Tenth U.S. National Conference on Earthquake Engineering
Frontiers of Earthquake Engineering
July 21-25, 2014
Anchorage, Alaska

SEISMIC PERFORMANCE EVALUATION AND DESIGN OF MULTI-TIERED STEEL CONCENTRICALLY BRACED FRAMES

A. Imanpour[1] and R. Tremblay[2]

ABSTRACT

This research investigates the seismic response of multi-tiered concentrically steel braced frames. 3- and 5-tiered X-braced frames with moderate ductility are designed according to current Canadian code provisions for steel structures. Nonlinear seismic response of the braced frames is examined through nonlinear dynamic analyses to study the influence of the design parameters including the number of bracing panels and the use of out-of-plane notional load in design. The focus is on the in-plane seismic demand imposed on the columns when buckling and yielding of the bracing members occur. Out-of-plane stability of the columns is also investigated. The results show that columns designed in accordance with the current provisions improves the 3-tiered braced frame response and induces uniform, less critical, ductility demand on the braces due to the columns' high in-plane flexural strength and stiffness. Higher ductility demand is induced in the frame with 5 tiers due to non-uniform vertical distribution of brace yielding. An alternative design method that explicitly accounts for column in-plane bending moment demands and ensures proper distribution of the inelastic demand is proposed.

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A. Imanpour and R. Tremblay. Seismic Performance Evaluation and Design of Multi-Tiered Steel Concentrically Braced Frames. *Proceedings of the 10th National Conference in Earthquake Engineering*, Earthquake Engineering Research Institute, Anchorage, AK, 2014.

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This research investigates the seismic response of multi-tiered concentrically steel braced frames. 3- and 5-tiered X-braced frames with moderate ductility are designed according to current Canadian code provisions for steel structures. Nonlinear seismic response of the braced frames is examined through nonlinear dynamic analyses to study the influence of the design parameters including the number of bracing panels and the use of out-of-plane notional load in design. The focus is on the in-plane seismic demand imposed on the columns when buckling and yielding of the bracing members occur. Out-of-plane stability of the columns is also investigated. The results show that columns designed in accordance with the current provisions improves the 3-tiered braced frame response and induces uniform, less critical, ductility demand on the braces due to the columns' high in-plane flexural strength and stiffness. Higher ductility demand is induced in the frame with 5 tiers due to non-uniform vertical distribution of brace yielding. An alternative design method that explicitly accounts for column in-plane bending moment demands and ensures proper distribution of the inelastic demand is proposed.

Introduction

In North America, multi-tiered braced frames (MT-BFs) are commonly used to brace tall single story structures or structures with large story heights. Industrial buildings, airplane hangars, or warehouse buildings are among those buildings for which lateral loads are generally resisted using multi-tiered braced frames. Typical MT-BFs in these buildings consist in concentrically steel braced frames with two or more bracing tiers stacked between the ground and roof levels (Fig. 1a). Multi-tiered braced frames represent an economical and practical option for such buildings as smaller braces with lower slenderness ratio and reduced probable resistances can be used in each tier compared to larger sections when braces span over the full height of the frame. Moreover, using single bracing members over the full building height is often impractical in tall structures. In MT-BFs, columns are typically I-shaped members oriented such that strong axis bending due to wind loading take places out-of-plane. Although the columns are laterally braced in the plane of frame by intermediate horizontal struts, they are laterally unsupported for out-of-plane buckling along the full height of the frame.

Numerical simulations show that brace inelastic response typically does not distribute

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uniformly along the height of MT-BFs under a strong seismic event, even if the braces are well proportioned to provide uniform story shear resistance over the frame height [1-4]. As shown in Fig. 1b for a 3-tiered BF, inelastic deformations tend to concentrate in the tier where brace tension yielding takes place first. This tier is referred to as the critical tier. This behavior raises two main concerns regarding the utilization of the MT-BFs in seismic areas: likelihood of large inelastic ductility demand on the braces of the critical tier, which may cause premature fracture of the bracing members due to low-cycle fatigue, and column in-plane or out-of-plane instability due to large in-plane bending demands induced by the non-uniform drift demand along the frame height (Fig. 1b). In 2009, special design provisions have been introduced for the first time in CSA S16 [5] for the seismic design of MT-BFs, with focus on the columns. According to these provisions, the framing configuration is limited to Type LD (limited ductility) braced frames that are designed with a ductility-related force modification factor $R_d = 2.0$. Moderately ductile (Type MD) braced frames designed with $R_d = 3.0$ are not allowed for MT-BFs, the reason being to limit the inelastic deformation demand in the bracing members. A horizontal strut must be provided between each tier to resist the brace horizontal unbalanced loads that can develop at the tier points due to the difference between tension and compression brace forces after the compression braces have buckled and the tension braces have yielded. The presence of struts allows the transfer of the brace unbalanced forces down to the foundations by truss action in the braced frame, rather than by flexure of the columns as is the case in K-type braced frames not permitted in CSA S16. Column axial forces and in-plane bending moments must be determined for the most critical brace loading scenario where brace yielding and buckling concentrate in anyone of the tiers [2]. Additionally, a notional out-of-plane transverse load equal to 10% of the compression load carried by the members meeting at the brace-to-column intersecting points must be applied at these points. These loads account for the out-of-plane forces and deformations that may develop upon brace buckling.

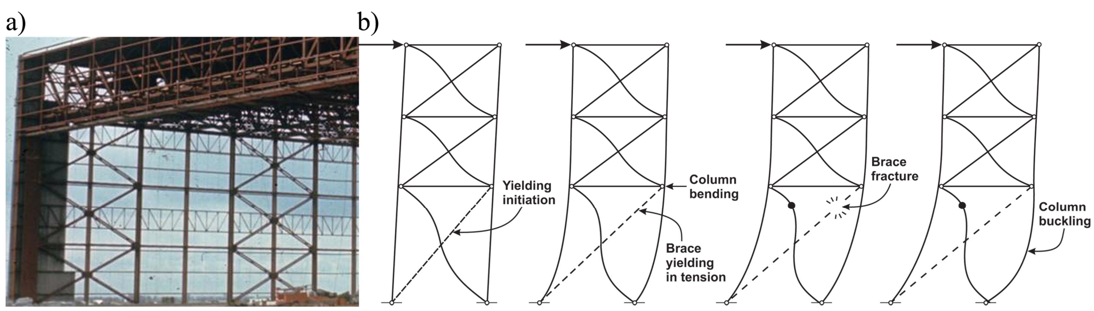

Figure 1. a) 3-tiered concentrically steel braced frame (Courtesy of the Canadian Institute of Steel Construction), and b) Concentration of inelastic deformations in a multi-tiered braced frame inducing high inelastic demand on bracing members and in-plane flexural demand on columns.

A preliminary study has been performed on the 2-, 3- and 4-tiered braced frames having different heights and designed in accordance with the provisions specified in CSA S16-09 for Type MD and LD braced frames [2-3]. The results confirmed that non-uniform distribution of the lateral displacements over the frame height induces in-plane bending moments in the columns that exceed the flexural demand specified in CSA S16-09 for ordinary multi-story braced frames. That in-plane flexural demand increases when increasing the number of tiers and the R_d factor. Although higher when compared to the same frames designed with $R_d = 2.0$, the deformation and force demands in frames designed with $R_d = 3.0$ were still acceptable. The results also showed that the columns experience very limited out-of-plane bending moment demand, significantly less than the bending moments induced by the 10% notional transverse load specified in CSA S16-09. The column out-of-plane bending moments were found to vary between 1% and 2% of the strong axis plastic moment, M_{pcx}, for the frames designed with CSA S16-09. One main conclusion of this study was that a more realistic and accurate design procedure that reflects more closely the actual response of the system had to be developed. Changes are being considered for the next edition of the standard: allowing MT-BFs for the Type MD braced frame category with $R_d = 3.0$, provided that the number of tiers is limited to 3; limiting the number of tiers to 5 for Type LD braced frames with $R_d = 2.0$, and reducing the notional out-of-plane transverse loading to 2% of the factored axial compression load in the columns below the brace-to-column intersecting points.

This paper presents a complementary study performed to examine the limit on the number of tiers proposed for MT-BFs designed as Type MD frames with $R_d = 3.0$ and specifically investigate the influence of the number of tiers and the out-of-plane notional load on the seismic performance of MT-BFs. The 3- and 5-tiered braced frame examples with concentrically X-bracing panels are selected to study the effects of these parameters. Three different amplitudes of the out-of-plane notional loads are considered in the design of the 3-tiered frame to examine the possibility of using a smaller out-of-plane load for design of MT-BFs. The 5-tiered frame has the same total height as the 3-tiered frame, which permits to isolate the effect of the number of tiers on the seismic response, particularly the stability response of the columns. In each case, roof drifts, drift demands in the critical tier and in-plane and out-of-plane bending moment demands on the columns as obtained through nonlinear dynamic analysis are examined and compared to assess the design assumptions.

Frame Studied

Geometry & Loading

A tall single-story, clear span industrial building with 119 m x 56 m plan dimensions was selected for this study. The building height, H, is equal to 20 m. The spacing of the exterior columns is 7 m and four MT-BFs are placed in each direction (two BFs per wall). X-bracing is used to form 3- or 5-tiered BFs. The elevations of the frames are presented in Fig. 2. For both frames, the height of the first tier (Tier 1) is relatively larger and the remaining of the 20 m height is equally distributed among the other tiers.

The building is located on a class C site in Vancouver, British Columbia. The design earthquake loads are determined in accordance with the 2010 National Building Code of Canada (NBCC) [6]. The design roof dead load and snow load are equal to 1.2 and 1.64 kPa respectively. The columns of the braced frame support 56 m roof trusses that span over the full width of the building, resulting in column gravity loads $P_D = 235$ kN and $P_S = 321$ kN. Ductility-related and overstrength-related seismic force modification factors of 3.0 and 1.3 were used, as specified in NBCC 2010 for Type MD concentrically braced frames. The equivalent static force procedure was used to calculate the seismic load, and accidental torsion was considered to calculate the story shear resisted by the braced frames.

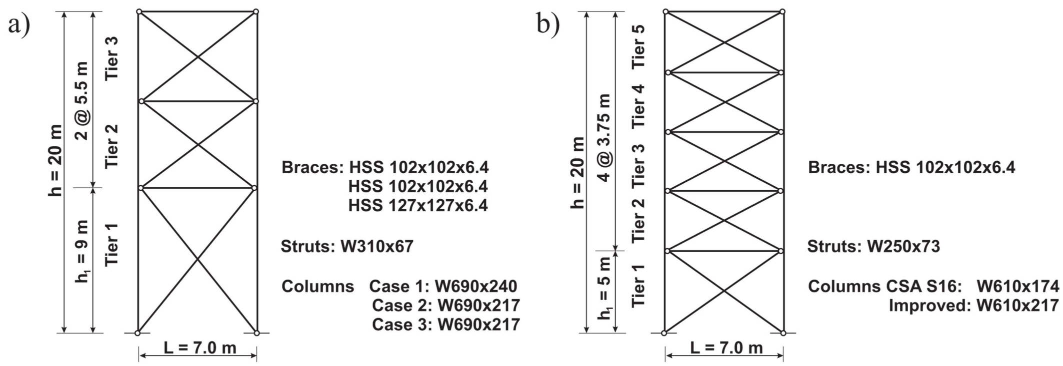

Figure 2. Elevation of the: a) 3-tiered, and b) 5-tiered braced frames.

Design of the 3-Tiered X-braced Frame in accordance with Canadian Steel Code

Design of the three-tiered braced frame in Fig. 2a is presented here to illustrate the design procedure in accordance with CSA S16 and the parameters used to define the column demands. The columns are W shapes oriented such that in-plane bending occurs about their weak axis. They are assumed to be pinned at their top and bottom ends for bending about both directions.

The building fundamental period is equal to 1.0 s, resulting in a design spectral acceleration (S) of 0.33 g and a design story shear per braced frame V = 398 kN. The bracing members are the first components to be designed. The braces in each tier are designed to resist in tension and compression the seismic story shear, V. For this frame, the resulting brace axial compression forces are equal to 324 kN in Tier 1 and 253 kN in Tier 2&3. Gravity induced compression brace forces of 18 and 13 kN are combined to these seismic effects. The braces are designed for compression assuming an effective length factor K = 0.45, taking into account the size and fixity of the brace end connections and the mid-support provided by the intersecting tension-acting braces. The braces are selected from available ASTM A500, grade C, (F_y = 345 MPa) HSS members. The selected braces must satisfy the requirements of CSA S16-9 regarding the brace slenderness as well as width-to-thickness ratios limits. The selected braces are shown in Fig. 2a and their properties are given in Table 1. The probable brace resistances in tension, T_u, in compression, C_u, and in the post-buckling range, C'_u, are computed using the probable steel yield strength for HSS members, R_yF_y = 460 MPa.

Table 1. Brace properties for the 3-tiered BF.

Tier	Braces	KL (mm)	A (mm^2)	C_r (kN)	T_u (kN)	C_u (kN)	C'_u (kN)	V_u (kN)
2&3	HSS 102x102x6.4	4006	2170	305	998	394	200	1095
1	HSS 127x127x6.4	5131	2770	384	1274	496	255	1087

Columns and struts must be designed using the brace axial forces assuming that yielding develops in the tension acting bracing member located in the critical tier. The identification of the critical tier is done by comparing the horizontal shear capacity of all tiers as obtained based on the brace probable resistances. In frames with uniform, or nearly uniform tier properties and tier shear capacity, all tiers could potentially be critical tiers and all possible critical tier scenarios must be considered to define the column demands. For the frame studied, the horizontal shear capacities of the tiers, V_u, are given in Table 1. As shown, the capacity of Tier 1 is close to that of Tier 2 or Tier 3, so that all tiers could be critical. Since the same column cross-section is used over the full frame height and compression load is maximum in Tier 1, the first tier is chosen as the critical tier to obtain the combination of maximum axial force and flexural in-plane bending moments in this tier.

The frame deformed shape and brace axial forces for the case where Tier 1 is critical are shown in Fig. 3. Two brace force scenarios are shown, one when tension yielding is reached in Tier 1 and one when the frame reaches the anticipated story drift including inelastic effects ($R_d\,R_o\,\delta_e$) [4]. For both scenarios, it is assumed that brace tension yielding develops only in the critical tier (Tier 1). In the first scenario in Fig. 3a, all compression acting braces have just buckled and the tension brace in the critical tier has developed its resistance T_u. The drifts are still small and degradation of the brace compressive strengths has just started. For simplicity, all compression braces are assigned their expected compression strength, C_u, neglecting the post-buckling strength reduction in the upper tiers, the brace tension forces are determined such that the story shear contributed by the two braces is equal to the story shear V_u resisted by the critical tier.

In Fig. 3b, large inelastic deformations have taken place in the critical tier and the compression brace in that tier has reached its post-buckling compression strength, C'_u. In all other tiers, the drifts are still small and degradation of the brace compressive strengths is small and can be neglected, so the brace compression strength is maintained equal to C_u. The brace tension force in the critical tier is still equal to T_u. In Tiers 2 & 3, the brace tension force is adjusted so that the story shear is equal to the story shear resisted by the critical tier with brace forces in that tier equal to C'_u and T_u.

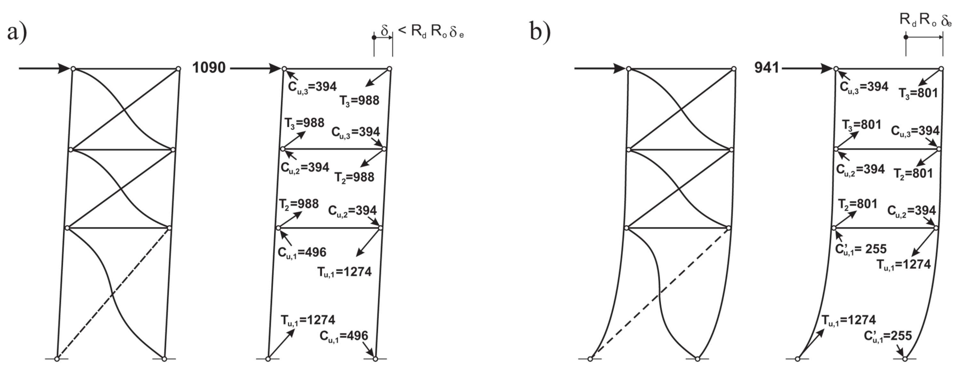

Figure 3. Brace axial forces: a) At initiation of brace tension yielding in Tier 1, and b) At anticipated story drift including inelastic effects (forces in kN).

The struts are provided between adjacent panels and at the roof level. The struts are assumed as pin-connected to the columns. They are made from W shapes oriented with the web in the horizontal plane such that they can effectively resist horizontal wind loads and provide torsional bracing at tier levels. A maximum axial compression force of 473 kN occurs in the strut at Tier 1 under the load scenario shown in Fig. 3b. A W310x67 was selected to resist this force.

The columns are designed to resist the axial forces that are induced by the gravity loads together with the forces induced by the bracing members. The column axial compression forces in Tier 1 resulting from the two brace force scenarios in Fig. 3 are 2706 kN and 2475 kN, respectively.

The gravity load induced compression force is added to these seismic effects for the load combination E + D + 0.25S, which gives total factored axial compression forces of 3021 kN and 2791 kN. As prescribed in CSA S16-09, in-plane bending moments induced by non-uniform inelastic tier deformations and transverse notional loads at tier joints must also be considered. For the scenario shown in Fig. 3a, when tension brace at Tier 1 reaches yielding, significant inelastic deformations have not developed yet and it can be assumed that the columns are still straight and in-plane bending demand is neglected. For the second scenario, in-plane bending demands as induced by the frame lateral deformation pattern must be determined. For this calculation, the cross-section area and moment of inertia of the columns must be known and a column section is first selected to resist the axial load from the first scenario. Using that trial section, the frame lateral deformations and column bending moments for the scenario of Fig. 3b are determined and the column is verified under combined axial and flexural demand. The column section is then adjusted as necessary and the process is repeated until convergence is reached.

As prescribed in NBCC 2010, the total roof displacement including inelasticity effects is determined by multiplying the elastic roof displacement under the code specified base shear, δ_e, by the factors R_dR_o: $\delta_{max} = R_dR_o\delta_e$. For the frame studied with the final column cross-section, δ_e from computer analysis is equal to 31 mm and $\delta_{max} = 3.0\times1.3\times31 = 120$ mm (= 0.60% H). It is noted that axial deformations of the two columns and all bracing members have been taken into account in the calculation of the elastic lateral displacement of the frame. Once the total frame inelastic drift is known, the horizontal displacements at the top of Tier 1 and 2 can be back calculated by removing from the roof displacement δ_{max} the lateral displacements due to the elastic axial deformations of the columns under the axial loads shown in Fig. 3b and the relative lateral displacements of tiers 2 and 3 caused by the elastic axial elongation of the tension braces in these tiers using the brace forces shown in Fig. 3b. These calculations lead to tier drifts equal to 77, 21, and 22 mm, respectively, for Tiers 1 to 3. In-plane bending moments in the column at the tier points under that deformation pattern can be computed using the equations given in Imanpour et al. [3] and a maximum in-plane bending moment of 43 kN-m is found at the top end of Tier 1 columns. Alternatively, this result can be obtained using a static incremental (push-over) analysis where the roof displacement would be gradually increased up to $\delta_{max} = 120$ mm.

The columns under loading scenario in Fig. 3b must also be verified under a concomitant out-of-plane bending moment resulting from the transverse notional loads applied at the top of Tier 1 and Tier 2. These notional loads are taken equal to 2% of the axial compression load in Tier 1 and Tier 2, respectively, as is proposed for the upcoming CSA S16 edition. For this frame, the resulting out-of-plane bending moment is maximum and equal to 360 kN-m at the lowest brace-to-column intersecting point, at the same location where the maximum in-plane bending demand and axial compression load exist. A W690x240 column section is required to resist the combined demands. The factored axial compression, strong and weak axis bending moment resistances for this section are respectively equal to 3582 kN, 2171 kN-m, and 556 kN-m.

Influence of the Out-of-Plane Notional Load

In the previous design example, an out-of-plane notional load equal to 2% of the factored axial compression load in the columns below the brace-to-column intersecting points was used. This value is lower than the design notional load of 10% that is currently prescribed in CSA S16-09.

In this section, the possibility of using reduced out-of-plane flexural demand for the design of MT-BF columns is studied by examining the following three cases for the 3-tiered BF example: 1) notional load equal to 2% of the factored axial compression load in the columns below the brace-to-column intersecting points, 2) out-of-plane bending moment equal to 4% of the column strong axis plastic moment, M_{pcx}, and 3) columns designed without out-of-plane bending demand. For Case 2, 4% of M_{pcx} is selected based on the moment demand observed in the past studies. The member sizes are given in Fig. 2a for the three cases. As shown, the same section was required for Cases 2 and 3 and that column size is smaller than the one selected in Case 1.

Nonlinear Time History Analysis

A numerical model was created using the *OpenSees* program [7] to assess the nonlinear seismic response of the frame example and evaluate the out-of-plane demand on the columns. The bracing members were modelled to reproduce axial buckling and yielding responses. Column flexural buckling could also be reproduced with the model. Detail of the modeling and analysis assumptions are given in Imanpour et al. [1, 4]. A series of ten ground motions were selected and scaled based on the work by Atkinson [8]. The ground motions were scaled such that the average value of the response spectra did not fall below the NBCC 2010 spectrum for periods ranging from 0.2T to 1.5T, where T is the fundamental period of the structure [6].

For three cases studied, no in-plane or out-of-plane instability was observed for the columns in the response history analyses. The column section, the axial-bending interaction equation result, the column out-of-plane deflection, the out-of-plane flexural bending demand, and the ratio of the out-of-plane strong axis bending from design to average value from analysis are given in Table 2 for the three cases studied. For the first case, the out-of-plane bending demand on the columns is much less than what was considered in design, indicating conservatism. For the second case, the column out-of-plane bending demand is very close to design value of 0.04 M_{pcx}. The results obtained for the third case indicate that out-of-plane bending moments cannot be neglected when designing the columns in MT-BFs.

Table 2. 3-tiered braced frame: column section and out-of-plane bending demand.

3-tiered BF	Case 1	Case 2	Case 3
Column Section	W690x240	W690x217	W690x217
Interaction Equation Result	1.00	1.01	0.97
$\Delta_{out\text{-}of\text{-}plane}$ / h (%)	0.030	0.035	0.035
$M_{cx\text{-}analysis}$ / M_{pcx}	0.025	0.030	0.030
$M_{cx\text{-}design}$ / $M_{cx\text{-}analysis}$	5.39	1.35	-

Finite Element Analysis of an Isolated Column

In order to study the stability condition of a column in MT-BFs when it is subjected to seismic demands, a three-dimensional finite element model of an isolated column was developed using the *Abaqus* FEA software [9]. The W690x217 column section of Case 2 or 3 was selected for this study. Column demands including axial force and lateral displacement were obtained from the nonlinear analysis of the frame in *OpenSees* under the El Centro (0˚ comp.) record from the 1987 Superstition Hills earthquake. That record induced the highest drift ductility demand in the bracing members of Tier 1 and consequently, the highest in-plane bending demand in the columns. The finite element model and boundary conditions considered for the selected column are illustrated in Fig. 4a. In that analysis, the maximum roof drift is equal to 1.26%, which is equal to two times the value assumed in design, and the maximum drift in Tier 1 is equal to 203 mm. The column was modeled using four-node shell elements with reduced integration (S4R). Material nonlinearities were specified through the von Mises yield criterion with associated flow rule. Isotropic strain hardening was used in the material model to simulate steel cyclic behavior. Geometric nonlinearities were incorporated in the models through use of a large-displacement formulation. Column is torsionally fixed in tier level to simulate the torsional restraint provided by the struts. Additional detail is given in Imanpour et al. [4].

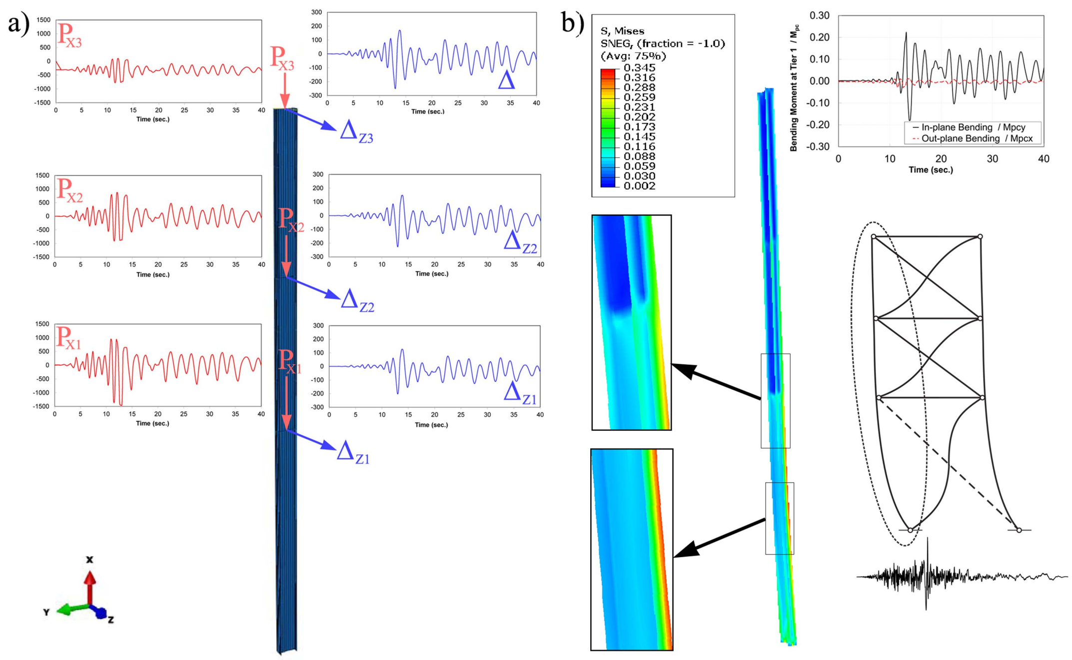

Figure 4. Finite element analysis of an isolated MT-BF column: a) Model and boundary conditions, b) Von-Mises stress contours (kN/mm^2) at t = 13 s, and bending demands.

The results of the analysis showed that the column remains stable under the demands applied. No in-plane or lateral-torsional buckling was observed during the analysis. Fig. 4b shows the Von-Mises stress contours for the column studied at t = 13 s. At this time, the drift in tier 1 is equal to 2.3% and the in-plane moment in the column reaches 0.29 M_{py}. As shown, the large drifts together with the axial compression forces and bending moments caused yielding of the column segment in Tier 1. Column buckling did not occur because a second plastic hinge did not form at the upper end of the Tier 1 as would be needed to initiate column instability.

Influence of the Number of Tiers

A 5-tiered steel braced frame (Fig. 2b) with the same height and characteristics as the 3-tiered frame example was selected to study the effect of the number of bracing panels on the seismic performance of MT-BF frames. The ductility demand on the bracing members of the critical tier and the stability of the columns are investigated. The same design procedure was followed with design out-of-plane moment set equal to 4% of M_{pcx} as confirmed from the nonlinear analyses of the 3-tiered braced frame (see Table 2). For this 5-tiered braced frame, Tier 1 is also the critical tier, and it is expected that the inelastic frame deformation will concentrate in that tier.

Nonlinear Time History Analysis

The results of the nonlinear response history analyses for the 5-tiered BF are presented in Table 3. Detailed results for the 3-tiered (Case 2) are also given in this table for comparison. For both frames, the average peak story drift exceeds the anticipated roof drift as specified in NBCC 2010 and the lateral deformations concentrated in the first tier (critical tier) for all ground motions. For 5-tiered braced frame, the average critical tier drift increases from 1.23 to 1.71% and the maximum value reaches 3.31%. A recent study shows that such drift level in critical tier can produce large ductility demand on the bracing members capable of causing brace fracture [10]. In the table, the drift ratio compares the critical tier drift to the roof drift and reveals the non-uniformity of the lateral deformation along the height of frame. As shown, the 5-tiered BF shows larger drift variations compared to the 3-tiered BF. For both frames, the ratio of the design in-plane bending moment to the average in-plane bending from analysis ($M_{cy\text{-}design}$ / $M_{cy\text{-}analysis}$) is maximum at Tier 1 and the values are also given in Table 3. Values less than 1.0 suggest that the in-plane flexural demand calculated according to the seismic design procedure of CSA S16-09 is underestimated.

Table 3. Statistics of peak frame response from nonlinear time history analyses.

	3-tiered BF		5-tiered BF		5-tiered BF - Improved	
Parameter	Mean	Range	Mean	Range	Mean	Range
Total story drift (%)	0.80	(0.55 – 1.26)	0.78	(0.53 – 1.19)	0.74	(0.42 – 1.13)
Roof displacement / $R_d R_o \delta_e$	1.31	(0.90 – 2.04)	1.18	(0.80 – 1.78)	0.92	(0.67 – 1.78)
Critical tier drift (%)	1.23	(0.64 – 2.25)	1.71	(0.68 – 3.31)	1.47	(0.47 – 2.95)
Drift ratio	1.47	(1.16 – 1.79)	2.03	(1.28 – 2.79)	1.95	(1.11 – 2.79)
$M_{cx\text{-}design}$ / $M_{cx\text{-}analysis}$	1.35		0.89		1.18	
$M_{cy\text{-}design}$ / $M_{cy\text{-}analysis}$	0.58		0.62		1.38	

Special Design Requirements for the Columns

Drift can be reduced in the critical tier and more uniform drift demand can be achieved over the height of an MT-BF by making use of the continuity of the columns along the frame height. If the columns are provided with sufficient in-plane flexural strength and stiffness, they can trigger tension yielding of the braces in tiers adjacent to the critical tier and, thereby, reduce the ductility demand on the bracing members in the critical tier and moments imposed on the columns.

This approach can be used to reduce the critical tier drift to an acceptable level, and better assess the in-plane bending demand of the columns for the 5-tiered braced frame. For instance, applying this method, the columns were redesigned such that the critical tier drift does not exceed 1.5% when the story drift is equal to $R_dR_o\delta_e$. The new column sections for the improved 5-tiered BF are shown in Fig. 2b. The results of the nonlinear response history analyses for this frame are presented in Table 3. From nonlinear response history analysis, the average critical tier drift is reduced from 1.71% to 1.47% for the improved design and the in-plane bending demand is well predicted when these special design requirements are utilized.

Conclusion

- The results of numerical studies showed that the out-of-plane bending demand on the columns of MT-BFs is smaller than the value resulting from the notional transverse load specified in CSA S16. For the 3-tiered BF, no out-of-plane (strong axis) buckling was observed in the finite element study of the isolated column when 0.04 M_{pcx} was used for the column design.
- Non-uniform distribution of the nonlinear lateral deformation over the frame height imposes large inelastic demand in the bracing member of critical tier and induces in-plane bending moments in the columns.
- For the frames studied, in-plane bending demand increased with the number of tiers. For the 5-tiered BF, tier drifts reached values that may cause HSS brace fracture and the bending moments exceeded the values calculated according to the seismic provisions of CSA S16-09.
- Excessive drift can be reduced and column in-plane flexural demand can be better predicted by using a design method where minimum flexural strength and stiffness are provided for the columns to achieve a more uniform seismic drift demand over the height of MT-BFs.

Acknowledgments

Funding from the Natural Sciences and Engineering Research Council (NSERC) of Canada is acknowledged. The authors would like to express their thanks to Prof. Ali Davaran for his assistance in development of the numerical model of the bracing member.

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