Seismic Response of Steel Multi-Tiered Eccentrically Braced Frames

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ABSTRACT

Steel multi-tiered braced frames are commonly used as the lateral load resisting system of tall single-story buildings such as convention centers, sports facilities, warehouses or industrial applications. In multi-tiered configurations, the frame height between the ground and roof levels is divided into multiple bracing panels because the application of a single braced panel with long braces is neither practical nor economical. Various braced frame systems including concentrically braced frames (CBFs), bucking restraint braced frames (BRBFs) and eccentrically braced frames (EBFs) can be used with the multi-tiered configuration. Although multi-tiered concentrically braced frames are often preferred in practice, multi-tier eccentrically braced frames could also represent a cost-effective solution as a highly ductile and stable inelastic response exhibited by the yielding mechanisms in link beams can be achieved in multi-tiered EBFs. However, there has been no research into the seismic response of such frames and it is currently prohibited by the Canadian steel design standard. This paper aims to investigate the seismic response of steel multi-tiered EBFs. A two-tiered EBF was first designed in accordance with the current provisions prescribed by the Canadian steel design standard for conventional EBFs. A nonlinear numerical model of the frame was then developed. The lateral performance of the frame was finally evaluated using a nonlinear static analysis method. Two lateral support scenarios were considered for the link at the intermediate strut level. The results of the numerical simulations showed that the nonlinear deformations of the frame are not evenly distributed along the frame height. Lateral instability of the link at the strut level was observed when no lateral support is provided, which led to the column out-of-plane buckling.

Keywords: multi-tiered eccentrically braced frames, numerical simulations, buckling, seismic design.

INTRODUCTION

Steel multi-tiered braced frames (MT-BFs) are commonly used as the lateral load resisting system of tall single-story buildings such as convention centers, sports facilities, warehouses or industrial applications. In the multi-tiered configuration, the frame height between the ground and roof levels is divided into multiple bracing panels as the application of a single braced panel with long braces is neither practical nor economical. The brace length is reduced in multi-tiered braced frames, allowing smaller braces to be selected. Furthermore, the columns can be assumed to be laterally braced in the plane of the frame as a result of horizontal struts typically placed between braced panels to avoid the unsatisfactory K-type braced frame response. This will reduce the buckling length of the braced frame columns in the plane of the frame. From the seismic design perspective, the selection of smaller braces can efficiently satisfy stringent brace slenderness limits prescribed by seismic design provisions. Furthermore, reduced brace sizes will result in lower seismic force demands on adjacent members and connections.

Wide-flange steel columns are typically used in the design of multi-tiered braced frames. Columns are considered laterally unbraced between the ground and floor levels in the out-of-plane direction, which results in a buckling length equal to the full frame length. To achieve an economical column design, the columns are oriented such that out-of-plane bending due to the wind load occurs about the column strong axis. Various braced frame systems including concentrically braced frames (CBFs), buckling-restraint braced frames (BRBF) and eccentrically braced frames (EBFs) can be used with the multi-tiered configuration. Although multi-tiered concentrically braced frames are often preferred in practice because of ease of design and fabrication, multi-tiered eccentrically braced frames could also represent a cost-effective solution for such single-storey buildings as a multi-tiered EBF can offer a highly ductile and stable inelastic response exhibited by the yielding mechanisms in link beams, while providing a favorable combination of strength and stiffness. Two examples of chevron-type steel MT-EBFs are shown in Fig. 1.

The majority of research studies over the past decade has been limited to MT-CBFs including understanding the seismic response and developing seismic design procedure [1–3]. Several analytical and experimental studies confirmed that large in-plane flexural bending moments induce in the columns of MT-CBFs due to uneven distribution of inelastic deformations developed after brace tension yielding in any one of the braced panels along the height of the frame while other braced tiers remain essentially elastic. Such response led to the concentration of inelastic deformations in the tier where brace tension yielding takes place first, which is also referred to as the critical tier. Column moment demands if not considered in design may lead to column plastic hinging and eventually instability of the column. The results of nonlinear response history analyses conducted in the past confirmed that column instability occurs in the plane of frame as a result of combined axial force and in-plane flexural moment demands, which led to flexural-torsional buckling of the column under major earthquakes [3]. Additionally, it was shown that large inelastic deformations induced in one of the tiers due to uneven yielding of the bracing members may induce large deformation demands on the braces of the critical tier, which may cause premature fracture of bracing members due to low-cycle fatigue [2,4].

(a) (b)

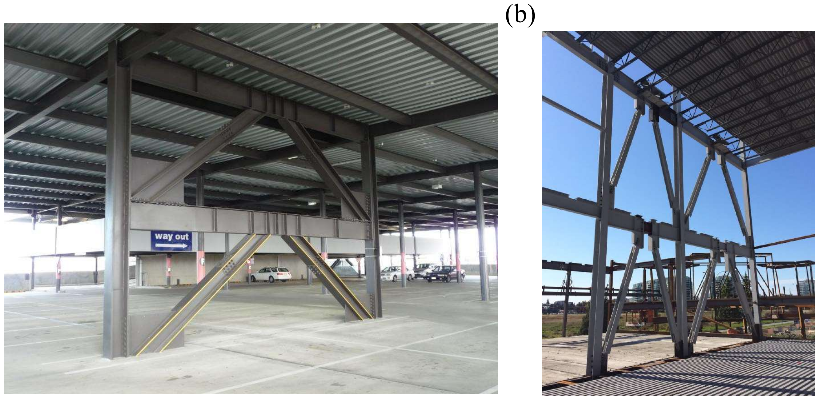

Figure 1. (a) Two-tiered eccentrically braced frame (MCEER report) [5]*; b) Two-tiered eccentrically braced frame with replaceable links in Richmond B.C.* [6].

Seismic design requirements in the Canadian design standard (CSA S16) [7] for MT-CBFs were updated based on the results obtained from the past studies. In the U.S., new seismic design provisions were introduced in the 2016 AISC Seismic Provisions [8] to address the concerns raised regarding the stability of the MT-BF columns under seismic loading. The seismic design requirements in Canada and the U.S. require the MT-CBFs be designed for in-plane and out-of-plane flexural bending moment demands combined with the axial compression force induced by the inelastic brace response plus gravity loads. The in-plane bending moment is induced in the columns as a result of sequential yielding of bracing members along the height of the frame and is computed using the difference between storey shear resistances of adjacent bracing panels. This shear force is obtained when the compression brace force in one of the tiers reaches its probable compressive resistance C_u and the probable tensile resistance T_u is achieved in the tension brace of the same tier while in the adjacent tier the compression brace develops its probable post-buckling resistance C'_u and the tension brace reaches its probable tensile resistance T_u. Various parameters contribute to the out-of-plane bending moment including brace out-of-plane buckling, plastic hinge forming in the brace gusset plate and column initial out-of-straightness. Although there exist differences in design requirements, both S16 and AISC Seismic Provisions acknowledge the fact that the braced frame columns in MT-CBFs should be designed for the combined effects of the axial force and biaxial bending moments. CSA S16 requires the force demands be computed at the design storey drift. AISC Seismic Provisions however enforces a stiffness requirement to limit the deformation demands induced in the individual braced panels for special steel braced frame systems. Although special seismic design provisions are prescribed by the current CSA S16 and AISC Seismic Provisions for multi-tiered CBFs and multi-tiered BRBFs, no design guidelines exist for steel multi-tiered EBFs and this framing configuration is still not allowed in the current North American standards.

Eccentrically braced frames aim to integrate the advantages of both moment resisting frame (MRF) and concentrically braced frame seismic force resisting systems where a seismic fuse with rigid ends are combined with laterally stiff vertical truss. EBFs provide larger openings compared to CBFs, allowing a better architectural versatility. Several configurations exist for an EBF with different yielding mechanisms such as V-type EBF, Chevron-type EBF and diagonal EBF. The segment between the end of the braces or the end of the brace and beam-to-column connection is expected to act as a seismic fuse during severe earthquakes and therefore is designed to dissipate seismic input energy through stable yielding in shear, flexure or a combination of shear and flexure while maintaining link deformations below the acceptable limits. As prescribed by the capacity design principles, all other members of the frame are proportioned to remain essentially elastic. Due to excessive inelastic rotation capacity and stable inelastic response exhibited by shear yielding, short links are preferred in design. Although, past studies confirmed that the elastic response of the frame elements other than links is not always achieved, the occurrence of limited yielding of outer beam segments is expected [9,10]. Such inelastic deformations are acceptable provided that the lateral stability of the member is not compromised.

Under lateral seismic loads, it is expected that MT-EBF columns experience in-plane bending moments due to frame non-uniform deformations caused by sequential yielding of links. Additionally, the stability of links may be compromised as no lateral braces are provided by the perpendicular framing system along the height of the frame. Since columns in MT-EBFs possess long unbraced lengths in the direction perpendicular to the braced frame, such instability may also affect the lateral stability of the columns subjected to the combined axial compression force due to gravity loads and inelastic link response, and in-plane bending moment demands due to non-uniform yielding of links. Although, special design requirements have been prescribed by North American steel design standards [7,8] for MT-CBFs and MT-BRBFs, MT-EBFs are still not allowed in the current standards, primarily because of the lack of supporting research data. In view of the extensive use of MT-BFs in North America and the advantages of using eccentrically braced frames in particular in high seismic zones, there is a need to assess the seismic behaviour of steel MT-EBFs.

This paper aims to investigate the seismic behaviour of multi-tiered eccentrically braced frames with a focus on the link and column response. A two-tiered steel eccentrically braced frame part of an industrial building in Vancouver B.C. was selected. The frame was designed in accordance with the current CSA S16 as a conventional Ductile EBF system. The numerical model of the frame was then developed in the *OpenSees* program [11]. The cyclic and kinematic hardening parameters of the steel material used for braces and columns were validated using cyclic coupon test data. Additionally, the shear force-shear deformation response of the link beam was validated against available test results. The static nonlinear (pushover) analysis method was finally used to investigate the seismic response of the frame and to evaluate the response of the links and columns. Two scenarios were considered to perform the numerical analysis: 1) lateral braces are provided at both ends of the link beams at strut and roof level; 2) lateral braces are only provided at both ends of the link beam at the roof level. Lateral force-storey drift, tier drifts, force and deformation response of links, in-plane and out-of-plane bending moment demands of the columns obtained from the analyses were used to evaluate the frame local and global responses. Finally, the applicability and adequacy of the current CSA S16 requirements for eccentrically braced frames were discussed.

FRAME SEISMIC DESIGN

Building geometry

A two-storey industrial building with 119 m x 42 m plan dimensions was selected for this study. The building height is equal to 9 m. The building is located in Vancouver, B.C. The columns support 119 m long roof trusses that span over the full width of the building. The spacing of the exterior columns is 7 m. Four steel Type D (ductile) eccentrically braced frames are placed in each direction (two braced frames per exterior wall). One of the braced frames located on the longitudinal wall was studied here (Fig. 2a). The height of the storey is equally divided between two Chevron type EBF tiers as shown in Fig. 2.

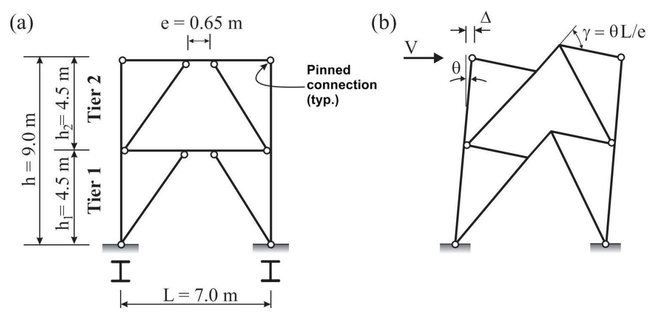

Figure 2. (a) Geometry of the selected two-tiered EBF; (b) Expected plastic mechanism of two-tiered EBF with shear links under seismic loads, θ = tier plastic rotation Δ = tier lateral deflection (deformations and rotations shown for Tier 2 only).

The building studied is located on a site class C (360 m/s ≤ Vs ≤ 760 m/s, where *Vs* is the average shear wave velocity). Seismic loading was performed in accordance with the National Building Code of Canada (13). The design roof dead load (D) and snow load (S) are equal to 1.0 kPa and 1.64 kPa, respectively. The unit weight of the exterior walls is assumed to be 0.5 kPa. Thus, gravity loads induced in each of the braced frame column under the load combination D + E + 0.5L + 0.25S is P_g = 517 kN. The importance factor I_E is equal to 1.0. The factor accounting for the increase in base shear because of higher mode effects M_v = 1.0. The ductility-related and over-strength related modification factors R_d and R_o are equal to 4.0 and 1.5, respectively. The empirical structural period T_a ($T_a = 0.025h_n$ where h_n is the total height of the structure above the ground) was obtained equal to 0.225 s. This resulted in a design spectral acceleration of 0.767g based on the probability of exceedance of 2% in 50 years. The total seismic weight of the building is W = 7758 kN, which was distributed between four EBFs in the longitudinal direction of the building. As per NBCC, the equivalent static force procedure can be used for this building to obtain the seismic load, resulting in a total seismic base shear per braced frame equal to V = 306 kN, including the effects of accidental torsion and notional load. The design base shear was determined using the increased period equal to $2.0T_a$, as permitted by NBCC [12]. This assumption was justified using a modal analysis performed by the *OpenSees* model. Since the fundamental period of the structure does not exceed 0.7 s, no concentrated load is considered at the roof level of the building.

Member design

The design of the frame was performed in accordance with CSA S16. The links as the deformation-controlled members of the selected EBF were the first components to be designed. The shear yielding mechanism was chosen as shown in Fig. 2b. A parametric study was then performed to define the optimal link length. Based on this study, a link length of 650 mm was selected. All the connections were considered as pinned as shown in Fig. 2a. For this frame, the resulting link shear force $V_f = Vh/L$ is equal to 197 kN in Tiers 1 and 2. The continuous wide-flange cross-section was chosen for the links at both roof and strut levels. The links conform to ASTM A992 Grade 50 steel with F_y = 345 MPa. The link was sized to possess the required factor shear resistance with zero axial load $V_r = \varphi V_p = 0.55\varphi wdF_y$, where $\varphi = 0.9$, w and d are the thickness of the web and depth of the cross-section, respectively. V_p is the nominal plastic shear resistance of the link. The length of the link e was selected such that shear yielding takes place prior to flexural yielding, $V_p < 2M_p/e$ where M_p is the plastic moment of the link. Additionally, the length of the link was set $e \leq 1.6M_p/V_p$ to achieve a shear link as per CSA S16. The selected link length e was equal to 1.18 M_p/V_p to promote shear yielding in both tiers. The width-to-thickness of the section must comply with the Class 2 flanges and Class 1 webs, when the expected plastic mechanism of the link is shear yielding. An identical wide-flange W200x35.9 was selected for both braced tiers. The details of the design calculations for the links are given in Table 1.

Table 1. Design calculations for the EBF link.

Tier	V_f kN	$A_{w,req}$ mm^2	Section	A_w mm^2	V_p kN	Z_x mm^3	$3.6Z_x/A_w$ mm	$1.6M_p/V_p$ mm
1 & 2	197	1151	W200x35.9	1250	237	379000	1091	720

Beams outside the link and columns were selected from wide-flange sections conforming ASTM A992 Grade 50 steel. Braces were chosen from hollow structural sections (HSSs) with ASTM A500, grade C steel. As shown in Fig. 3a, the braces and the beams outside the link were designed to resist the forces induced by the probable link resistance, which is equal to $1.3R_y/\varphi$ times the factored shear resistance of the link V_r, where R_y is the ratio between the expected R_yF_y = 385 MPa and nominal yield strength. CSA S16 permits multiplying the factored resistance of the beam outside the link by φR_y when verifying the resistance of the beam under capacity-induced forces. The moment induced at the end of the link $0.65R_yV_re/\varphi$ imposed at the beam outside the link. The axial force in the beam outside the link was equal to the horizontal projection of the brace axial force (Fig. 3b). Thus, the combination of the axial compression force $C_{f\text{-}b2}$ = 267 kN and strong axis bending moment demand $M_{f\text{-}b2}$ = 110 kN-m under gravity and seismic load effects was used to check the adequacy of the beam outside the link in the roof level. Note that the gravity-induced bending moment demand was equal to M_{fg2} =2.0 kN-m. For the design of the beam outside the link at the intermediate strut level, the combination of the axial compression force $C_{f\text{-}b1}$ = 269 kN and strong axis bending moment $M_{f\text{-}b1}$ = 112 kN-m induced by the seismic load only was considered. The selected W200x35.9 section was sufficient for both links in Tiers 1 and 2. This section has a factored compressive resistance $C_{r\text{-}b}$ = 1418 kN and factored flexural bending moment resistance equal to $M_{r\text{-}b}$ = 118 kN-m. This section is classified as a Class 2 section as per CSA S16 provisions.

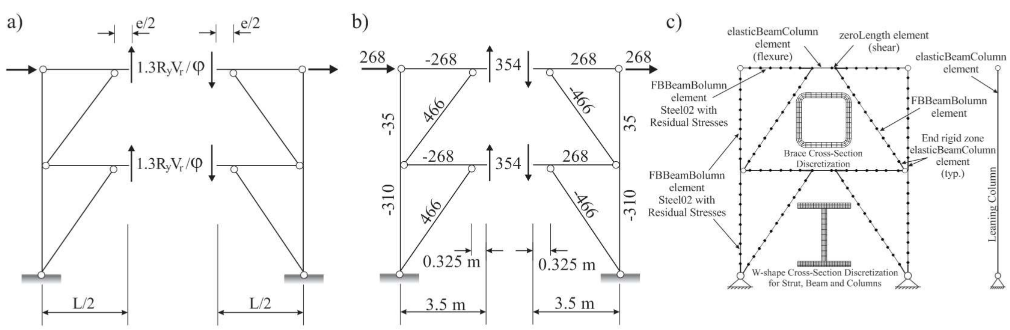

Figure 3. (a) Probable link resistances; (b) Capacity-induced seismic forces under the probable link resistance (forces in kN); and (c) Numerical model of the two-tiered eccentrically braced frame.

The design axial force of the braces was obtained from the vertical force equilibrium at the brace-to-beam connection when the probable link resistances are applied as shown in Fig. 3a. The braces were sized to resist an axial compression force C_f = 466 kN (Fig. 3b). A square HSS152.4x152.4x4.8 section that complies with CSA S16 width-to-thickness limit for Class 2 sections was chosen for both tiers. The factored axial compressive resistance of this member is $C_{r\text{-}br}$ = 465 kN. It should be noted that had a fixed end brace assumed in design, the moment induced at the brace-to-beam connection could be distributed between the beam outside the link and the brace based on their relative flexural stiffness as proposed by Engelhardt and Popov (9).

The columns were selected from available W-shape sections and considered continuous over the full height of the frame. W-shapes were oriented such that their web is perpendicular to the plane of the braced frame as shown in Fig. 2a. Axial forces induced by the gravity loads were combined with the forces developed by the probable link resistance shown in Fig. 3a. CSA S16 requires an additional bending moment equal to $0.2M_p$ (M_p is the plastic moment resistance of the column) be considered in the plane of the frame when designing a braced frame column to provide the column with adequate strength and stiffness, reducing non-uniform lateral deflections between adjacent stories. This requirement was neglected in the design of the two-tiered EBF example here to better evaluate the influence of the differential lateral displacement of adjacent tiers on the column in-plane moment demands. Thus, the columns were designed to resist in compression the seismic load induced by inelastic force of the links plus gravity load effects, resulting in a design axial compression force $C_{f\text{-}c}$ = 517 kN in the first tier. An effective length 0.87h, 0.81h_1 and h_1 were used in the design of columns for in-plane, out-of-plane and torsional buckling, respectively. The in-plane and out-of-plane effective length factors were obtained taking into account the in-plane bracing provided by an intermediate strut as well as the effect of distributed axial loads along the height of the frame [13]. An elastic Eigen buckling analysis was performed using SAP 2000 [14] on an individual column under the respective axial loads applied at the column top and at the strut level. A W250x38.5 conforming ASTM A992 steel was selected with a factored axial compressive resistance $C_{r\text{-}c}$ = 545 kN. The selected column satisfies the CSA S16 width-to-thickness ratio limits for Class 2 sections, which also meets the seismic design provisions specified for EBF columns.

NUMERICAL MODEL

The lateral response of the two-tiered EBF of Fig. 2a was evaluated using the nonlinear static (pushover) analysis method. The numerical model of the frame was developed in *OpenSees* 2.5.0 as shown in Fig. 3c. In the model, columns and braces are modeled with fiber discretization of the cross-section using the force-based beam-column element. Past studies have shown that this model can appropriately predict inelastic flexural buckling response of W-shape columns [15] and HSS bracing members [16,17] under cyclic loading. The capability of the model to predict flexural buckling of W-shape MT-CBF columns was also verified against a three-dimensional finite element model [2]. The Giuffre-Menegotto-Pinto material model from the *OpenSees* material library (Steel02 Material) was chosen to account for the Baushinger effect as well as isotropic and kinematic strain hardening behaviour of steel. The input parameters of the material were first calibrated against the cyclic coupon test data performed on CSA G40.21-350WT steel coupons by Dehghani et al. [18] using a 2-dimensional truss element to reproduce a steel coupon loaded under an incrementally increasing strain demand in room temperature. A MATLAB subroutine [19] was developed to minimize the error between the simulated response by the *OpenSees* model and the test data. Fig. 4a compares the stress-strain response obtained from the calibration performed in *OpenSees* with the cyclic coupon test data. As shown, the *OpenSess* Steel02 material model was able to adequately reproduce the cyclic response of the steel material. The calibrated material parameters were used to define the Steel02 material in the numerical model (Fig. 3c) along with modulus of elasticity E = 200 GPa, and yield strength F_y = 345 MPa for the braces, columns and beams outside the link. Residual stresses based on the pattern proposed by Galambos and Ketter [20] were included in the model for the W-shape beam, strut and columns. To trigger buckling of the beam, strut, braces and columns, each member was divided into ten elements [16]. A corotational formulation that accounts for P-Δ effects and large deformations was used to simulate geometric nonlinearities.

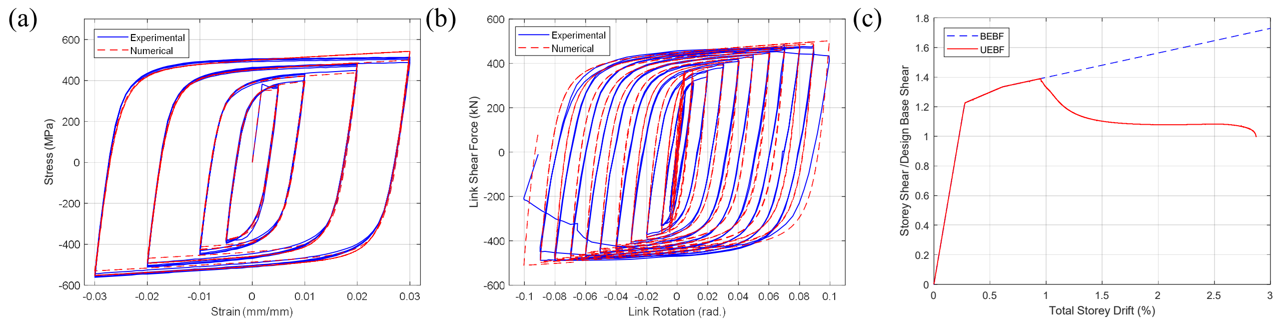

Figure 4. (a) Calibration of Giuffré-Menegotto-Pinto material model (data by Dehghani et al. 2016 [18]); (b) Shear force-shear deformation beahaviour of the W250x28 link (data by Okazaki et al. [21]*); (c) Lateral force-lateral deformation response.*

The beam and strut were assigned initial sinusoidal imperfections in the in-plane and out-of-plane directions. For in-plane buckling, a half sinewave was assigned to the length between the beam-to-column connection and link-to-brace connection with a defect amplitude equal to 0.001 times the respective length; however, a half sinewave with a maximum amplitude of 0.001 times the beam length between the lateral out-of-plane supports was specified for out-of-plane buckling. The braces were assigned an initial sinusoidal imperfection corresponding to their first out-of-plane buckling mode shape with a maximum amplitude of 0.001 times the length between the brace ends excluding the connection lengths. For the columns, bi-directional initial sinusoidal out-of-straightness corresponding to the first in-plane and out-of-plane buckling modes was considered. Defect amplitudes were set equal to 0.001 times the full frame height and 0.001 times the tier height for out-of-plane and in-plane buckling, respectively. The columns were pinned at their bases and roof level in the plane and out of the plane of the frame. Lateral braces were assigned to the columns at their top ends as well as the ends of Tier 2 link. Two lateral bracing scenarios were considered for the strut in this study. In the Unbraced EBF (UEBF) model, no lateral brace was considered for the strut; however, the strut was laterally braced at the end of its link in the Braced EBF (BEBF).

The link shear and flexural behaviour was simulated using an elasticBeamColumn element and an inelastic zeroLength spring element. The elastic element was assigned a modulus of elasticity of 200 GPa, cross-sectional area and moment of inertia of the selected section to represent the elastic flexural stiffness of the link. The single spring approach with nonlinear properties was used to simulate the shear force-shear deformation response of the link. The inelastic behaviour of the link was described using the Giuffré-Menegotto-Pinto material model calibrated against data from cyclic experimental tests performed on short links by Okazaki et al. [21]. A separate numerical model including an elasticBeamColumn element and an inelastic zeroLength spring element with Steel02 material was created to calibrate the steel material model in the *OpenSees* program. A 0.635 m W250x28 link specimen with relatively close geometry to the link selected in this study was selected to perform this calibration. The measured yield stress reported by Okazaki et al. [21] was used in the link model. The comparison between the experimental data for W250x28 and the corresponding numerical model under cyclic displacement history is shown in Fig. 4b. The parameters assigned to the Steel02 material obtained from the calibration are as follows: b = 0.0035, R0 = 20, CR1 = 0.925, CR2 = 0.15, a1 = a3 = 0.11; and a2 = a4 = 10. As shown in Fig. 4b, the parameters used to define the Giuffré-Menegotto-Pinto material in the *OpenSees* link model can well predict the shear response of the link and small variations observed can be attributed to the fact that a bilinear model was used to reproduce the link bahaviour. A better match can be obtained using a multilinear model [11,22]. Expected yield strengths equal to $0.9R_yF_y$ = 347 MPa and R_yF_y = 385 MPa were assigned to the links in Tiers 1 and 2, respectively. The 10% reduction in the yield strength of Tier 1 was considered to account for plausible material variations between the links. Thus, it is expected that shear yielding initiates first in the Tier 1 link.

The braces, beam and strut outside the link were modeled using forceBeamColumn elements with an inelastic material response. Elastic beam-column elements with stiffness representing the properties of the connection and adjacent members were used to reproduce connection sizes at the end of braces, column segments where they meet braces, beam and strut segments outside the link where they meet the braces. The length of rigid links was set to match the connection geometry obtained from the connection design. A leaning column represented by an elasticBeamColumn element with relatively large axial and flexural stiffness was included in the model to reproduce the P-Δ effects. The horizontal translation of the top end of the leaning column was constrained to the braced frame roof. Gravity loads tributary of the braced frame were applied to the top end of columns of the braced frame and the remaining frame tributary gravity loads were applied to the leaning column. Once the gravity load is applied in the model, a pushover analysis was performed by gradually increasing the frame roof displacement to 3% of the frame height using the static analysis approach in the *OpenSees* program.

ANALYSIS RESULTS AND DISCUSSION

Nonlinear static analyses were performed on two two-tiered EBFs: Unbraced EBF (UEBF) and Braced EBF (BEBF) as described earlier. This section presents the analysis results and the response observed in the links and columns. The storey shear-storey drift responses are illustrated in Fig. 4c. Braced EBF showed stable inelastic behaviour without significant loss of lateral strength over the total drift range, however, the lateral resistance of Unbraced EBF reduced significantly because of link instability. Both frames exhibited significant lateral overstrength, mainly due to the strain hardening of the link, probable yield strength and the reserved capacity available in the link as a result of the applied resistance factor. Fig. 5a shows tier drifts plotted against the storey drift. Tier drift was calculated by dividing each tier relative lateral displacement by respective tier heights. As shown, the response of both frames is relatively identical at the beginning of the analysis. The lateral deformations of both tiers increase linearly until 0.3% storey drift. Shear yielding occurred in the Tier 1 link at 0.3% storey drift as expected. Additional lateral frame deformations were then concentrated in Tier 1 while no further lateral deformation was observed in Tier 2. However, the Tier 2 link yielded at approximately 0.65% storey drift, which resulted in sharing the lateral displacement between tiers until the end of the analysis. As opposed to the Braced EBF, buckling of the Tier 1 link of Unbraced EBF at 0.95% storey drift changed the rate of deformations in tiers of this frame. The results of the out-of-plane displacement of the Tier 1 link for the Unbraced EBF shown in Fig. 6a confirms the out-of-plane buckling of the link at 0.95% storey drift. Link instability initiated first at its left end and propagated to the right end of the link. The buckling of the Tier 1 link in Unbraced EBF imposed large in-plane and out-of-plane bending moments on the columns. Figs. 5b and 5c show the in-plane and out-of-plane moments of the columns, respectively, normalized to the respective plastic moments. No significant in-plane moment was observed in the columns of both Unbraced EBF and Braced EBF before the initiation of shear yielding in the Tier 1 link. Once the first link yields, the column in-plane moment increases due to the differential lateral displacement between tiers. Nonetheless, this rate of increase diminishes as soon as the Tier 2 link yield in both frames at approximately 0.65% storey drift. The in-plane moment amplitude remained the same until the end of the analysis for the Braced EBF. While, the out-of-plane buckling of the Tier 1 link imposed a significant in-plane bending moment on the columns of Unbraced EBF, in particular, on the left-hand-side column because of the larger deformations developed at the left end of the Tier 1 link (Fig. 6a). The in-plane moment of the left column of Unbraced EBF dramatically increased by increasing the storey drift, which combined with the out-of-plane bending moment and axial compression force demands led to column instability at approximately 2.8% storey drift. Column out-of-plane bending moments were shown in Fig. 5c for both frames. For Braced EBF, limited out-of-plane moments were observed in both columns mainly due to the effects of the initial out-of-straightness. However, Unbraced EBF experienced significant bending moment demands as a result of the out-of-plane buckling of the Tier 1 link. The out-of-plane bending moment was more pronounced in the right-hand-side column because this column experience compression force under the applied lateral displacement. Column in-plane and out-of-plane moment demands at buckling reached 0.75M_{py} and 0.10M_{px}, respectively, where M_{py} and M_{px} are the plastic moment of the column about its minor and major axes respectively.

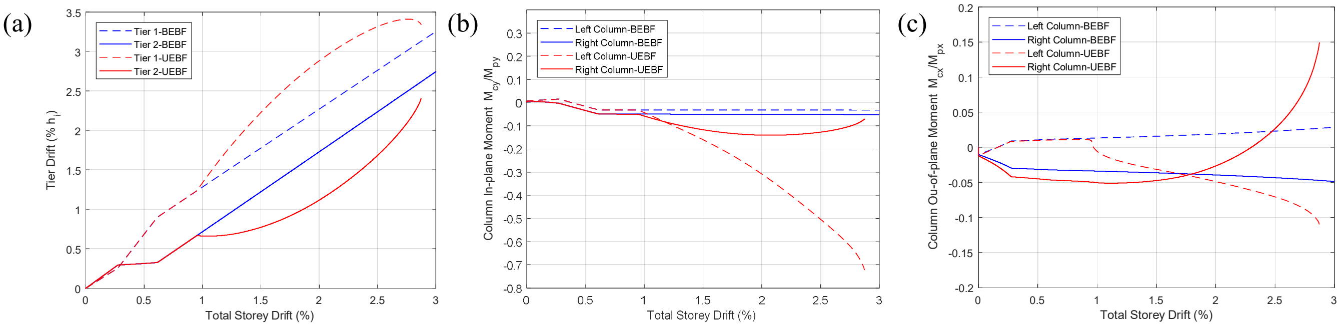

Figure 5. (a) Tier drifts; (b) Column in-plane bending moments; and (c) Column out-of-plane bending moments.

The shear force developed in the links were plotted against the link rotation in Fig. 6b. In this figure, the vertical axis is normalized by the link probable shear resistance V_{pr}. As shown, the link strength in both tiers of Braced EBF increased by increasing the lateral displacement, whereas, very limited inelastic deformation was observed in the Tier 2 link of Unraced EBF because of the concentration of inelastic deformations in the Tier 1 link followed by the bucking of this link. For Unraced EBF, the shear rotation observed in Tier 1 and Tier 2 links were approximately 0.12 rad. and 0.05 rad., respectively, when the Tier 1 link buckled. Fig. 6 (c) shows the normalized link shear force against the storey drift. The shear force measured in Unbraced EBF links decreased at the storey drift of 0.95% as a result of instability in the Tier 1 link.

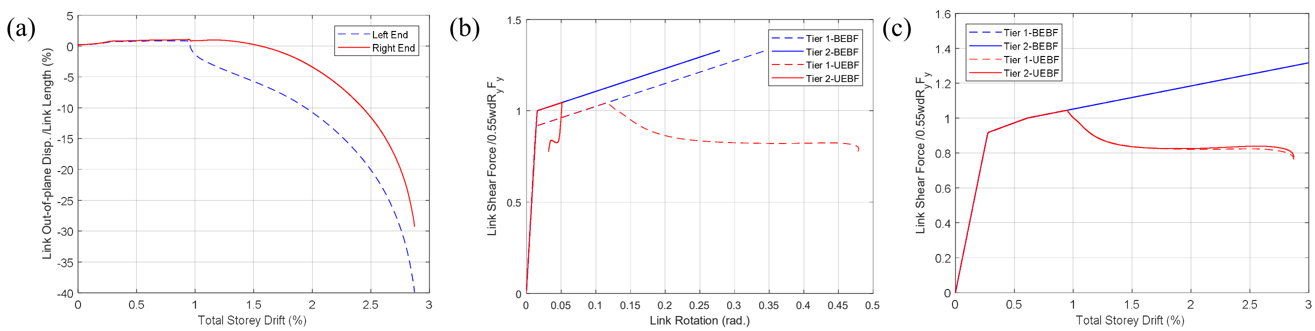

Figure 6. (a) Out-of-plane displacement of the link ends in Unbraced EBF; (b) Link shear force-rotation response; and (c) Link shear force versus storey drift.

CONCLUSIONS

This paper investigates the seismic behaviour of multi-tiered eccentrically braced frames. A two-tiered EBF part of an industrial building was designed in accordance with the current Canadian steel design standard as a conventional EBF. A numerical model of the frame was developed using the *OpenSees* program. The inelastic hysteretic response of the link was reproduced using the available test data. Two link lateral support scenarios were examined for the link in the intermediate strut level: 1) the link is laterally braced at its both ends; and 2) no lateral support is considered. Nonlinear static analysis was performed to study the lateral response of the frame and evaluate the response of links and columns. The main findings of this study are summarized as follows:

- The spring-beam model developed in the *OpenSees* environment can appropriately reproduce the inelastic cyclic response of short links. It was found that the parameters assigned to the Giuffré-Menegotto-Pinto material model in the *OpenSees* program significantly affect the cyclic behaviour of the link and in turn the seismic performance of the MT-EBF.
- The numerical model of the multi-tiered eccentrically braced frame developed in the *OpenSees* environment can properly predict the expected plastic mechanism in steel EBFs.
- In Braced EBF where lateral out-of-plane supports are specified at the ends of the Tier 1 link, although shear yielding first initiated in the Tier 1 link, the frame lateral deformations were relatively uniform as the Tier 2 link yields. Neither link nor column instability was observed for this EBF model.
- In Unbraced EBF with no lateral support assigned to the Tier 1 link, nonuniform distribution of lateral inelastic deformation was observed, which led the out-of-plane buckling of the Tier 2 link at approximately 0.95% storey drift due to the lack of out-of-plane lateral support and large inelastic shear deformations developed in the link.

- In Unbraced EBF, link instability at the strut level eventually led to column out-of-plane buckling at approximately 2.8% storey drift in the presence of axial compression and bi-axial bending moment demands.
- Column in-plane and out-of-plane bending moments were pronounced in the Unbraced frame due to the buckling of the Tier 1 link. Such moments should be accounted for in design for multi-tiered eccentrically braced frames.
- The results of the numerical analysis of Unbraced EBF suggest that the seismic design provisions implicit in CSA S16 for the conventional EBFs should be revisited to address the potential instability in EBFs with a multi-tiered configuration.

In this study, the braces were simulated using a pinned end condition neglecting the plastic response of the brace gusset plate. Future studies should consider a more realistic brace end conditions and allow for inelasticity in members other than the links. Additionally, nonlinear dynamic analysis should be performed to obtain a realistic estimation of the seismic-induced demands in MT-EBF members.

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