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**Seismic Response and Design of Steel Multi-Tiered Concentrically Braced
Frames of the Conventional Construction Category in Moderate Seismic
Regions of Eastern Canada**

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Abstract: This paper examines the seismic behaviour of steel Multi-Tiered Concentrically Braced Frames (MT-CBFs) of the Conventional Construction category (Type CC) in moderate seismic regions of eastern Canada and proposes a new seismic design method for such structures. A total of 32 prototype MT-CBFs part of a single-storey building are selected and designed in accordance with the 2019 Canadian steel design standard (CSA S16-19) provisions for Type CC structures, excluding the design requirements prescribed for frames taller than 15m, namely the amplification of seismic forces varying by the frame height and the amplification of the column factored seismic force. Nonlinear response history analyses are then performed to examine their seismic response. The results confirmed that tiers experience limited lateral deformation, intermediate horizontal struts are required between tiers, and columns are subjected to combined axial force and bi-axial bending, which in some cases led to column buckling. The results are used to propose a new seismic design method in the framework of CSA S16.

Keywords: steel multi-tiered concentrically braced frames, conventional construction, numerical simulation, seismic design.

1 INTRODUCTION

Steel Concentrically Braced Frames (CBFs) are widely used as a Seismic Force-Resisting System (SFRS) in the construction of steel buildings around the world. CBFs comprise columns, beams, and braces, creating a vertical truss to transfer horizontal loads, such as wind or earthquakes. Multi-Tiered Concentrically Braced Frames (MT-CBFs) are a special CBF configuration that consists of multiple concentric bracing panels that are stacked between two adjacent floor levels (Fig. 1a) when it is impractical or uneconomical to employ only a single braced panel along the frame height (Fig. 1b). The multi-tiered configuration favours the use of short braces, smaller brace sizes, or smaller connections while efficiently reducing the column in-plane unbraced length, provided that horizontal intermediate struts are used between tiers. Two examples of MT-CBFs are shown in Figs. 1c and 1d. MT-CBFs are commonly used in Canada in tall single-storey buildings, such as industrial buildings, airplane hangars, or sports facilities. A large proportion of such buildings is located in regions of low-to-moderate seismicity in eastern Canada, where design engineers often prefer to use Conventional Construction (Type CC) structures for seismic applications to avoid intricate structural detailing and prohibitively expensive strengthening requirements. Such requirements were primarily developed for high-ductility systems, which are often used in high seismic regions.

Various combinations of structural shapes are used in the construction of Type CC MT-CBFs. The choice of sections depends on the building type, seismicity level, availability of the sections in the market, and connection details. Braces are typically made of Hollow Structural Sections (HSSs), single- or double-angles. Wide-flange shapes or HSSs are often used for columns. The struts are typically made of HSSs, and wide-flange shapes are used for the roof beam.

Under seismic loads, Type CC structures are expected to exhibit some capacity to dissipate seismic energy through localized yielding and friction that inherently exists in the structure. In the 2019 Canadian steel design standard, CSA S16-19 (CSA 2019), capacity design is not required for Type CC structures, which simplifies their seismic design. Although S16 specifies special design requirements for moderately-ductile and limited-ductility MT-CBFs, no special seismic design provisions have yet been introduced for Type CC MT-CBFs.

Extensive numerical simulations were performed on MT-CBFs in the last decade with an emphasis on areas with high seismicity. These studies primarily served to better understand the seismic response of such frames and develop enhanced seismic design provisions in Canada and the United States (Imanpour and Tremblay 2016; Imanpour et al. 2016a, 2016b, 2017; Cano and Imanpour 2020). It was confirmed that the inelastic lateral deformations of the frame are non-uniformly distributed over the frame height because the tension-acting braces yield in only one of the braced tiers rather than all of the tiers. This response imposes significant bending on the columns in the plane of the frame. The column moment demands, if not considered in the design, may result in plastic hinge forming in the column, which in turn compromises the stability of the column and, consequently, the integrity of the frame.

To the authors' knowledge, no research to date has been conducted to evaluate the seismic performance of Type CC MT-CBFs designed as per Canadian design standards. Additionally, Type CC frames are not detailed to achieve a ductile inelastic response; thus, their governing limit states are mostly unknown (Tremblay et al. 2009; Glinas et al. 2012; Decaen 2015). Finally, S16 is silent about Type CC structures designed using the multi-tiered configuration, with the exception of a large out-of-plane moment demand for the columns of such frames, which lacks sufficient background research. In light of the extensive use of such structures in regions of low and

moderate seismicity in eastern Canada, there is an urgent need to assess their seismic performance, evaluate the need for special seismic design requirements, and develop a new seismic design method.

This paper investigates the seismic response of Type CC MT-CBFs part of a single-storey building and proposes new seismic design requirements for such frames. A single-storey building with multi-tiered CBFs located in a moderate seismic region of eastern Canada is first selected. A series of CBFs consisting of two- to six-tiers is then designed per CSA S16-19, excluding the special seismic design provisions prescribed for Type CC structures taller than 15m to evaluate the need for such requirements in MT-CBFs. The numerical model of the braced frame is developed, and nonlinear dynamic analyses are performed to examine the seismic response of the frames and estimate seismic-induced demands in their members, particularly columns, and assess the necessity of the current S16 requirements for Type CC frames. The results of the analyses are used to propose a new seismic design method. Finally, the seismic response of a set of eleven prototype frames that contain eight new designs are numerically evaluated to validate the proposed design method.

2 PROTOTYPE BUILDING AND LOADING

2.1 Building Selected

The building selected in this study is a single-storey industrial building as shown in Fig. 2a with plan dimensions of 42 m $\times$ 252 m, located in Montreal, Quebec, which is a moderate seismic hazard region (Seismic Category 3 or SC3) in eastern Canada. The longer dimension of the building is chosen such that the factored seismic base shear always exceeds that produced by the wind load. In this direction, the building is divided into two parts that are connected by an expansion joint to accommodate the temperature gradient in the roof. The exterior wall columns

were spaced 6.0 m apart to support 42.0 m-long roof trusses that span the full width of the building. The building SFRS comprises eight steel X-bracing MT-CBFs in the longer direction (four frames per exterior wall), and eight steel X-bracing MT-CBFs in the shorter direction, all placed on the perimeter. The study focused on one of the braced frames located along the longer side of the building.

2.2 Parameters Considered

In this study, 32 Type CC MT-CBFs are defined by varying four parameters: total height of the frame, h , and number of braced tiers, n : $h = 10\text{m}$ ($n = 2$ and 3), $h = 16\text{m}$ ($n = 2, 3$, and 4), and $h = 24\text{m}$ ($n = 3, 4$, and 6); first-tier to second-tier height ratio $h_1/h_2 = 1.0$ and 1.75 ; column and brace sections: Tension-Compression (T/C) square HSS braces with wide-flange (W) columns, and Tension-Only (T/O) single-angle (L) braces with square HSS columns. The roof beam and struts of all frames are made of wide-flange sections and square HSSs, respectively. The selected parameters are intended to reflect the current design and construction practice in eastern Canada. Table 1 lists the selected parameters. The geometry of a generic six-tiered CBF is shown in Fig. 1d.

2.3 Gravity, Seismic, and Wind Loading

Loads were calculated as per the 2015 National Building Code of Canada, NBCC (NRC 2015). The roof and exterior walls' dead loads (D) are equal to 1.0 kPa and 0.5 kPa, respectively. The roof dead load includes self-weight and superimposed dead load. The roof live load (L) is equal to 1.0 kPa. The roof snow load (S) for the selected site was found to be 2.61 kPa. To evaluate the selected frames under seismic and gravity loads, the NBCC seismic load combination, $1.0D + 1.0E + 0.5L + 0.25S$, was considered. The factored gravity load (i.e., $1.0D + 0.5L + 0.25S$) imposed on each column located along the longer direction of the building was computed as $P_G = 208\text{ kN}$.

Equivalent Static Force Procedure (ESFP) was used to calculate the design seismic base shear. The selected building is of Normal importance with the seismic importance factor $I_E = 1.0$. The higher mode factor M_v is taken as 1.0. The ductility-related force modification factor is $R_d = 1.5$, and the overstrength-related force modification factor is $R_o = 1.3$. The building is located on Site Class C. The design period T_d of the building was taken as the minimum of the period obtained from a modal analysis and two times that calculated using the NBCC empirical equation for braced frames, $2 \times 0.025h$. For the frames studied here, the latter is always lower, which results in design periods T_d equal to 0.5s, 0.8s, and 1.2s for frames with $h = 10, 16$, and 24 m, respectively. The design response spectral acceleration $S(T)$ is equal to 0.31g, 0.21g, and 0.13g for frames with $h = 10, 16$ and 24 m, respectively. The total seismic weight W is equal to 18948, 19830, and 21006 kN for frames with $h = 10, 16$, and 24 m, respectively. The total seismic base shear per braced frame V including the effects of accidental torsion, P- Δ , and notional loads, is therefore equal to 464, 336, and 224 kN, respectively, for frames with $h = 10, 16$ and 24 m.

The specified wind pressure plus suction calculated as 0.65 kPa was used to obtain the lateral wind load (W). The governing load combination for wind loading was found to be $1.25D + 1.4W + 0.5S$. For all three building heights considered, the factored seismic base shear was greater than the factored wind base shear. Thus, the seismic load governs the design of the main structural members of the selected braced frames.

3 SEISMIC DESIGN OF BRACED FRAMES

3.1 CSA S16 Provisions for Type CC Braced Frames

Seismic force-resisting systems of the conventional construction (Type CC) category in moderate regions should be designed to carry the design seismic base shear if the total height of the frame is below 15 m. No local or global slenderness limits are required by the standard for

such frames. For Type CC structures taller than 15m, two amplification factors must be considered in design: 1) factored seismic forces should be amplified linearly by 2% per meter for a fraction of the frame height above 15 m, and 2) the factored seismic forces of the columns and connections should be amplified by a second amplification factor equal to 1.3. The members' design forces shall not exceed the forces corresponding to $R_d R_o = 1.3$. Beams, columns, and HSS braces shall comply with the section slenderness requirements for Class 2 sections. For angle braces, the width-to-thickness ratio is verified against $170/(F_y)^{0.5}$ only when the brace global slenderness ratio, $KL/r \leq 200$ where KL is the effective length of the member and r is the minimum radius of gyration of the cross-section. For columns that are intersected by bracing members at an unbraced location (e.g., MT-CBF columns), an additional out-of-plane transverse force equal to 10% of the axial load carried by the column at every unbraced brace-to-column intersection must be considered to account for out-of-plane moments due to brace out-of-plane buckling, P- δ effects, or plastic hinging of gusset plates, which often occur when braces experience large inelastic response (Cano and Imanpour 2020).

In this study, the section class limits specified in S16 for frames taller than 15m were also considered in the design of 10 m-tall frames to avoid associated local buckling limit states in 10 m-tall frame components. The seismic design requirements specified in S16 for Type CC structures exceeding 15 m in height and the additional out-of-plane transverse notional load specified for unbraced columns were not considered in design in order to examine the need for such requirements when the multi-tiered configuration is employed.

3.2 Design of Tension-Compression Braced Frames

Tension-Compression braces consist of square HSS fabricated from ASTM A500 Gr. C steel (ASTM 2015a) with the yield strength of $F_y = 345$ MPa. The braces were designed to resist the

design seismic load in compression with effective length factors equal to $K_x = K_y = 0.45$ for in-plane and out-of-plane buckling modes (Sabelli and Hohbach 1999; Tremblay et al. 2003). For braces with $KL/r \leq 200$, width-to-thickness ratio was verified against $525/(F_y)^{0.5}$. The selected sections are given in Table 1.

The columns and roof beams were selected from wide-flange sections conforming to ASTM A992 (ASTM 2015a) with $F_y = 345$ MPa to resist the combination of gravity and seismic loads (i.e., $1.0D + 0.25S + 1.0E$). Columns were continuous and oriented such that the out-of-plane wind load imposes strong-axis bending as typically done in practice, i.e., out-of-plane bending due to the wind load is resisted by strong-axis bending of W-shape sections located along the building exterior walls. To calculate the column resistance in the plane of the frame, the unsupported length was taken as equal to the tier height since the intermediate struts brace the column in the plane of the frame. In contrast, an unsupported length equal to the full-frame height was used to calculate the column out-of-plane buckling resistance because the column can buckle about its full-length out-of-plane. The columns' sections satisfy the width-to-thickness ratio limits for Class 2 sections. Once the columns' sections were determined, their capacity was verified under the gravity and wind load combination (i.e., $1.25D + 1.4W + 0.5S$), where gravity loads induce an axial compression force and the lateral wind load imposes strong-axis bending. Table 1 gives the selected sections for columns.

The roof beams were designed as beam-columns under an axial compression force induced by lateral seismic load and strong-axis bending resulting from the roof gravity load and the self-weight of the member. Struts were selected from square HSSs conforming to ASTM A500 Gr. C steel and designed as a compression member resisting a nodal bracing force required to brace the column against in-plane instability as specified in Clause 9.6.2 of S16. Beams and struts conform

to the width-to-thickness ratio limits prescribed for Class 2 sections. Table 1 shows the selected sections for roof beams and struts.

Once the members were selected, the lateral displacement of each frame under the seismic load, including inelastic effects, $R_d R_o \delta_e / I_E$, was verified against the limit prescribed by the NBCC, $2.5\%h$, where δ_e is the elastic roof displacement under the design seismic base shear. The NBCC limit of $h/500$ was then used to check the lateral displacement of the frames under the specified wind load. Frame drifts under the seismic and wind loads are given in Table 1.

3.3 Design of Tension-Only Braced Frames

Single-angle braces made of A36 steel (ASTM 2015b) with $F_y = 248$ MPa were designed as a tension-only member. The width-to-thickness ratio limit of $170/(F_y)^{0.5}$ was verified for the legs of angle braces when $KL/r \leq 200$. The sections selected for braces are given in Table 1.

Square HSSs conforming to ASTM A500 Gr. C were selected for columns. The columns were designed to resist the axial compression force due to gravity and seismic loads. The columns' strength and stability were verified using an effective length equal to the tier height for in-plane buckling and an effective length equal to the frame height for out-of-plane buckling. The selected sections meet the width-to-thickness ratio limit for Class 2 HSSs. The selected column sections were then verified under wind and gravity loads. Table 1 presents the final column sections.

Wide-flange sections were used for roof beams. The design procedure for roof beams was identical to that described in Section 3.2. HSS struts conforming to ASTM A500 Gr. C steel were selected and designed under the combined unbalanced axial compression force arising from the horizontal component of the tension-acting brace at every strut-to-column joint and the axial compression force required to brace the column in the plane of the frame as specified in Clause 9.6.2 of S16. Table 1 gives the selected sections for roof beams and struts.

4 NUMERICAL ANALYSIS

4.1 Numerical model development

A numerical model of the braced frame was developed in the *OpenSees* program (McKenna et al. 1997) to evaluate the seismic performance of Type CC MT-CBFs. The modelling techniques used in this study were successfully employed in the past to reproduce the inelastic cyclic response of HSS braces (Aguero et al. 2006; Uriz et al. 2008) and angle braces (Jiang 2013). Additionally, the program's capability to predict inelastic buckling of steel wide-flange columns was confirmed in past experimental and numerical studies (Lamarche and Tremblay 2011; Imanpour et al. 2016, 2017). The results of the validation studies performed to verify the cyclic inelastic and stability response of single- and double-angle braces, and HSS members can be found in Derakhshan Horeh and Imanpour (2020), and those carried out to verify the stability of wide-flange columns are available in Imanpour (2015) and Imanpour et al. (2017).

The numerical models for T/C and T/O frames are shown in Figs. 3a and 3b, respectively. Braces and columns were represented using nonlinear force-based elements with fibre discretization of the cross-section. Elastic elements with Young's modulus $E = 200$ GPa were used to model the roof beam and struts as they are expected to remain elastic. The Steel02 material model was used for braces and columns. The material was defined using $E = 200$ GPa, specified yield strength, $F_y = 345$ MPa assigned to columns, and expected yield strength, $R_y F_y = 460$ MPa and 385 MPa assigned to HSS and angle braces, respectively. It is expected that brace tensile yielding naturally occurs in the tier that has the least storey shear resistance (see Table 1), as determined by the horizontal components of brace probable tensile and compressive resistances (Imanpour et al. 2016a). The expected yield strength of the braces in that tier, however, was reduced by 10% to increase the storey shear difference between tiers and create more critical in-plane bending

demands on the columns. This reduction accounts for plausible variabilities in the material yield stress (Schmidt and Bartlett 2002), brace cross-sectional properties (Schmidt and Bartlett 2002), initial out-of-straightness, member length, increase in material strength due to strain rate effects (Lamarche and Tremblay 2011) or connection details (Davaran 2001; Davaran et al. 2014). For frames with identical tiers and similar storey shear resistance (see Table 1), the reduction was applied to the first tier, where the column carries the largest axial compression force, to create a more critical axial force and bending moment interaction on the column. Residual stresses reported by Adluri and Madugula (1996) were assigned to the angle cross-section. No residual stresses were considered for HSS sections because residual stresses in HSS braces have a negligible impact on their buckling load (Koval 2018). The residual stress pattern proposed by Galambos and Ketter (1959) was assigned to wide-flange cross-sections. To trigger global buckling, initial imperfections were introduced with a sinusoidal shape corresponding to the first in-plane and out-of-plane buckling mode shapes of columns and the first out-of-plane buckling mode shape of braces. The maximum initial out-of-straightness at the mid-length of the column unsupported length was set equal to 0.001 times the tier height and 0.001 times the full-frame height for in-plane and out-of-plane buckling modes, respectively. For braces, the maximum amplitude of the initial out-of-straightness is equal to 0.002 times the brace length between the corner and middle connections. A rotational spring with the Steel02 material was created at each end of the brace to reproduce the nonlinear response of the brace gusset plate connections when the brace buckles out-of-plane. In the T/C frame model, rotational springs were also provided at the ends of discontinuous brace segments where they meet the continuous brace to trigger brace buckling in compression within each half of the brace. The rotational spring was connected to the beam-to-column joint or the middle brace connection using a relatively rigid link simulated by an

elastic element with the axial and flexural stiffness equal to those of the gusset plate connection. The lengths of the rigid links were set to match the brace connection geometry.

The columns were pinned at their ends to represent the column base condition, which typically consists of four anchor rods in such light steel frames, and the connection between the roof beam and the column, which is typically a simple shear connection. The columns were laterally braced in the out-of-plane direction at their top end to represent the roof trusses framing into the braced frame and providing out-of-plane bracing (Fig. 3). The roof beam and strut were pin-connected to the columns. A pinned base leaning column was simulated to account for $P-\Delta$ effects. The building seismic weight tributary of the braced frame was assigned as lumped masses to the top end of the columns. Rayleigh damping, with a mass proportional damping corresponding to critical damping ratio of 0.02 in the structure first vibration mode, was specified in the model. It is worth noting that MT-CBFs have only one dominant dynamic degree-of-freedom, which corresponds to the roof lateral displacement. The gravity load is first applied using static analysis. The nonlinear response history analysis (NLRHA) was then performed using 30 ground motion records that are selected and scaled per the 2015 NBCC using the procedure described by Tremblay et al. (2015) to match, on average, the design response spectra of the selected site. The selected 30 records include two suites each representing a dominant magnitude-distance (M-R) scenario of earthquakes for the site. The first suite composed of 14 simulated ground motions from M6 events at distances ranging 10 – 30 km, and the second suite consists of 16 simulated ground motions from M7 earthquakes in the 15 – 70 km range. Response acceleration spectra of the scaled records and the average of each suite are plotted in Fig. 2b against the design spectrum. Additional information on the selection and scaling of the records can be found in Derakhshan Houreh and Imanpour (2020).

4.2 ANALYSIS RESULTS

The statistics of key seismic response parameters, including storey and tier drifts, brace forces, strut forces, column axial forces, column in-plane and out-of-plane bending moments, were calculated for each frame as recommended by the 2015 NBCC-Commentary J (NRC 2015). As per this recommendation, the maximum of means over the earthquake ensembles of the peak response parameter obtained under each ground motion record was computed for each response parameter and reported here. It should be noted that due to the large number of column buckling cases in some prototype frames (Table 2), which resulted in significantly large lateral displacements, the results of the analyses where such large displacements are observed were not considered in the statistics to allow for the comparison between the mean values for each response parameter.

4.2.1 Global response

Table 2 gives the peak frame drift response and the number of cases where column instability, which involves loss of the member axial load-carrying capacity as axial shortening increases, was observed. Column instability was detected in several cases in 16 m- and 24 m-tall T/C frames. Column instability involved out-of-plane buckling under large axial compression force and out-of-plane bending moments arising from brace out-of-plane buckling and $P-\delta$ effects. No column instability was observed in the 10 m-tall T/C frames, which is attributed to the lower out-of-plane moment induced in their columns. As shown in Table 2, no column buckling was reported for T/O braced frames.

In Table 2, δ_{St}/h represents the total storey drift, where δ_{St} is the peak lateral deflection measured at the roof level, and h is the storey height. As shown, significantly lower storey drifts compared to the NBCC limit were observed (0.29% – 0.44% for T/C frames and 0.34% – 0.37% for T/O frames). The frame storey drifts were compared to the respective design storey drift, including the

inelastic effects (i.e., $R_d R_o \delta_e / h$). Note that δ_e of T/O frames was calculated assuming that both braces of each tier contribute to the lateral stiffness of the frame. The results indicate a good correlation between the design storey drift of the frames with T/C braces and the analysis results. Slightly lower drifts observed in 16 m-tall frames with T/C braces were due to a large number of column buckling cases, which were excluded from the statistics. For T/O frames, however, the observed storey drift was higher than the respective design storey drift due to the loss of stiffness in compression-acting braces that experienced buckling, although limited as described later in the brace response.

The profiles of peak tier drifts are shown in Figs. 4a and 4b. The tier drift was calculated by dividing the difference between the lateral displacements measured at the top and bottom of each tier by the respective tier height. Tier drifts are limited to 0.59% for T/C frames and to 0.42% for T/O frames. On average, larger tier drifts were observed in T/C frames compared their T/O counterparts. For 10 m- and 16 m-tall frames, tier drifts are slightly larger in the second tier; however, tier drifts are almost uniformly distributed between the tiers in the frames with a height of 24 m. The observed tier drifts do not suggest possible low cycle fatigue fracture (Tremblay et al. 2003; Tremblay 2008; Fell et al. 2009; Roeder et al. 2011; Hsiao et al. 2013).

4.2.2 Brace response

Peak brace tension force T_{b-NLRH} and compression force C_{b-NLRH} at each tier normalized to their respective design forces T_f and C_f are presented in Table 3. For T/C frames, tension demands in all tiers were significantly higher than the design forces, ranging from $T_{b-NLRH}/T_f = 1.09$ to 1.95. The examination of the brace results, however, confirmed that no tension yielding occurred in braces. The normalized compression demands C_{b-NLRH} were also found to be significantly higher than the demand calculated in design $C_{b-NLRH}/C_f = 1.08 - 1.71$. However, the majority of braces did not

experience post-buckling responses even after several loading cycles. This is due to limited inelastic lateral displacements that the braced frames experienced (Table 2), which resulted in lower axial deformations in the braces and in turn, less pronounced post-buckling response.

For T/O frames, normalized tension forces vary between $T_{b-NLRH}/T_f = 0.97$ and 1.23 (Table 3), suggesting that brace forces exceed their design values. However, no tensile yielding occurred in the braces. The forces in compression-acting braces were normalized by their respective design tension forces to provide insight into the demands induced by T/O braces on the columns. The forces range from $C_{b-NLRH}/T_f = 0.14$ to 0.67, which indicates that although the observed compression forces are relatively small compared to their tension counterparts, they should still be considered in the design of columns, beams, and struts in T/O Type CC MT-CBFs to ensure these members can carry the seismic forces induced by the inelastic response of both tension and compression braces.

4.2.3 Column response

Peak column axial force demands obtained from the NLRHA are given in Table 2. The axial forces were recorded in Tier 1, where the largest axial force occurs as expected in design. Column axial force demands C_{c-NLRH} were normalized by the respective axial compression resistance C_r with $\phi = 1.0$. Normalized axial forces range from 0.97 to 1.28 for T/C frames and 0.94 to 1.12 for T/O frames. The variation of the ratios can be explained by the additional strength that can be attributed to the sizing of columns, resistance factor less than 1.0, and brace forces exceeding design values. The latter significantly contributes to increasing the column axial force demands. The higher column compression forces along with several column instability cases that were observed in this study indicate that the amplification factors recommended in S16 for Type CC frames taller than 15 m, namely the amplification of the design seismic forces and column design demands ($= 1.3$)

must also be considered for Type CC MT-CBFs. However, due to observed high axial compression forces ($C_{c-NLRH}/C_r \geq 1$) across all frames, including those shorter than 15 m, such amplification factors are required for all MT-CBFs regardless of the frame height. It is worth noting that the columns did not experience buckling in several cases even though their axial force ratio exceeded unity, which is attributed to the continuity of the column between tiers (Toutant 2016; Toutant et al. 2017) and distributed axial loads applied along the member length (Dalal 1969).

Peak in-plane and out-of-plane moments of the columns recorded at every strut-to-column joint are plotted in Fig. 5. Each moment value was normalized by the respective plastic moment, M_{py} and M_{px} for in-plane and out-of-plane moments, respectively. In Figs. 5a and 5b, peak in-plane moments vary between $0.01M_{py}$ and $0.08M_{py}$ for T/C frames and $0.01M_{py}$ and $0.06M_{py}$ for T/O frames, respectively. The largest moments were generally observed in the first- or second-tier column segments due to larger differential lateral displacements between bottom tiers creating a kink in the column. As shown in Figs. 5a and 5b, the moments in the frames with non-uniform tiers are higher than those in uniform frames. This is due to the fact that a larger storey shear difference typically exists between the first and second tiers in the frames with non-uniform tiers compared to that in the frames with uniform tiers.

Peak out-of-plane moments of the column, as shown in Figs. 5c and 5d, range between $0.04M_{px}$ and $0.28M_{px}$ for T/C frames, and $0.02M_{px}$ and $0.20M_{px}$ for T/O frames with the maximum moments observed in the first-tier column segments. This can be attributed to greater P- δ effects in the first-tier column segments with a high axial compression force. In addition to P- δ effects, brace out-of-plane buckling, although limited, imposes out-of-plane bending demands on the columns. Slightly lower out-of-plane bending demands observed in T/O frames compared to T/C

frames is due to smaller brace buckling forces in T/O frames as their slender braces buckle under a lower axial force, which then produces smaller out-of-plane forces on the columns.

5 PROPOSED DESIGN METHOD

A seismic design method is proposed here for Type CC MT-CBFs using the results of the NLRHA. To validate the method, the proposed requirements are then applied to eleven case study frames of which eight are not part of the original prototype frames. The details of these new frames are presented in this section.

5.1 Tier drift

The results obtained from the NLRHA confirmed that the maximum tier drifts for T/C and T/O frames are 0.59% and 0.42%, respectively. The results of past experimental studies (Tremblay et al. 2003; Uriz et al. 2008; Fell et al. 2009; Roeder et al. 2011; Hsiao et al. 2013) showed that the fracture life of braces due to low-cycle fatigue for commonly-used slenderness ratios varies between 2.0% and 2.5% inter-storey drift or tier drift in the multi-tiered configuration. Although brace fracture was not explicitly simulated in the numerical model used to perform the NLRHA in this study, the observed tier drifts indicate that braces are not prone to low-cycle fatigue fracture. Thus, there is no need for an additional tier drift check for Type CC MT-CBFs.

5.2 Strut design

The difference between the horizontal components of the peak axial forces in tension- and compression-acting braces of two adjacent tiers produces a compression force in the intermediate horizontal struts. The compression forces induced in the struts of the prototype frames as obtained from the NLRHA were normalized by the adjacent brace design axial force C_{S-NLRH}/T_f and presented in Table 4. The unbalanced horizontal forces induced in the struts are larger in the frames with T/O braces because of the larger difference between the forces induced in the tension-

acting braces and compression-acting braces (e.g., more slender braces are used in T/O frames) that should be resisted by the intermediate strut. This further necessitates the need for horizontal intermediate struts (Fig. 1c vs. Fig. 1d) to avoid unsatisfactory K-braced frame response, which would produce additional in-plane bending on the columns. Thus, horizontal intermediate struts are required for Type CC MT-CBFs. The struts should be designed to resist in compression the combination of the nodal bracing force required to brace the column in-plane, plus a fraction of the design axial force of the adjacent braces that is proposed to be 60% for T/C bracing systems and 80% for T/O bracing systems. The proposed design force for the struts is based on the envelope of peak axial forces obtained from the NLRHA for each system (Table 4).

5.3 Column design

Columns are to be designed to resist the combined effects of an axial compression force, including gravity loads and in-plane and out-of-plane bending moments. The design axial force proposed for Type CC MT-CBFs here is consistent with the proposed approach by S16 for Type CC frames taller than 15 m where the factored seismic forces should be increased by 2% per meter of height above 15 m, and the column design force should be amplified by a factor of 1.3. Referring to the results obtained from the NLRHA for the columns of the prototype frames (Table 2), the first amplification factor is applied for the design of MT-CBFs exceeding 15 m in height. However, it is suggested to consider the second amplification factor to determine column forces for all MT-CBFs regardless of their height. Peak column axial compression forces normalized by the respective column compression resistances (with $\phi = 0.9$) reached the maximum value of 1.42 for T/C frames and 1.24 for T/O frames. An amplification factor of 1.4 is therefore proposed for columns to provide a safeguard for column yielding or buckling due to increased brace forces. Although the peak axial force values obtained from the NLRHA slightly exceed the proposed amplification

factor in some of the frames (e.g., 1.42 vs. 1.4), the proposed axial force amplification factor is deemed acceptable due to the conservative design moments proposed.

Referring to Figs. 5a and 5b, peak normalized in-plane moments for T/C and T/O frames obtained from the NLRHA are 0.08 and 0.04, respectively. The column design in-plane moment is therefore proposed to be 10% of the plastic moment of the column section in the direction of the braced bay. This requirement agrees with the S16 approach for the design of columns in other types of braced frames. A higher in-plane bending demand compared to the observed values from the NLRHA is proposed here to maintain consistency among all frames and retain a sufficient safety margin for cases not studied here.

To determine the design out-of-plane moment demand of the column, the notional load approach adopted by S16 was employed. The out-of-plane column moments obtained from the NLRHA were reproduced by applying an out-of-plane transverse notional load at each strut level with an amplitude equal to a fraction of the axial load in the column segments below the strut. To obtain the amplitude of the out-of-plane transverse notional load, peak out-of-plane transverse notional loads at strut levels were first divided by the respective peak axial compression loads in the columns below the strut as shown in Figs. 4c and 4d for T/C frames and T/O frames, respectively; then, the maximum values of notional loads were obtained as 0.87% and 0.48% for T/C and T/O frames, respectively. The same value of out-of-plane notional load equal to 1.0% times the axial load in the column segments below the strut is proposed at each strut level for both T/C and T/O bracing systems. The proposed notional load based on the results from the NLRHA of the prototype frames is significantly lower than that specified in the current S16 (i.e., 10%).

5.4 Validation of the method

The proposed design method was applied to eleven braced frame case studies (Table 5), three of which T/C F-16-3-1.0, T/C F-24-4-1.0, and T/O F-24-3-1.0 were chosen from the original prototype frame matrix, and the remaining eight frames were selected to further evaluate the proposed seismic design methods when the requirements are applied to the braced frames that are not part of the original prototype frames used to develop the proposed design requirements. These new frames represent Type CC MT-CBFs with a different brace cross-section (double-angle) and geometry (total height $h = 8$ and 20 m; and first-tier to second-tier height ratio $h_1/h_2 = 0.57$) as shown in Table 5. The case study frames were designed in accordance with the new design requirement proposed here. NLRHA was then performed using the same suites of ground motion records used for the response evaluation. The selected member sizes for the case study frames are given Table 5. The numerical model for the T/O frames with double-angle braces is shown in Fig. 3c. Additional details regarding the design of double-angle braces and the development of the numerical model can be found in Derakhshan Houreh and Imanpour (2020).

The statistics of the results obtained from the NLRHA of the prototype frames, including storey drift, tier drifts, column axial force, and column in-plane and out-of-plane moments, are presented in Table 6. Neither column instability nor frame collapse was observed when the proposed design method was applied. Note that T/C F-16-3-1.0 and F-24-4-1.0 had experienced column buckling under 10 and 5 records, respectively, when designed without applying the method proposed here. Referring to Table 6 for the case study frames, the storey drift is considerably smaller than the 2.5% limit specified in the NBCC. Tier drifts are significantly lower than the drift corresponding to low-cycle fatigue failure in HSS or angle braces. For the three frames part of the original prototype frames, lower tier drifts were generally observed when compared to the results

reported for the same frames designed without applying the method proposed here. As shown in Table 6, peak in-plane and out-of-plane moments of columns from the NLRHA, $M_{cy-NLRH}$ and $M_{cx-NLRH}$, do not exceed the respective design moments, M_{fy} and M_{fx} . The comparison between the peak measured moments and the respective design moments indicates that the proposed method for the calculation of the column bending moments results in a conservative estimation of the column seismic demands. This conservatism is built-in to the proposed method to achieve safe and simplified design requirements for Type CC MT-CBFs while protecting columns against instability under seismic loading. Therefore, the results of the NLRHA performed on the selected case study frames suggest that the proposed design requirements can prevent column buckling and improve frame stability response under seismic loading.

5.5 Discussion on the applicability of the proposed design method

The findings presented in this paper should be used within the range of parameters considered here. The following limitations should also be considered when applying the proposed design requirements for Type CC MT-CBFs. The current study is limited to single-storey industrial buildings with well-designed concentrically braced frames with X-bracing. The proposed analysis and design method only considers Type CC MT-CBFs located on Seismic Category 3 in eastern Canada and extrapolating the method to the frames located on moderate seismic regions in western Canada or high seismic areas (i.e., Seismic Category 4) is prone to error. The numerical model developed in this study to evaluate the seismic response of the prototype frames and verify the proposed design method is not capable of simulating brace low-cycle fatigue fracture. Although this limit state does not control the seismic response of the frames designed in this paper, it is a plausible limit state due to large in-plane and out-of-plane deformations induced after column buckling, as in the frames designed neglecting the CSA S16-19 provisions. Finally, the same set of

ground motion records scaled to the 2015 NBCC design level earthquake was used for the evaluation of the seismic response of the frames and validation of the proposed design method.

Future studies should consider different sets of ground motion records to examine the influence of ground motion characteristics on the proposed design requirements. Further NLRHA should be performed to propose a maximum number of tiers to achieve satisfactory seismic response without column buckling or frame collapse. Future research should also investigate the seismic response of Type CC MT-CBFs in high seismic regions as well as moderate seismic regions in western Canada and revisit the proposed design guidelines in this study if needed.

6 SUMMARY AND CONCLUSIONS

This study aimed to assess the seismic behaviour of Type CC MT-CBFs part of a single-storey building in moderate seismic regions of eastern Canada and develop a new seismic design procedure for such frames. A total of 32 MT-CBFs were selected and designed to S16 provisions, excluding the amplification factors in frames taller than 15 m, as well as the out-of-plane moment requirement for columns intersected by bracing members at an unbraced location to assess the need for such requirements. The numerical model of the frames was then developed and used to perform the NLRHA on the selected frames. The frame response was examined using the results obtained from these analyses. The results obtained from the NLRHA were used to propose new seismic design requirements for Type CC MT-CBFs in the framework of the Canadian steel design standard. The proposed requirements were finally applied to eleven case study frames to validate the method.

The key findings obtained from the evaluation of the seismic performance of the Type CC MT-CBFs designed to S16 provisions excluding the abovementioned requirements are as follows:

- Limited inelastic deformations were developed due to brace buckling. Brace tensile yielding was observed in a few frames.
- Multiple cases of column buckling were observed in the 16 m- and 24 m-tall T/C frames.
- Storey drifts of the frames did not exceed the limit prescribed by the NBCC. The maximum value of tier drifts suggests that low-cycle fatigue fracture is unlikely to occur in the braces.
- Higher axial compression forces were induced in the columns as compared to the respective compression resistances. In-plane bending moments, although limited, were produced in the columns due to the non-uniform distribution of frame lateral deformations.
- Out-of-plane moments were observed in the columns as a result of brace out-of-plane buckling and P- Δ effects. Lower moments were observed in frames with T/O braces because of the limited out-of-plane buckling response of their slender braces.

New seismic design requirements proposed using the results of the NLRHA are summarized below. These requirements should be used within the limits of the present study outlined in Section 5.5.

- No tier drift check is required.
- Intermediate horizontal struts should be used between adjacent tiers.
- Lateral seismic forces for the frames taller than 15 m should be increased by 2% per meter of height above 15 m, as prescribed by S16 for other Type CC structures.
- Columns are to be designed to resist an axial compression force due to the effects of gravity loads combined with 1.4 times the member factored axial compression force under seismic loads, an in-plane moment equal to 10% of the plastic moment of the column in the direction of the braced bay, and an out-of-plane moment resulting from an out-of-plane transverse

notional load at each strut level equal to 1.0% of the axial compression load carried by the columns below the strut.

- Eleven case study braced frames of which three frames were part of the original prototype frames, and eight braced frames were selected to represent MT-CBFs with different brace cross-section profiles, total heights, and first-tier to second-tier height ratios were designed in accordance with the proposed method. The seismic response of the case study frames confirmed that the frames perform satisfactorily with no column buckling. The column demands obtained from the NLRHA confirmed that the proposed method can appropriately predict the column seismic demands with a sufficient safety margin.

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27 Table 1: Prototype braced frames, selected member sizes, storey shear resistances and drifts.

Frame	Frame Height (m)	No. of Tiers	Height Ratio	T/C MT-CBFs							T/O MT-CBFs						
				Brace Square HSS	Storey Shear Resistance (kN)	Column W	Strut Square HSS	Beam W	$\frac{R_d R_o \delta_e}{\%h}$	$\frac{\delta_{e,w}}{\%h}$	Brace L	Storey Shear Resistance (kN)	Column Square HSS	Strut Square HSS	Beam W	$\frac{R_d R_o \delta_e}{\%h}$	$\frac{\delta_{e,w}}{\%h}$
F-10-2-1.0 ⁽¹⁾	10	2	1.0	114×4.8 ⁽²⁾	1055	250×58	89×3.2	310×39	0.39	0.01	102×102×15.9	1103	229×12.7	178×6.4	310×39	0.59	0.01
F-10-2-1.75	10	2	1.75	127×4.8 ⁽³⁾ 102×4.8 ⁽³⁾	1057 1025	310×67	89×3.2	310×39	0.40	0.01	102×102×15.9 102×102×12.7	949 1040	229×12.7	178×6.4	310×39	0.66	0.02
F-10-3-1.0	10	3	1.0	102×4.8	1058	360×58	89×3.2	310×39	0.44	0.01	102×102×12.7	1072	229×12.7	178×6.4	310×39	0.76	0.02
F-10-3-1.75	10	3	1.75	114×4.8 102×4.8	1098 1129	250×58	89×3.2	310×39	0.43	0.01	102×102×15.9 102×102×12.7	1145 1141	229×12.7	178×6.4	310×39	0.74	0.02
F-16-2-1.0	16	2	1.0	127×4.8	869	310×79	89×3.2	250×39	0.37	0.02	102×102×15.9	800	305×12.7	140×6.4	250×39	0.58	0.03
F-16-2-1.75	16	2	1.75	127×6.4 102×4.8	885 794	250×101	89×3.2	250×39	0.35	0.02	102×102×15.9 89×89×12.7	652 679	305×12.7	140×6.4	250×39	0.68	0.04
F-16-3-1.0	16	3	1.0	102×4.8	842	360×64	89×3.2	250×39	0.38	0.02	89×89×12.7	714	305×12.7	140×6.4	250×39	0.65	0.04
F-16-3-1.75	16	3	1.75	114×4.8 89×4.8	783 783	310×79	89×3.2	250×39	0.39	0.02	102×102×12.7 102×102×9.5	687 744	305×12.7	140×6.4	250×39	0.68	0.04
F-16-4-1.0	16	4	1.0	88×4.8	807	360×64	89×3.2	250×39	0.41	0.02	76×76×12.7	664	305×12.7	140×6.4	250×39	0.74	0.04
F-16-4-1.75	16	4	1.75	102×4.8 89×4.8	787 864	310×74	89×3.2	250×39	0.39	0.02	89×89×12.7 76×76×12.7	673 705	305×12.7	140×6.4	250×39	0.74	0.04
F-24-3-1.0	24	3	1.0	102×4.8	614	360×122	89×3.2	250×33	0.32	0.05	76×76×12.7	447	356×19	127×6.4	250×33	0.59	0.08
F-24-3-1.75	24	3	1.75	127×4.8 102×4.8	615 740	310×129	89×3.2	250×33	0.32	0.04	89×89×12.7 64×64×12.7	411 413	356×19	127×6.4	250×33	0.65	0.09
F-24-4-1.0	24	4	1.0	89×4.8	638	410×100	89×3.2	250×33	0.33	0.05	64×64×12.7	430	356×19	127×6.4	250×33	0.61	0.09
F-24-4-1.75	24	4	1.75	114×4.8 89×4.8	669 714	360×122	89×3.2	250×33	0.30	0.04	76×76×12.7 76×76×9.5	414 459	356×19	127×6.4	250×33	0.63	0.09
F-24-6-1.0	24	6	1.0	89×3.2	557	410×100	89×3.2	250×33	0.39	0.05	76×76×9.5	510	356×19	127×6.4	250×33	0.63	0.09
F-24-6-1.75	24	6	1.75	89×4.8 89×3.2	622 585	410×100	89×3.2	250×33	0.38	0.05	64×64×12.7 76×76×7.9	421 449	356×19	127×6.4	250×33	0.70	0.10

28 ⁽¹⁾Frame designation is 'F' followed by frame height – number of tiers – tier height ratio

29 ⁽²⁾Brace section is identical for frames with tier height ratio = 1.0

30 ⁽³⁾Brace sections are given for the first tier and other tiers of frames with tier height ratio = 1.75

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Table 2: Maximum of means of peak drift and column response.

Frame	F-10-2-1.0	F-10-2-1.75	F-10-3-1.0	F-10-3-1.75	F-16-2-1.0	F-16-2-1.75	F-16-3-1.0	F-16-3-1.75	F-16-4-1.0	F-16-4-1.75	F-24-3-1.0	F-24-3-1.75	F-24-4-1.0	F-24-4-1.75	F-24-6-1.0	F-24-6-1.75
T/C MT-CBFs																
(δ_{St}/h) (%)	0.39	0.39	0.44	0.44	0.32	0.35	0.25	0.36	0.32	0.31	0.30	0.32	0.30	0.29	0.38	0.38
$\frac{(\delta_{St}/h)}{R_d R_o \delta_e}$	1.00	0.98	1.01	1.02	0.86	0.99	0.66	0.93	0.79	0.79	0.93	1.03	0.95	0.98	0.97	1.00
No. of Column Instability	—	—	—	—	7	2	10	1	3	6	5	3	5	4	—	—
C_{c-NLRH}/C_t	1.08	1.28	0.97	1.02	1.12	1.27	0.99	1.08	1.12	1.11	1.08	1.20	1.11	1.13	1.24	1.23
T/O MT-CBFs																
(δ_{St}/h) (%)	0.34	0.36	0.36	0.35	0.35	0.37	0.36	0.35	0.37	0.37	0.36	0.37	0.36	0.35	0.35	0.35
$\frac{(\delta_{St}/h)}{R_d R_o \delta_e}$	1.32	1.26	1.18	1.21	1.35	1.16	1.28	1.17	1.19	1.21	1.31	1.20	1.33	1.25	1.25	1.11
No. of Column Instability	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
C_{c-NLRH}/C_t	1.07	1.02	1.06	1.06	1.12	1.00	1.11	1.09	1.04	1.05	1.02	0.94	1.01	1.00	1.07	0.98

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Table 3: Maximum of means of peak brace axial forces.

Frame	F-10-2-1.0	F-10-2-1.75	F-10-3-1.0	F-10-3-1.75	F-16-2-1.0	F-16-2-1.75	F-16-3-1.0	F-16-3-1.75	F-16-4-1.0	F-16-4-1.75	F-24-3-1.0	F-24-3-1.75	F-24-4-1.0	F-24-4-1.75	F-24-6-1.0	F-24-6-1.75
T/C MT-CBFs																
$(T_{b-NLRH}/T_f)_1$	1.91	1.77	1.89	1.85	1.49	1.79	1.09	1.63	1.35	1.39	1.55	1.64	1.55	1.64	1.68	1.81
$(T_{b-NLRH}/T_f)_2$	1.76	1.91	1.82	1.95	1.46	1.72	1.11	1.65	1.34	1.32	1.53	1.65	1.54	1.62	1.65	1.70
$(T_{b-NLRH}/T_f)_3$			1.69	1.71			1.09	1.57	1.33	1.33	1.53	1.50	1.54	1.59	1.64	1.66
$(T_{b-NLRH}/T_f)_4$									1.33	1.33			1.53	1.56	1.64	1.66
$(T_{b-NLRH}/T_f)_5$															1.65	1.67
$(T_{b-NLRH}/T_f)_6$															1.65	1.66
$(C_{b-NRLH}/C_f)_1$	1.47	1.56	1.53	1.62	1.39	1.35	1.12	1.28	1.21	1.18	1.14	1.08	1.25	1.24	1.39	1.31
$(C_{b-NRLH}/C_f)_2$	1.57	1.51	1.57	1.63	1.40	1.40	1.10	1.32	1.23	1.26	1.21	1.17	1.26	1.30	1.34	1.42
$(C_{b-NRLH}/C_f)_3$			1.59	1.71			1.12	1.33	1.22	1.24	1.16	1.18	1.26	1.29	1.37	1.43
$(C_{b-NRLH}/C_f)_4$									1.22	1.23			1.26	1.34	1.37	1.42
$(C_{b-NRLH}/C_f)_5$															1.34	1.41
$(C_{b-NRLH}/C_f)_6$															1.40	1.47
T/O MT-CBFs																
$(T_{b-NLRH}/T_f)_1$	1.17	1.20	1.04	1.12	1.19	1.11	1.14	1.16	1.04	1.08	1.23	1.15	1.22	1.23	1.16	1.18
$(T_{b-NLRH}/T_f)_2$	1.16	1.06	1.02	1.02	1.18	1.02	1.12	0.99	1.02	1.03	1.21	1.13	1.20	1.11	1.12	0.98
$(T_{b-NLRH}/T_f)_3$			1.02	1.01			1.12	0.97	1.02	1.03	1.22	1.13	1.20	1.09	1.14	1.01
$(T_{b-NLRH}/T_f)_4$									1.02	1.03			1.21	1.10	1.14	1.00
$(T_{b-NLRH}/T_f)_5$															1.12	0.99
$(T_{b-NLRH}/T_f)_6$															1.14	1.00
$(C_{b-NLRH}/T_f)_1$	0.50	0.36	0.58	0.54	0.34	0.22	0.35	0.32	0.31	0.31	0.18	0.14	0.17	0.15	0.35	0.16
$(C_{b-NLRH}/T_f)_2$	0.50	0.55	0.58	0.65	0.34	0.32	0.36	0.52	0.31	0.35	0.18	0.16	0.17	0.29	0.35	0.33
$(C_{b-NLRH}/T_f)_3$			0.58	0.67			0.35	0.52	0.31	0.36	0.18	0.16	0.17	0.28	0.36	0.34
$(C_{b-NLRH}/T_f)_4$									0.31	0.35			0.17	0.28	0.35	0.34
$(C_{b-NLRH}/T_f)_5$															0.35	0.34
$(C_{b-NLRH}/T_f)_6$															0.35	0.33

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Table 4: Maximum of means of peak strut axial forces.

Frame	F-10-2-1.0	F-10-2-1.75	F-10-3-1.0	F-10-3-1.75	F-16-2-1.0	F-16-2-1.75	F-16-3-1.0	F-16-3-1.75	F-16-4-1.0	F-16-4-1.75	F-24-3-1.0	F-24-3-1.75	F-24-4-1.0	F-24-4-1.75	F-24-6-1.0	F-24-6-1.75
T/C MT-CBFs																
$(C_{S-NLRH}/T_f)_{1,2}^{(1)}$	0.26	0.56	0.28	0.50	0.05	0.55	0.01	0.54	0.11	0.31	0.23	0.62	0.21	0.54	0.28	0.55
$(C_{S-NLRH}/T_f)_{2,3}$			0.20	0.22			0.01	0.26	0.10	0.07	0.22	0.32	0.20	0.25	0.25	0.23
$(C_{S-NLRH}/T_f)_{3,4}$									0.09	0.09			0.20	0.21	0.22	0.21
$(C_{S-NLRH}/T_f)_{4,5}$															0.26	0.22
T/O MT-CBFs																
$(C_{S-NLRH}/T_f)_{1,2}$	0.51	0.66	0.40	0.51	0.51	0.62	0.58	0.61	0.61	0.68	0.63	0.71	0.74	0.76	0.67	0.73
$(C_{S-NLRH}/T_f)_{2,3}$			0.38	0.33			0.58	0.38	0.59	0.59	0.62	0.66	0.73	0.63	0.66	0.58
$(C_{S-NLRH}/T_f)_{3,4}$									0.59	0.59			0.74	0.63	0.66	0.58
$(C_{S-NLRH}/T_f)_{4,5}$															0.66	0.57

⁽¹⁾Intermediate strut between Tier 1 and Tier 2

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Table 5: Case study frames and selected member sizes.

	Frame	Frame Height (m)	No. of Tiers	Height Ratio	Brace	Column	Strut	Beam
T/C MT-CBFs	F-10-3-0.57	10	3	0.57	HSS114×114×4.8 ⁽²⁾ HSS102×102×4.8	W310×74	HSS89×89×3.2	W310×38.7
	F-16-2-0.57	16	2	0.57	HSS127×127×6.4 HSS102×102×4.8	W360×110	HSS89×89×3.2	W250×38.5
	F-16-3-1.0	16	3	1.0	HSS102×102×4.8 ⁽³⁾	W360×110	HSS89×89×3.2	W250×38.5
	F-24-4-1.0	24	4	1.0	HSS102×102×4.8	W460×177	HSS89×89×3.2	W250×32.7
	F-24-6-0.57	24	6	0.57	HSS139×139×3.2 HSS102×102×3.2	W460×158	HSS89×89×3.2	W250×32.7
T/O MT-CBFs	F-8-2-1.0	8	2	1.0	L127×127×12.7	HSS254×254×12.7	HSS178×178×6.4	W310×44.5
	F-20-4-1.75	20	4	1.75	L102×102×9.5 ⁽⁴⁾ L64×64×12.7	HSS457×457×15.9	HSS127×127×6.4	W250×32.7
	F-24-3-1.0	24	3	1.0	L89×89×12.7	HSS457×457×19	HSS127×127×6.4	W250×32.7
T/O MT-CBFs – 2L ⁽¹⁾	F-10-3-1.75	10	3	1.75	2L76×51×11.1 2L76×51×9.5	HSS305×305×12.7	HSS178×178×6.4	W310×38.7
	F-16-4-1.0	16	4	1.0	2L76×51×7.9	HSS356×356×15.9	HSS152×152×6.4	W250×38.5
	F-24-3-1.0	24	3	1.0	2L76×64×7.9	HSS457×457×19	HSS127×127×6.4	W250×32.7

⁽¹⁾T/C MT-CBFs with double-angle braces⁽²⁾Brace sections are given for the second tier and other tiers of frames with tier height ratio = 0.57⁽³⁾Brace section is identical for frames with tier height ratio = 1.0⁽⁴⁾Brace sections are given for the first tier and other tiers of frames with tier height ratio = 1.7538
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Table 6: Maximum of means of peak frame response for case study frames.

Frame	T/C MT-CBFs					T/O MT-CBFs			T/O MT-CBFs – 2L ⁽¹⁾		
	F-10-3-0.57	F-16-2-0.57	F-16-3-1.0	F-24-4-1.0	F-24-6-0.57	F-8-2-1.0	F-20-4-1.75	F-24-3-1.0	F-10-3-1.75	F-16-4-1.0	F 24-3-1.0
(δ_{St}/h) (%)	0.40	0.35	0.34	0.30	0.33	0.33	0.36	0.33	0.39	0.34	0.32
(δ_{St}/h)	0.97	1.05	1.01	1.02	0.96	0.56	0.69	0.56	0.64	0.66	0.54
$R_d R_o \delta_e$											
(δ_1/h_1) (%)	0.30	0.32	0.30	0.24	0.26	0.32	0.33	0.30	0.33	0.31	0.29
(δ_2/h_2) (%)	0.49	0.37	0.36	0.30	0.25	0.35	0.37	0.34	0.45	0.34	0.32
(δ_3/h_3) (%)	0.38		0.36	0.33	0.35		0.38	0.35	0.45	0.36	0.34
(δ_4/h_4) (%)				0.34	0.40		0.39			0.36	
(δ_5/h_5) (%)					0.41						
(δ_6/h_6) (%)					0.39						
C_{e-NLRH}/C_r	0.70	0.79	0.74	0.65	0.76	0.67	0.45	0.54	0.53	0.65	0.53
$(M_{cy-NLRH}/M_{py})_{max}$	0.05	0.02	0.01	0.01	0.02	0.02	0.03	0.01	0.05	0.03	0.01
$M_{cy-NLRH}/M_{fy}$	0.50	0.19	0.15	0.10	0.26	0.21	0.26	0.11	0.53	0.25	0.10
$(M_{cx-NLRH}/M_{px})_{max}$	0.05	0.06	0.05	0.03	0.02	0.08	0.03	0.04	0.05	0.04	0.04
$M_{cx-NLRH}/M_{fx}$	0.79	0.83	0.45	0.34	0.29	0.97	0.30	0.51	0.57	0.36	0.56

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⁽¹⁾T/C MT-CBFs with double-angle braces

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Figure 1: a) Three-tiered CBF; b) Single-panel CBF; c) Single-storey two-tiered CBF with intermediate strut; d) Single-storey two-tiered CBF without intermediate strut; e) Generic MT-CBF elevation.

Figure 2: a) Plan view of the selected building; b) Acceleration response spectra of the scaled ground motion records.

Figure 3: Fibre-based numerical model of MT-CBF: a) T/C frame; b) T/O frame with single-angle braces; c) T/O frame with double-angle braces (two-tiered frame shown).

Figure 4: a) Peak tier drifts for T/C frames; b) Peak tier drifts for T/O frame; c) Profile of peak normalized out-of-plane transverse notional loads for T/C frames; d) Profile of peak normalized out-of-plane transverse notional loads for T/O frames.

Figure 5: Peak column response: a) In-plane moments for T/C frames; b) In-plane moments for T/O frames; c) Out-of-plane moments for T/C frames; d) Out-of-plane moments for T/O frames.

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