Seismic Response of Two-Bay Steel Multi-Tiered Concentrically Braced Frames

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Abstract. This paper investigates the possibility of using multi-tiered concentrically braced frames in two adjacent column bays to resist seismic loads. Three prototype frames part of a single-storey building were chosen and designed using current knowledge of multi-tiered behaviour. The columns were selected to resist in-plane bending and axial loads arising from tensile yielding and compression buckling of braces in critical tiers. The lateral response of the frame was then examined using the nonlinear response history analyses under ground motion accelerations. The analyses confirmed that all frames exhibited nonuniform brace tensile yielding between tiers, which resulted in the concentration of inelastic drifts in the uppermost tiers. Peak storey drift values remained under 2.5%, although higher than the design predictions, which influenced the prediction of column in-plane bending.

Keywords: Multi-tiered · Multi-bay · Steel · Braced frame · Seismic response

1 Introduction

Multi-tiered concentrically braced frames (MT-CBFs) are often used as a lateral load resisting system in tall single-storey buildings such as industrial facilities, airplane hangars, and warehouses. MT-CBFs consist of multiple braced tiers stacked along the height of the storey (Fig. 1) as it typically becomes impractical to brace the entire storey height in such buildings. Moreover, the configuration involves shorter braces in multiple tiers, resulting in more effective braces in compression. Horizontal intermediate struts are placed between each tier to redistribute unbalanced force developed after brace buckling and prevent unsatisfactory K-brace response. The struts can also be used as intermediate in-plane support for columns at every tier level, while columns lack out-of-plane support over the full frame height. In Canada, MT-CBFs in two adjacent bays (two-bay MT-CBFs) are permitted only when using the same geometry and brace cross-sections in both bays [1]. However, other geometries including two dissimilar MT-CBFs located in two adjacent bays can occur in practice as shown in Fig. 1.

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Fig. 1. Example of a two-bay two-tiered CBF.

Past studies have extensively studied the seismic behaviour of single-bay MT-CBFs, and demonstrated that under seismic loads, brace tensile yielding only occurs in one of the braced tiers, concentrating inelastic deformations in the weakest (critical) tier [2, 3]. This, in turn, produces in-plane flexural demands on the columns.

This article presents the seismic behaviour of two-bay three-tiered concentrically braced frames. Attention is given to drift demands, as well as in-plane flexural demands in the columns. A design procedure adapted from the seismic design requirements of the Canadian steel design standard (CSA S16) [1] is first described for prototype frames. Nonlinear response history analyses are then performed on the selected frames and the results are analysed and presented.

2 Frames Studied

2.1 Building Geometry and Loading

Three three-tiered steel concentrically braced frames with two adjacent braced bays were selected in this study. The same 13 m-tall, single-storey building was selected to design the three braced frames, with only the braced bays varying in width. The building plan dimensions are 35 × 84 m and it is located on a site class C in Vancouver, BC. The frames are placed on the perimeter walls parallel to the long side of the building. Figure 2 shows the frames along with the sections selected for braces, columns, and struts as well as the bay and tier dimensions. The bay dimension was used to designate the frames, i.e., the frame having 6- and 8-m wide bays is labelled as Frame 6–8. Tiers 1, 2, and 3 refer to the position of a given tier relative to the ground. The left-hand-side (LHS) and right-hand-side (RHS) columns are referred to as exterior columns, while the column in between the two braced bays is named the interior column.

The building is of the normal risk category, with an importance factor $I_E = 1.0$. Type MD (Moderately Ductile) CBF with ductility- and overstrength-force modification factors, $R_d = 3.0$ and $R_o = 1.3$, respectively, were considered. The loading was performed in accordance with the 2015 NBC [4]. Dead load = 1.35 kPa, Live load = 1.0 kPa, and Snow load 1.64 kPa at the roof level were considered, along with an exterior cladding load of 1.0 kPa. This results in a frame tributary seismic weight of 6751 kN. The design base shear was calculated using the analytical period, which was

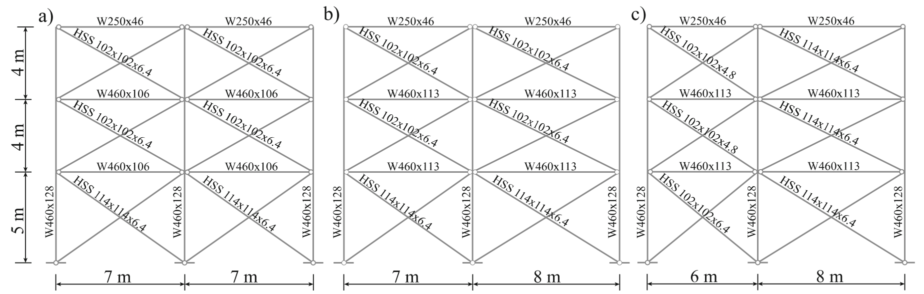

Fig. 2. Three-tiered two-bay CBFs with selected member sections: a) Frame 7–7; b) Frame 7–8; and c) Frame 6–8.

equal to 0.554 s, 0.530 s, and 0.526 s for Frame 7–7, Frame 7–8, and Frame 6–8 respectively. This resulted in a design base shear of 934 kN, 951 kN and 954 kN for Frame 7–7, Frame 7–8, and Frame 6–8, respectively.

2.2 Braced Frame Design

The braces were first selected from square HSSs conforming to ASTM A1085 with yield stress F_y = 345 MPa. The total design base shear for the frame was distributed between the two braced bays as a function of their respective lateral stiffness as provided by the braces. The effective length factor K was taken equal to 0.45 to account for the length of the end gusset plates and the support provided at mid-length by the tension-acting brace. The probable resistances of the braces were determined using the probable yield strength of $R_y F_y$ = 460 MPa.

The roof beam, struts and columns were selected from wide flange members conforming to ASTM A992 steel with F_y = 345 MPa. The roof beam was designed to carry the distributed gravity load and the axial load resulting from the tension brace in Tier 3 reaching its probable tensile resistance, T_{prob}, and the compression brace in this same tier reaching its probable post-buckling compressive resistance, C'_{prob}. Similarly, struts were designed under the unbalanced axial load occurring when the tension braces in the tiers above and below the strut reach their respective values for T_{prob}, while the compression braces reach their probable buckling resistance, C_{prob}, in the tier below or above and C'_{prob} in the other tier, depending on the condition that results in the highest axial load. The same section was used for all four struts.

Columns were oriented such that strong axis bending occurs out-of-plane. Both exterior and interior columns were designed under a loading condition at which all the tension-acting braces reach their respective T_{prob} while all the compression-acting braces reach their respective C_{prob}. The same wide flange section was selected for both exterior and interior columns as this would typically be the case in practice to reduce the number of connection types and ease fabrication and erection process.

Similar to single-bay MT-CBFs, non-uniform yielding of braces along the frame height must be taken into account in design. This is achieved by using a non-linear static analysis where the displacement corresponding to the design storey drift,

$\Delta_m = R_d R_o \Delta_d$, ($\Delta_d$ represents the elastic displacement at the roof under the design base shear) is applied at the roof. In the analysis, the tension-acting brace in the critical tier was replaced with the force corresponding to its probable tensile resistance and the compression-acting brace in the same tier was replaced with its probable post-buckling resistance. While only compression-acting braces in all other tiers were replaced with their respective compression resistances while tension-acting braces in these tiers remain in-place as they are expected to respond in the elastic range. The critical tier corresponds to the tier that has the lowest storey shear resistance, V_{prob}, when its braces reach their probable resistances. The location of the critical tier in two-bay MT-CBFs depends on the frame geometry and brace resistances. For Frame 6–8, the lowest storey shear resistance is 1060 kN and occurs in both Tiers 2 and 3 of the LHS bay. Because of inherent variability in the material strength or erection tolerances, either Tier 2 or 3 can be critical and must be examined in design. The scenario associated with critical Tier 3 in the LHS bay is evaluated first. Figure 3 shows the condition where braces reach their probable resistances as descried earlier at the roof displacement Δ_m = 75.7 mm corresponding to the design storey drift. At the applied displacement, the tension-acting brace in RHS bay Tier 3 also reached its resistance T_{prob} = 1214 kN, suggesting that both bays in the third tier are critical. The brace force scenario was then updated as shown in Fig. 3b to reflect this condition. Using the brace force scenario of Fig. 3b, an axial force C = 1969 kN including the effects of gravity loads (C_g = 160 kN) and an in-plane bending moment M_y = 20.4 kN-m were obtained. A concomitant out-of-plane bending moment resulting from a notional load applied at each strut level with an amplitude of 2% of the column axial load below the strut level was also considered. A W430 × 128 column was finally selected to carry the combined effects of axial force, in-plane and out-of-plane moments. The critical LHS Tier 2 and LHS Tiers 2 and 3 scenarios were then investigated, taking into account the possibility of propagation of tensile yielding to adjacent tiers as it was done for the scenario associated with critical Tier 3 in the LHS bay. These demands were finally used to verify the column.

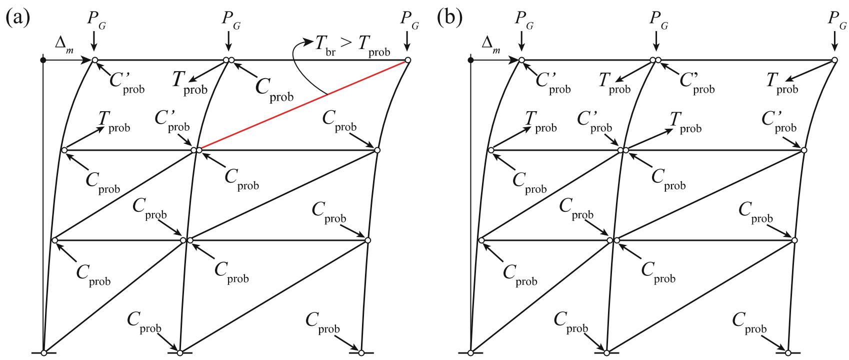

Fig. 3. Frame 6–8 brace force scenario for: a) LHS bay Tier 3 is critical; and b) both bays in Tier 3 are critical.

3 Seismic Response Evaluation

3.1 Numerical Model

The numerical model of the two-bay MT-CBF was developed in the *OpenSees* [5] program based on the modelling technique and assumptions proposed in [3, 6]. The steel material was defined with Young's modulus $E = 200$ GPa, and yield strength $F_y = 345$ MPa for columns and struts and probable yield strength $R_yF_y = 460$ MPa for braces. Force-based nonlinear BeamColumn elements with the *Steel02* material were selected to account for the Bauschinger effect and simulate kinematic and isotropic hardening behaviour of steel [7, 8]. The columns were pinned at their bases and restrained at the roof level in the out-of-plane degree of freedom. Initial geometric imperfections were assigned to brace and column members using the corotational formulation. Braces were assigned sinusoidal out-of-plane imperfections with a maximum amplitude of 0.002 times the member unbraced length. Columns were assigned bi-directional in-plane and out-of-plane sinusoidal imperfections with a maximum amplitude equal to 0.001 times the respective unbraced length of the column. Point masses corresponding to the frame tributary seismic weight were specified at the top end of the columns. A P-Delta column carrying the gravity loads of the gravity load-carrying system tributary to the braced frame was included. The Rayleigh damping method with mass proportional damping set to 2% of critical in the first mode of vibration was used to reproduce classical viscous damping. Gravity loads were applied to the braced frame and to the P-Delta and nonlinear response history analysis (NLRHA) was performed.

A total of 15 ground motions (GMs) records were selected and scaled to match, on average, the NBC design spectrum for Vancouver, BC using Method A of the NBC Commentary J [9]. The selected records include three suites of five GMs each representing a distinct tectonic source contributing to the seismic hazard in the west coast of Canada namely, crustal, interface subduction, and in-slab subduction earthquakes. Each suite is covers a specific period range, and together they cover a period range of 0.2 to 2 s.

3.2 Frame Response

The Statistics of the peak of key response parameters as obtained from NLRHA are visually shown in Fig. 4. For each frame, median values together with the average of the five most critical values are presented. Drift ratios correspond to the maximum demand from each GM record at tier and storey heights. In-plane bending in the exterior and interior columns were normalized by the weak axis plastic moment of the section, M_{py}.

As shown, all three frames exhibit similar drifts and columns experienced nearly identical in-plane bending response. For all records and frames, no column buckling was observed. The expected design storey drift Δ_{m} for each frame correlates well with the median drift obtained from NLRHA. However, when observing the average of the five maximum values corresponding the most demanding GMs, inelastic deformations tended to concentrate in Tier 3 for all three frames. This resulted in higher in-plane bending demands in exterior and interior columns at the tier levels, with maximum demand $0.095M_{py}$ observed at Tier 3 level. Nonetheless, storey drifts remained well under 2.5% and tier drift never exceed 2%.

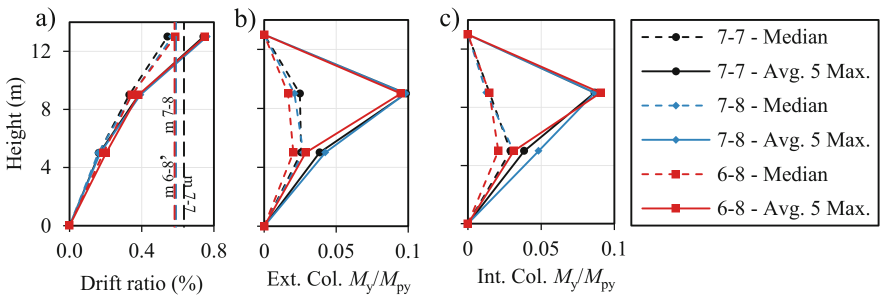

Fig. 4. Median and peak frame response parameters for two-bay three-tiered CBFs: a) drift ratio; b) in-plane bending moments in exterior columns; and c) in-plane bending moments in interior columns (frames labelled using their bay width).

Figure 5 shows the history of storey and tier drifts for Frame 6–8 under the 1989 Loma Prieta – Hollister Differential Array record. This ground motion was selected as it produces the largest storey drift for all the frames studied. At 8.7 s, the maximum storey drift of 0.9% is attained, which induces a Tier 3 drift of 1.7%. After experiencing several inelastic cycles and brace tensile yielding, residual drifts were observed in Tier 3. Storey drifts remained well under 2.5% and tier drift never exceed 2%.

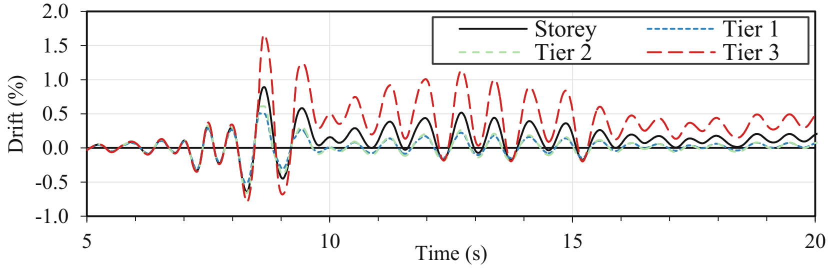

Fig. 5. Storey and tier drifts for frame 6–8 under the 1989 Loma Prieta – Hollister differential array record.

Figure 6 shows the hysteretic response of the continuous braces under the 1989 Loma Prieta – Hollister Differential Array record. The braces in Tier 3 underwent a large inelastic cycle, slightly exceeding their probable tensile resistance ($1.06T_{prob}$). In the LHS bay, braces in Tiers 1 and 2 just reached their respective T_{prob} before unloading. In the RHS bay, braces in Tiers 1 and 2 never exceeded $0.93T_{prob}$, which stems from the fact that smaller cross-sectional areas were selected for the braces in the LHS bay compared to the ones in the RHS bay because of their shorter effective length.

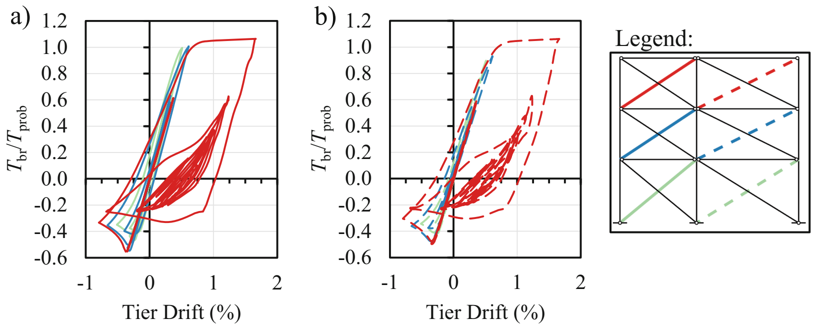

Fig. 6. Brace axial forces of frame 6–8 under 1989 Loma Prieta-Hollister differential array record.

3.3 Evaluation of the Design Method

The global response observed using the NLRHA for the three frames studied matched well the prediction by design provisions. The average of the five maximum storey drift values from the NLRHA was on average 27% larger than the maximum expected design drift, which led to higher in-plane bending in the columns as compared to design. Moreover, the design procedure relies strongly on the Δ_m value to predict if multiple tiers will be critical. When using a larger expected storey displacement in design, some additional critical tier scenarios might arise in which multiple tiers are critical, hence increasing the in-plane bending demands in the columns. Frame 6–8 was redesigned using an increased storey displacement of Δ_{max} = 116 mm obtained from the 1989 Loma Prieta – Hollister Differential Array record. The design scenario remained the same, with Tier 3 in the LHS and RHS bays being critical, although with higher in-plane bending moment and axial force in the columns due to the amplified Δ_m. For the exterior columns, the ratios between the in-plane column moments obtained from NLRHA and those predicted in design were equal to 0.97 and 0.86 at a height of 5 m at a frame height of 9 m respectively. The same ratios for the interior column were equal to 0.81 and 0.86 respectively. With the redesign, the prediction of the column moment has improved because of the use of a more accurate displacement in design, especially for moments at the intersection of Tiers 1 and 2.

4 Conclusions

This paper investigated the seismic behaviour of two-bay MT-CBFs using the NLRHA method. Three braced frames were selected and designed using the seismic provisions of the Canadian steel design standard. The frames were then analysed under a suite of 15 GM accelerations. The results of NLRHA confirmed that the storey drifts remain below the code-specified limit of 2.5%. The average of the five most critical values showed that inelastic displacement can concentrate in the critical tiers but does not exceed the 2% limit prescribed by CSA S16–19. In-plane moments in exterior and interior columns were affected by brace tensile yielding in braced tiers and reached a maximum value of $0.095M_{py}$. No column buckling was observed under the GM records studied. The comparison between the results obtained from the analyses and design predictions indicated that the current design method can predict well the location of the critical tier, and the amplitude of column in-plane moments. However, the expected storey drift is underestimated compared to the average of the five largest storey drifts from NLRHA, which may result in lower in-plane moment demands on interior and exterior columns. This can be addressed by using an increased roof displacement in computing column moments where such roof displacements can be obtained from NLRHA results. Future studies should examine the seismic response of a broader set of two-bay MT-CBFs, which may happen in practice, to further enhance the design method for such frames.

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