Seismic Behaviour and Design of Chevron Multi-tiered Concentrically Braced Frames

Christophe Comeau[1(✉)], Pablo Cano[2], Ali Imanpour[2], and Robert Tremblay[1]

[1] Structures Research Group, Department of Civil, Geological and Mining Engineering, Polytechnique Montreal, Montreal, QC, Canada
christophe.comeau@polymtl.ca

[2] Department of Civil and Environmental Engineering, University of Alberta, Edmonton, AB, Canada

Abstract. This paper investigates the possibility of using the chevron bracing configuration for multi-tiered concentrically braced frames subjected to seismic excitations. A prototype two-tiered braced frame part of a single-storey building structure was designed using three different brace force scenarios for the roof beam and the intermediate strut. Columns were designed to resist the bending expected at the maximum anticipated storey drift. The lateral response of the frame was examined through nonlinear static and dynamic analyses. For all cases studied, frame lateral deformations tend to concentrate in the first tier, where brace buckling initiated first, due to the reduced tier lateral stiffness in the brace post-buckling range. The flexural action in the intermediate struts was engaged when a reduced force was used for tension-acting braces in design, limiting nonlinear response in braces. Finally, the frames exhibited stable inelastic response with limited residual deformations, as a result of the re-centring capacity provided by the strut acting in flexure.

Keywords: Multi-tiered chevron braced frame · Brace buckling · Beam yielding

1 Introduction

Multi-tiered braced frames (MT-BFs) consist of concentrically braced frames with two or more bracing panels stacked over the storey height. This configuration is frequently adopted for tall single-storey buildings when full height diagonal members become impractical or less effective. As shown in Fig. 1a for a two-tiered X-braced frame, bracing members in MT-BFs are shorter, which generally permits smaller (lighter) brace sections for a given design storey shear.

MT-BFs however exhibit complex inelastic seismic response that must be considered in design to achieve satisfactory performance [1–3]. The inherent difference between brace compressive and tension resistances results in unbalanced horizontal brace forces at brace-to-column intersecting points, which imposes in-plane flexural demands on the columns. Brace yielding in tension also initiates in the critical (weakest) tier, and subsequent brace inelastic response tends to concentrate in that tier

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as its horizontal shear capacity gradually reduces. This behaviour can impose excessive ductility demand on the braces leading to brace fracture in the critical tier.

Special design requirements have been introduced in Canada to achieve satisfactory MT-BF seismic response [4]. Although not identical, the essence of these requirements is that horizontal struts must be provided between tiers to create a direct load path between tension-acting braces and, thereby, eliminate resultant unbalanced brace forces applied to the columns; and columns must be designed as beam-columns to resist axial compression and moments resulting from braces reaching their probable resistances when inelastic lateral deformations concentrate in the critical tier (Fig. 1b).

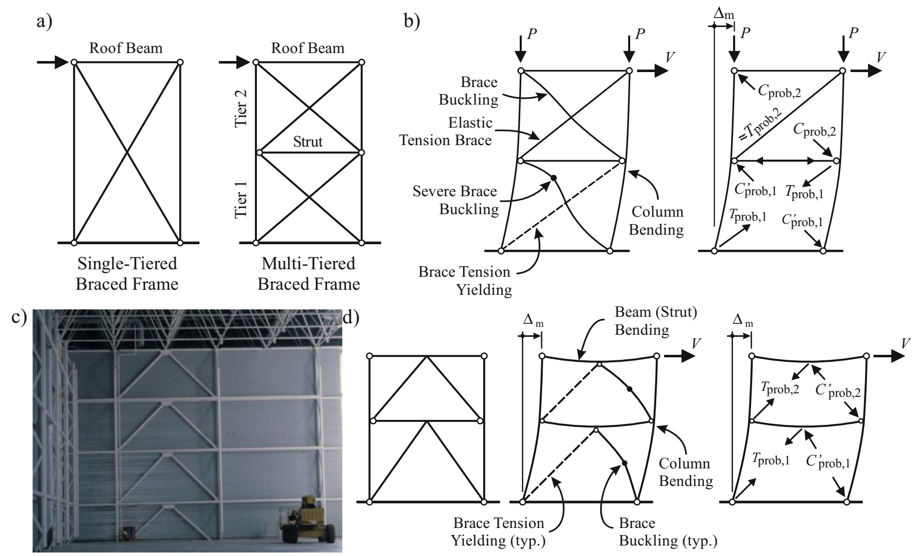

Fig. 1. a) Single-tiered *vs.* two-tiered X-braced frame; b) Expected inelastic behaviour and brace forces for column design (T_{prob} probable tensile resistance; C_{prob} probable compressive resistance; C'_{prob} probable post-buckling compressive resistance); c) Four-tiered chevron braced frame [Courtesy of CISC]; d) Chevron MT-BF (expected inelastic behaviour and brace forces for beam & strut design).

Past studies have focused on MT-BFs with X-bracing configuration and the new design provisions were essentially developed for that configuration. In practice, chevron (inverted-V) bracing shown in Fig. 1c is also used in MT-BF applications. In this configuration, struts are already present between each tier, and current code provisions would require that beams and struts be designed to resist axial loads and moments from the maximum unbalanced brace force scenario illustrated in the figure. As soon as brace buckling occurs in a tier with chevron bracing, the tier horizontal shear stiffness and the rate of increasing load in the tension brace heavily depends on the flexural stiffness of the beam or strut in that tier. Strut and beam stiffness would therefore influence the frame nonlinear response, namely, vertical distribution of brace buckling, brace tension yielding, tier drifts, and, thereby, the amplitude of the in-plane bending demand on the columns. In the Canadian steel design standard [4], it is permitted to consider a reduced brace tension load (=0.6 T_{prob}) for the design of the beams of chevron braced frames up to 4-storeys, thereby allowing beam flexural yielding in lieu of brace tension yielding [5]. This approach results in smaller and more flexible beams, while reducing column axial loads from braces. Preventing brace tension yielding is also seen as effective for reducing the likelihood of low-cycle fatigue fracture of HSS braces. The adequacy of this modified yielding mechanism was experimentally verified [6, 7]. Adopting this design scheme would result in a more cost-effective MT-BF, but weaker and more flexible beams and struts can result in excessive localized drifts, adversely affecting the frame response and column stability.

This article presents a preliminary study that examines the seismic response of a prototype two-tiered chevron braced frame for which beam, strut and columns are designed using three different brace tension load levels. Additionally, the use of intermediate struts designed using wide flange and HSS profiles is investigated. The sample frame is assumed to be part of an industrial building located in southwest British Columbia, Canada. The behaviour of the structures is first evaluated using nonlinear static analysis (NLS). Nonlinear response history analysis (NLRHA) is finally performed to estimate the seismic demands and verify the beam design approaches examined here.

2 Design of Chevron Multi-tiered Braced Frames

2.1 Prototype Building and Seismic Loading

A two-tiered Chevron CBF part of a rectangular, 9 m-tall, single-storey building having 39 m by 84 m plan dimensions was selected. The building is located on a site class E (soft soil) in the vicinity of Vancouver, BC. The design was performed in accordance with the 2015 NBC [8]. The gravity loads include roof dead load (D) of 1.35 kPa, snow load (S) of 1.64 kPa, and live load (L) of 1.0 kPa. A dead load of 1.0 kPa was considered for the exterior wall cladding. The resistance to lateral loads is provided by four identical two-tiered chevron braced frames in each principal direction of the building. One of the braced frames shown in Fig. 2a is studied here. For the braced frame, the tributary seismic weight W is equal to 1718 kN, including roof and wall dead loads and 25% of the roof snow load. The building is of the normal risk category, with an importance factor I_E = 1.0, and the braced frames are of the Type MD (Moderately Ductile) category for which ductility- and overstrength-force modification factors, R_d and R_o, are equal to 3.0 and 1.3, respectively. The building fundamental period was taken equal to the analytical period equal to 0.42 s, which resulted in a design base shear $V_d = 0.261(W) = 449$ kN per frame.

2.2 Frame Design

The frame design was performed using the applicable provisions of [4]. The bracing members are ASTM A1085 HSS members (F_y = 345 MPa) selected to resist the axial compression force from the design seismic load plus a concomitant roof gravity load of 2.78 kN/m: HSS 127 × 127 × 7.95 in Tier 1 and HSS 127 × 127 × 6.35 in Tier 2. The probable resistances of these two members, as determined with probable yield strength $R_y F_y$ = 460 MPa and an effective length factor K = 0.85, are, respectively: T_{prob} = 1668 and 1362 kN, C_{prob} = 618 and 638 kN, and C'_{prob} = 334 and 272 kN. A comparison of the horizontal storey shear associated with the brace compressive load C_{prob} in each tier showed that brace buckling occurs first in Tier 1 (V = 709 kN *vs.* 839 kN).

The roof beam and intermediate strut were designed for two compression brace conditions: at brace buckling ($C = C_{prob}$) and after brace buckling $\left(C = C'_{prob}\right)$. For the first condition, illustrated in Fig. 2b, brace tension forces T_{br} in both tiers were obtained assuming brace elastic response. For the second condition, three different values were used for the brace tension load: T_{prob}, $0.6T_{prob}$, and C_{prob}, as illustrated in Fig. 2c. The first case ($T_{br} = T_{prob}$) aims at constraining inelastic response in the bracing members and develop the yielding mechanism of Fig. 1d specified for chevron braced frames in current code provisions. In cases 2 and 3, reduced brace tension forces are used to obtain more cost-effective designs by reducing the size of the roof beam and intermediate strut. In these cases, plastic hinging is expected to form in the roof beam and strut, in lieu of brace tension yielding in both tiers. In all three cases, column shears V_{c1} and V_{c2} required for horizontal equilibrium at the strut level with the assumed brace forces are calculated and used to determine the concomitant strut axial loads. In all designs, the roof beam and strut were assumed to be pin-connected to the columns.

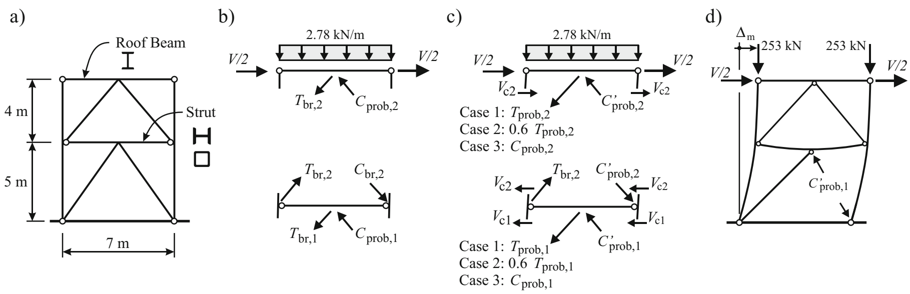

Fig. 2. a) Two-tiered braced frame studied; b) Brace forces for beam and strut design at brace buckling; c) Brace forces for beam and strut design after brace buckling; and d) Brace inelastic response assumed to determine column moments.

The roof beam and strut are ASTM A992 wide flange members with F_y = 345 MPa. At the roof level, it was assumed that continuous lateral support was provided by the roof steel deck at the beam top flange and that point bracing at the bottom flange. Out-of-plane bracing is not possible for the strut and stability in the horizontal plane was ensured by orienting the cross-section such that strong axis bending occurs in that plane (see Fig. 2a). For the third design case ($T_{br} = C_{prob}$), an alternative HSS member solution was also examined for the strut. As plastic hinging was expected in design cases 2 and 3, the selected shapes for these cases had to satisfy Class 1 (plastic design) cross-section requirements. The columns are ASTM A992 wide flange members oriented such that strong axis bending occurs out-of-plane. They were designed for the combined axial compression load and bending conditions that prevailed in the first tier. For the first design case with $T_{br} = T_{prob}$ in both tiers, axial compression from roof gravity loads (253 kN) and from assumed brace forces was considered together with in-plane bending due to the column shear V_{c1} in Fig. 2c. For the second and third design cases, axial load and in-plane moments in the first-tier column segment were obtained from an analysis in which brace buckling was assumed to occur only in the critical first tier ($C_{br,1} = C_{prob,1}$) and the structure was laterally deformed up to a roof displacement of Δ_m, the maximum anticipated value including inelastic response that is specified in NBC ($\Delta_m = R_d R_o \Delta_d$, where Δ_d is the elastic displacement). This behaviour is illustrated in Fig. 2d and the roof drifts, Δ_m/h_n with h_n = 9 m, are given in Table 1. It assumes that all frame members remain elastic after brace buckling in the first tier, which is confirmed by the force demand obtained from the analysis. In this framing configuration, brace buckling in one tier significantly reduces the frame lateral stiffness as the lateral stiffness becomes essentially governed by the bending stiffness of the intermediate strut (or beam) in that tier after brace buckling, resulting in reduced force demands in the remaining members, which can highly impact the seismic behaviour of the system as will be discussed in the next section.

Table 1 presents the member sizes selected for each design case, together with the steel tonnage per frame, building period T_1 and anticipated storey drift Δ_m. As shown, using lower brace tension loads for the design of the roof beam and strut leads to significant reductions in required steel. T_1 and Δ_m are not much affected as they are based on elastic response that essentially depend on the brace axial stiffness.

Table 1. Properties of the frames studied.

Design case	Roof beam	Strut	Columns	Steel (t)	T_1 (s)	$\Delta_m h_n$ (%)
$T_{br} = T_{prob}$	W610 × 155	W360 × 592	W310 × 107	7.75	0.42	0.48
$T_{br} = 0.6\ T_{prob}$	W530 × 92	W360 × 314	W250 × 58	4.49	0.43	0.51
$T_{br} = C_{prob}$	W460 × 74	W310 × 179	W250 × 58	3.41	0.43	0.52
$T_{br} = C_{prob}$	W460 × 74	HSS305 × 305 × 13	W250 × 58	2.96	0.43	0.52

3 Seismic Response Evaluation

3.1 Numerical Model

The nonlinear numerical model of the Chevron braced frames was constructed using the *OpenSees* program [9]. The modelling assumptions and techniques are based on the recommendations by Imanpour et al. [3]. Only the modelling assumptions specific to chevron braced frames are described here. The yield stress, F_y = 345 MPa and 460 MPa were assigned to columns and HSS members, respectively. The wide flange roof beams and struts were defined with F_y = 385 MPa. Columns were pinned at their base. The roof beam was constrained in the out-plane direction at each nodal point to represent the restraints provided by the steel deck and lateral support provided at mid-span. The strut was assigned a sinusoidal out-of-plane imperfection with a maximum amplitude of 0.001 times its effective length, i.e. the length between end connections. The braced frame was analyzed using NLS (pushover) and NLRHA methods. The pushover analysis was performed by applying a horizontal drift of 2.5%. For the dynamic analysis, a suite of 15 site specific ground motion (GM) records was applied at the base of the frame.

3.2 Nonlinear Static Analysis Results

The results of the pushover analysis for the frames studied are presented in Fig. 3. The analysis results reveal a stable lateral response for all four design cases. In all cases, the inelastic drift is limited to the critical tier (bottom tier). Furthermore, it was found that the maximum frame base-shear decreases as the design force of the tension brace is reduced. The maximum base shear, within the range of the maximum anticipated design storey drift, Δ_m, occurs at the instant when the compression brace in the critical tier reaches its buckling load. Moreover, yielding of the tension-acting braces does not occur in any of the four designs when the frame roof displacement reaches Δ_m. Only the frame designed using a tension force $T_{br} = T_{prob}$ was able to yield the tension brace at three times Δ_m. In the other cases (i.e., $T_{br} < T_{prob}$ or $T_{br} = 0.6T_{prob}$ and $T_{br} = C_{prob}$), the flexibility and the inelastic flexural mechanism developed in the intermediate strut prevents yielding in tension-acting braces. For these cases, flexural plastic hinging was observed at strut mid-span at large story drifts ($\Delta >> \Delta_m$). Relatively small in-plane moment was observed in the columns of design cases where the compression brace in the non-critical second tier buckles (i.e., $T_{br} = T_{prob}$ and $T_{br} = 0.6T_{prob}$).

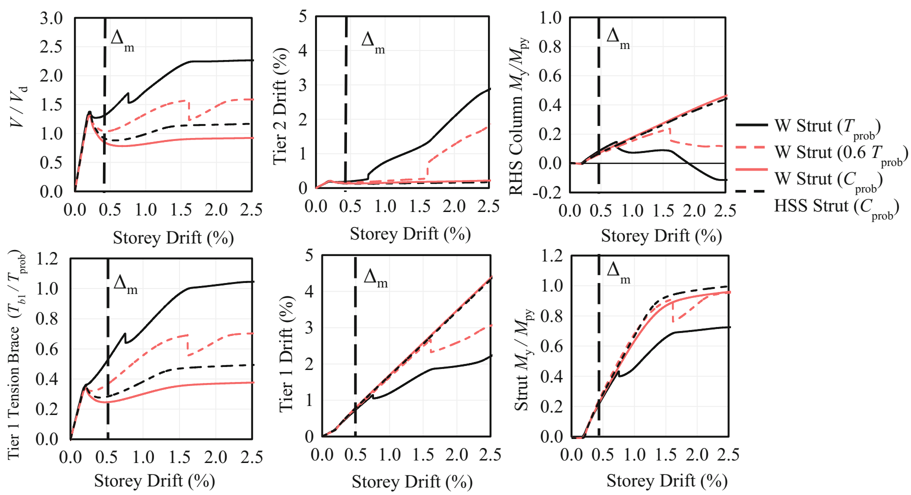

Fig. 3. Lateral response of two-tiered chevron braced frame using pushover analysis.

However, the columns of other design cases where $T_{br} = C_{prob}$ experienced higher moment demands in particular at large storey drifts, reaching approximately $0.45M_{py}$ at 2.5% storey drift. Compression brace buckling in the non-critical second tier is characterized by a sharp reduction in base shear followed by a gradual increase as the lateral stiffness of the second tier is controlled by its tension brace, as can be observed at 1.6% storey drift for the frame designed with $T_{br} = 0.6T_{prob}$.

3.3 Nonlinear Response History Analysis Results

The statistics of the results obtained from the NLRHA of the four chevron braced frame studied under the 1989 Loma Prieta GM are given in Table 2. The frame responses from NLRHA match well to those of obtained from NLSA. No column buckling or frame collapse was observed in dynamic analyses. Storey drift demands are relatively higher in the frames designed assuming $T_{br} = 0.6T_{prob}$ and $T_{br} = C_{prob}$ in the critical tier, which is attributed by a lower lateral stiffness compared to the frame designed using $T_{br} = T_{prob}$. Consistent with the design strategy, the frame lateral deformation tends to concentrate in the critical first tier, similar to the NLSA results.

Referring to in-plane moments in the struts, flexural plastic hinge developed in the struts designed assuming $T_{br} = C_{prob}$ regardless of the cross-section shape. Except the design where $T_{br} = T_{prob}$, the roof beams did not experience flexural moments demands as a vertical unbalanced load was not developed since neither buckling nor yielding of the braces in the non-critical tier occur on the frames designed using $T_{br} = 0.6T_{prob}$ and $T_{br} = C_{prob}$. Similar to the NLSA, relatively higher in-plane moment was observed in the columns of the design cases where $T_{br} = C_{prob}$, compared to the moments induced in the columns of other two cases. The frames designed using the brace force of $T_{br} = 0.6T_{prob}$ and $T_{br} = C_{prob}$ resulted in columns reaching higher in-plane bending moment than the assumed design value due to: 1) larger storey drift demands exceeding the maximum expected storey drift, and 2) the inability of the non-critical tier to trigger brace buckling.

The comparison between the results obtained for the frames designed assuming $T_{br} = C_{prob}$ show that both exhibit nearly the same response and seismic demands; however, the frame with an HSS strut is 15% lighter (see Table 1).

Table 2 also presents the ratios of bending moments obtained from NLRHA to the corresponding design value. For the frames designed using $T_{br} < T_{prob}$, the column bending moments obtained from NLRHA surpass the design value, due to an assumed design storey displacement Δ_m smaller than the maximum storey displacement from NLRHA.

Table 2. Seismic behaviour of MT-BFs under the 1989 Loma Prieta earthquake record.

	Drift (%)			Brace max. forces				
Design case	Storey	Tier 1	Tier 2	Tier 1 T_1/$T_{prob,1}$	Tier 2 T_2/$T_{prob,2}$	Column My/Mpy [M_y/M_d*]	Strut My/Mpy [M_y/M_d]	Roof beam Mx/Mpx [M_x/M_d]
W Strut (T_{prob})	1.0	1.2	0.7	0.72	0.71	0.16 [0.83]	0.51 [0.39]	0.43 [0.48]
W Strut ($0.6T_{prob}$)	1.1	1.9	0.2	0.60	0.37	0.19 [1.66]	0.86 [0.66]	0.00 [0.00]
W Strut (C_{prob})	1.8	3.0	0.2	0.37	0.35	0.33 [2.54]	1.03 [0.80]	0.00 [0.00]
HSS Strut (C_{prob})	1.5	2.5	0.2	0.47	0.37	0.27 [2.10]	1.02 [1.36]	0.00 [0.00]

* Design value of the corresponding bending moment

4 Conclusions

The results of the pushover analyses at the design storey drift match well with those obtained using the NLRHA. The results of NLRHA confirmed that inelastic response of the frames designed using a reduced tension force is generally limited to brace buckling in the critical tier where brace buckling occurs first. Flexural plastic hinging was observed in the struts when the tension-acting brace force was set equal to compressive brace resistance. In these frames, brace yielding in tension was not observed nor was flexural bending in the roof beam. This behaviour is due to the fact that frame lateral stiffness significantly reduces after brace buckling, as it becomes governed by the bending stiffness of the intermediate strut in the critical tier. Such reduced stiffness diminishes the axial force demand in the remaining frame members. Furthermore, this reduced frame stiffness resulted in peak storey drifts largely exceeding design predictions based on the equal displacement principle, as well as high column in-plane bending moments due to uneven tier drifts. Although a heavier design case associated with tension-acting brace force equal to 0.6 times the probable tensile resistance was able to engage the flexural action in the strut with a lower in-plane bending induced in the columns compared to the two designs with the tension-acting brace force set equal to compressive brace resistance. Nevertheless, all frames studied exhibited stable seismic performance with limited residual displacements due to the recentering effects of the elastic beam, strut and braces acting in tension. This result suggests that chevron bracing with reduced design brace tension loads could represent a cost effective configuration for multi-tiered applications. A potential design strategy could involve selecting bracing members such that a critical tier is clearly defined. The strut in that tier would remain elastic up to the expected tier drift. Moreover, this design method should account for the concomitant out-of-plane bending induced in the columns due to strut and brace out-of-plane deformations.

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References

1. Imanpour A, Tremblay R, Davaran A, Stoakes C, Fahnestock L (2016) Seismic performance assessment of multi-tiered steel concentrically braced frames designed in accordance with the 2010 AISC Seismic Provisions. J Struct Eng 142(12):04016135
2. Imanpour A, Tremblay R, Stoakes FL, C, (2016) Analysis and design of two-tiered steel braced frames for enhanced in-plane seismic response. J Struct Eng 142(11):04016115
3. Imanpour A, Tremblay R (2016) Seismic design and response of steel multi-tiered concentrically braced frames in Canada. Can J Civ Eng 43(10):908–919
4. (2019) CSA. Design of steel structures, CSA S16. Canadian Standard Association, Toronto
5. Tremblay R, Robert N (2001) Seismic performance of low- and medium-rise chevron braced steel frames. Can J Civ Eng 28(4):699–714
6. Sen AD et al (2016) Experimental investigation of chevron concentrically braced frames with yielding beams. J Struct Eng 142(16):04016123
7. Roeder CW et al (2019) Effect of beam yielding on chevron braced frames. J Constr Steel Res 159:428–441
8. (2015) NRCC. National Building Code of Canada 2015, 14th (edn) National Research Council of Canada, Ottawa
9. Mckenna F, Fenves GL (2004) Open System for Earthquake Engineering Simulation (OpenSees). Pacific Earthquake Engineering Research Center (PEER), U of California, Berkeley