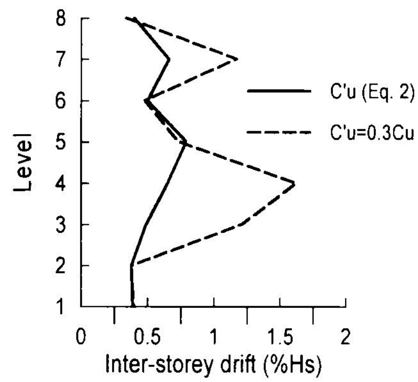


Figure 8 Influence of the post-buckling resistance of the braces.

## CHEVRON BRACED FRAMES WITH STRONG BEAMS

In order to evaluate the benefit of using stronger beams on the performance of chevron braced frames, the 8-storey braced frame for the large size building located in Vancouver was re-designed. In this strong beam design, the beams were sized to carry the gravity loads together with the beam axial and bending forces that develop when the tension braces yield and the compression braces develop a post-buckling strength equal to 0.3 $C_u$. Therefore, in this design, $T_{max}$ corresponds to the tensile yield resistance of the braces. An R factor of 3.0 was also used in this design to investigate the possibility of classifying this strong beam chevron braced frame system in the Ductile Braced Frames category.

Figure 9 compares the behaviour of the strong beam braced frame to that of the R = 2.0 chevron braced frame with nominal ductility (weak beam design). For both designs, $C'_u$ was taken equal to 0.3 $C_u$ in the analytical models. The M+SD values of the peak ductility demand in the tension braces, peak interstorey drifts, and peak normalised tension brace and column axial loads are presented in the figure. As expected, higher tension forces developed in the braces in the stronger beam design frame. This resulted in a more uniform, and generally lower, interstorey drift demand over the building height, which represents a significant improvement. For the strong beam design, the predicted values of $T_{max}$ agree well with the results of the dynamic analysis only at the first and sixth floors, where the braces developed their tensile yield resistance. The prediction of the maximum column loads in the strong beam design appears to be also more conservative than in the frame with the weak beams, most likely because $T_{max}$ and $C_u$ do not occur simultaneously in the strong beam braced frame. For the weak beam design, the difference between $T_{max}$ and $C_u$ is small and both brace forces may occur nearly at the same time.

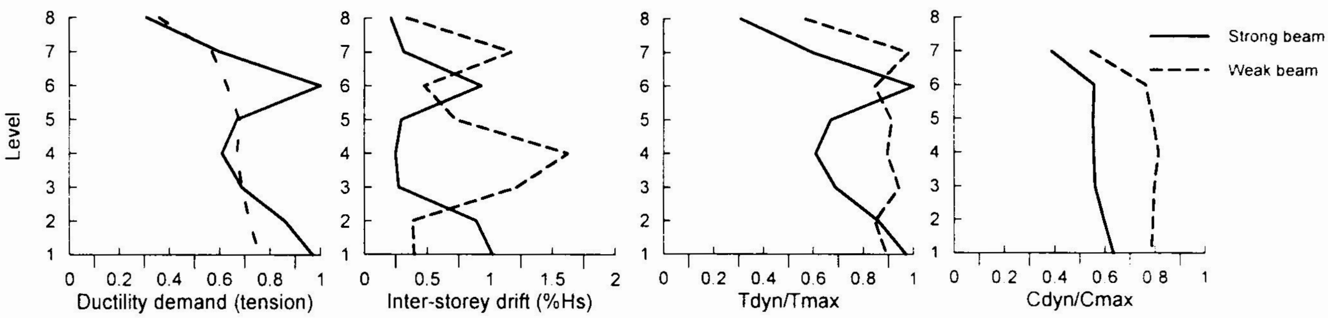


Figure 9 M+SD of peak brace ductilty, interstorey drifts, normalised brace tension loads, and normalised column loads.

## CONCLUSIONS

Values for the maximum member forces likely to develop in chevron braced frames subjected to strong seismic shaking have been proposed based on a first mode response collapse mechanism of the structure. Nonlinear dynamic analyses were performed on typical chevron braced frames to validate these values and examine the seismic performance of this braced frame system. The predicted values were found to be adequate for structures up to 8 storeys located in Vancouver, even when oversized braces were used. For taller buildings or structures located in Montreal, the static model tends to overestimate the member forces. This study also confirmed that multi-storey chevron braced frames can be prone to the formation of soft storey mechanisms. This undesirable behaviour could be prevented by imposing building height limits for this bracing system or by using strong beams to mobilise the tensile yield resistance of the braces. Preliminary studies indicate that chevron bracing with such strong beams could perform satisfactorily when designed with an R factor of 3.0.

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