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Evaluation of Seismic Design Methods for Steel Multi-Tiered Special Concentrically Braced Frames

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Abstract

Steel Multi-Tiered Concentrically Braced Frames (MT-CBFs) are commonly used in North America as a lateral load-resisting system of tall single-story buildings. Multi-Tiered configurations are typically used when a single braced panel between the roof and ground levels is impractical in the case of tall building. Past studies show that MT-CBF columns designed in accordance with the 2010 US Seismic Provisions are prone to buckling due to a high axial compression force and in-plane bending demands induced in columns as a result of non-uniform distribution of brace inelastic deformations along the frame height. Special design provisions have been introduced in the current US Seismic Provisions to address flexural demands imposed on MT-CBF columns and prevent column instability. Nevertheless, the recent improvements lack full-scale experimental testing and comprehensive finite element simulations featuring current guidelines. In this study, the seismic design methods for Multi-Tiered Special Concentrically Braced Frames are evaluated using the nonlinear finite element method. First, a two-tiered SCBF was designed in accordance with the 2010 and 2016 requirements. A detailed finite element model of the frames was then created and a non-linear cyclic-pushover analysis was performed to evaluate the seismic performance of both frames. Special attention was paid to the validation of the design forces prescribed in the 2016 Seismic Provisions. Analyses results confirmed that the inelastic deformations in the frame designed using the 2010 requirements are not uniformly distributed but rather concentrated in one of the tiers; whereas, the current design method significantly reduces the concentration of inelastic deformations in a single tier and prevents column instability. Furthermore, it was found that column in-plane flexural demands are overestimated when designed in accordance with the current provisions. Further numerical simulations on a large number of frame configurations using the dynamic response history analysis procedure are required to verify this observation.

1. Introduction

Multi-tiered Concentrically Braced Frames (MT-CBFs) are widely used in North America as a lateral-resisting system of tall single-storey buildings such as airplane hangars, recreational facilities, shopping centers, and industrial buildings. MT-CBFs consist of multiple bracing tiers along the height of the frame separated by horizontal strut members. Various bracing

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configurations are used in MT-CBFs including Chevron, V-shape, diagonal and X-bracing. Multi-tier arrangements are typically used when it is not practical to use a single bracing panel along the height of the frame. By introducing multiple bracing tiers stacked on top of each other (Fig. 1), the length of braces is reduced resulting in a lower slenderness ratio, which allows for smaller brace sizes. From the seismic design perspective, the limits on width-to-thickness and global slenderness ratios can be easily satisfied when using shorter braces. Moreover, reduced brace sizes result in smaller design forces on the adjacent members including struts, beams, columns and connections. MT-CBF columns are typically W-shape oriented such that out-of-plane bending moments act about the section major-axis. The columns can be considered braced in the plane of the frame as a result of horizontal struts; however, no out-of-plane bracing exists at the tier level and the column buckling length is equal to the full frame height in this direction.

Figure 1: Typical steel single-story industrial building with a multi-tier configuration

The 2010 AISC Seismic Provisions (AISC 2010), did not include design provisions for MT-CBFs. In the absence of special design provisions, MT-CBFs were designed based on the provisions prescribed for multi-story steel braced frames. However, past numerical studies showed that ductile MT-CBFs designed in accordance with the 2010 provisions are prone to column instability due to the concentration of inelastic deformations in a single tier rather than a uniform distribution of such deformations along the height of the frame (Imanpour et al. 2013; Imanpour et al. 2016a). Consequently, the non-uniform deformations induce in-plane flexural demands on the columns that can lead to yielding of the column, which in turn may result in column flexural-torsional buckling (Imanpour et al. 2016a). The 2016 AISC Seismic Provisions (AISC 2016a) have introduced requirements specifically for multi-tiered special concentrically braced frames to address such unsatisfactory response by considering in-plane bending moment demands in addition to column axial forces in design. Additionally, these provisions require the out-of-plane bending demand be considered in design to account for the moments induced by the brace out-of-plane buckling and plastic hinge forming in the gusset plate. Although significant improvements have been made in the design of MT-CBFs, there is still lack of validation studies for the current design requirements for various frame configurations, in particular, the in-plane and out-of-plane bending moment demands should be carefully examined and design forces or methods should be improved if necessary.

This paper aims to examine the seismic design provisions for Multi-Tiered Special Concentrically Braced Frames (MT-SCBFs) designed in accordance with the 2010 and 2016 Seismic Provisions. A review of current and previous seismic design provisions together with MT-CBF response is first given. Then, the seismic design of a case study frame is presented. The numerical model used to perform the nonlinear analyses is demonstrated. The response of the frames is evaluated using a detailed nonlinear finite element model using a cyclic-pushover analysis. Finally, the results of the nonlinear analyses are compared with the design values, in particular, the flexural demands imposed on columns are discussed.

2. Seismic Response of MT-CBFs

The seismic response of MT-CBFs is driven by the inelastic behavior of their braces. The response of a two-tiered braced frame under lateral seismic load is described here. A two-tiered concentrically braced frame is shown in Fig. 2(a). Under the lateral load, compression braces buckle first in both tiers Fig. 2(b); by increasing the load, the tensile yielding occurs only in one of the tiers along the height of the frame (e.g. lower tier in Fig. 2(c)), which reduces the shear resistance of that tier. The difference between the shear resistances of the adjacent tiers induces an unbalanced shear force on the columns. Such force develops large in-plane bending moments in the columns. Tensile yielding is initiated in the tier which has the lowest shear capacity; this tier is referred to as critical tier. Even if tiers are identical, slight variations between the initial geometric imperfections or material properties can create a difference in shear resistance of tiers, thus resulting in yielding of the tension brace in one of the tiers (Imanpour et al. 2013). If the columns do not possess sufficient strength and stiffness, the concentration of inelastic deformations in a single tier can lead to column yielding where the combination of axial force and moments is critical, which in turn may lead to column buckling and even frame collapse (Imanpour et al. 2016a).

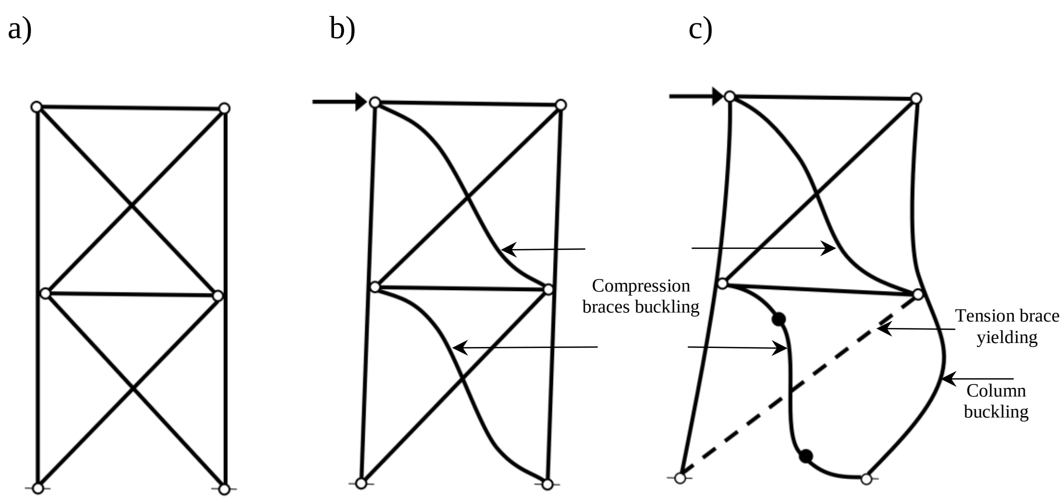


Figure 2: (a) Undeformed braced frame; (b) Deformed shape of brace frame upon buckling of the compression braces; (c) Concentration of inelastic drift in bottom storey caused by yielding of tension brace

There were no special design guidelines for the design of MT-SCBFs in the 2010 AISC Seismic Provisions. Two analyses cases prescribed for SCBFs were previously used to determine the forces in the columns, struts, and connections of such frames. Cases 1 and 2 are shown in Figs. 3(a) and (b). Recent numerical simulations demonstrated that seismic demands of MT-CBF columns are different than multi-story steel concentrically braced frames, which, if not considered in the design, may result in column instability (Imanpour et al. 2014; 2016a).

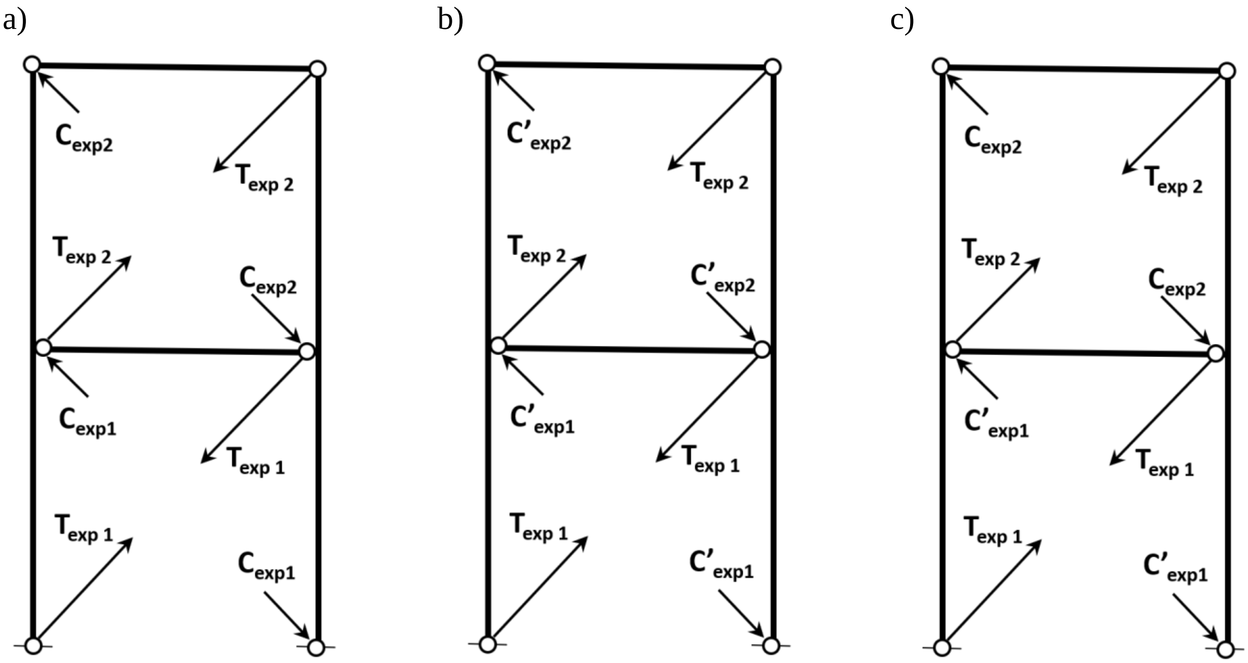


Figure 3: (a) Analysis case 1: tension braces reach their expected tensile strength T_{exp} and compression braces reach their expected buckling strength C_{exp}; (b) Analysis case 2: tension braces reach their expected tensile strength T_{exp} and compression braces reach their expected post-buckling strength C'_{exp}; (c) Analysis case 3: progressive yielding and buckling of braces starting at the critical tier (lower tier): T_{exp} and C'_{exp} in the critical tier (lower tier) and T_{exp} and C_{exp} in the non-critical tier (upper tier)

Various limit states have been identified in the columns of multi-tier braced frames subjected to seismic loading. These limit states include flexural-torsional buckling due to due to biaxial moment demands and flange and web local buckling (Stoakes and Fahnestock 2014; Imanpour et al. 2016a). The 2016 Seismic Provisions have introduced guidelines and requirements for designers to prevent such limit states in ordinary and special MT-CBFs. As per the current provisions, special multi-tiered braced frames should be analyzed under a new analysis case introduced in Chapter F of AISC Seismic Provisions. This analysis case represents the progressive yielding and buckling of braces in MT-SCBF, which corresponds to the initiation of tensile yielding in the weakest tiers and propagation to the strongest. This analysis case is shown in Fig. 3(c) for a two-tiered frame of Fig. 2(a). For this analysis, it is assumed that the compression brace in the critical tier (lower tier) has reached its post-buckling strength, C'_{exp}, and the compression brace in the adjacent tier has reached its expected buckling strength, C_{exp}. Concurrently, the tension braces in the critical tier and in the adjacent tier are assumed to be at their expected tension strength, T_{exp}. The unbalance story shear force can be determined by analyzing this frame under applied brace loads. Then, the column in-plane bending demand can be computed for design purposes. This estimate is considered conservative since it is possible that the brace in the adjacent non-critical tier has already experienced several loading cycles, which leads to a decrease in buckling strength at the time when the tension brace yields in the critical tier (Tremblay 2002; Imanpour and Tremblay 2014).

An out-of-plane bending moment can also be induced in the columns of MT-SCBFs due to initial geometric imperfections in columns, out-of-plane buckling of braces, and plastic hinge forming in the gusset plate. To account for such demands, the 2016 AISC Seismic Provisions requires a horizontal notional load be applied on the column at the strut level. The notional load is equal to 0.006 times the vertical component of the compression brace that meets the column at the tier level. This notional load should be amplified by the B_1 factor (AISC 2016b) to account for the P-δ effect. In addition, the columns must be designed to resist the out-of-plane moment that the braces produce upon buckling, but less than the maximum bending resistance of the brace connections. Given that column out-of-plane demand is affected by several concomitant responses, further studies are needed to accurately quantify and formulate such demands. Additionally, MT-SCBF columns should be torsionally braced at the strut-to-column connections. Stoakes and Fahnestock (2014) showed that providing rotational bracing, along the height of the column at the strut-to-column connections, can improve the strong-axis buckling strength in the presence of in-plane flexural yielding, particularly when the location of weak-axis flexural moment matches the location of the strong-axis flexural moment (e.g. two-tier braced frame with identical tier heights). The 2016 AISC Seismic Provisions also require that a strut be placed between two adjacent tiers to prevent the unsatisfactory K-brace frame response. Finally, the provisions have established a maximum tier drift ratio of 2% to prevent excessive brace deformations that can cause brace fracture (Tremblay et al. 2003).

3. Building Configuration and Design Loads

A single-story steel building located in Seattle, WA was selected as a case study. The building has plan dimensions of 35 m x 189 m, and the height of the building is 9 m. In each principal direction, the building has four concentrically braced frames (two per each exterior walls). The frame height is divided into two tiers with X-bracing configuration as shown in Fig. 4(a). As illustrated, the bottom tier, *Tier 1*, is 4.7 m tall, and the top tier, *Tier 2*, is 4.3 m tall. The purpose of having tiers of different heights is done to predict the critical tier, simplifying the design procedure. Specially concentrated braced frame (SCBF) was selected for the frame and the braces were designed to carry the seismic load in tension and compression.

The design loads for the selected building were determined in accordance with the ASCE 7-10 standard (ASCE 2010). A Risk Category II was chosen and it was assumed that the building is located on a Site Class C with a Seismic Design Category D. The seismic load parameters include a response modification factor R of 6.0, overstrength factor Ω_o of 2, and a deflection amplification factor C_d of 5.0. The mapped risk-targeted Maximum Considered Earthquake (MCE_R) ground motion response parameters, S_S = 1.362g and S_1 = 0.458g for short and 1.0 s periods, respectively, were used to obtain the design spectral response acceleration parameter S_{DS} = 0.908g and S_{D1} = 0.458g. The empirical fundamental period was calculated using C_t = 0.0488 and x = 0.75, and is equal to T_a = 0.507 s. Using these values, the seismic design coefficient C_s = 0.15 was obtained. The seismic weight of the building W is equal to 7623 kN, based on a roof dead load D_{roof} = 1.0 kPa, and the exterior wall dead load D_{wall} = 0.5 kPa. The equivalent lateral force procedure was used to calculate the frame seismic base shear V, which is the product of the seismic coefficient and the seismic weight. This force was amplified to account for accidental torsion, resulting in a seismic design base shear per frame equal to 316 kN.

The gravity loads were calculated using a live load, L = 0.96 kPa, in addition to the dead load of the roof and exterior walls. The tributary area considered per column was calculated on the basis that steel roof trusses support the roof system between the exterior columns of the building. The resulting gravity factored load at the top of each column was then calculated to be 287 kN.

The height of each column is 9 m. The columns are made of W-shapes and oriented such that the out-of-plane bending occurs about the section strong-axis. A 7 m horizontal strut is placed between tiers to prevent K-braced frame response and ensure the seismic load is appropriately transferred to the base the structure through the truss action once the brace respond in inelastic range.

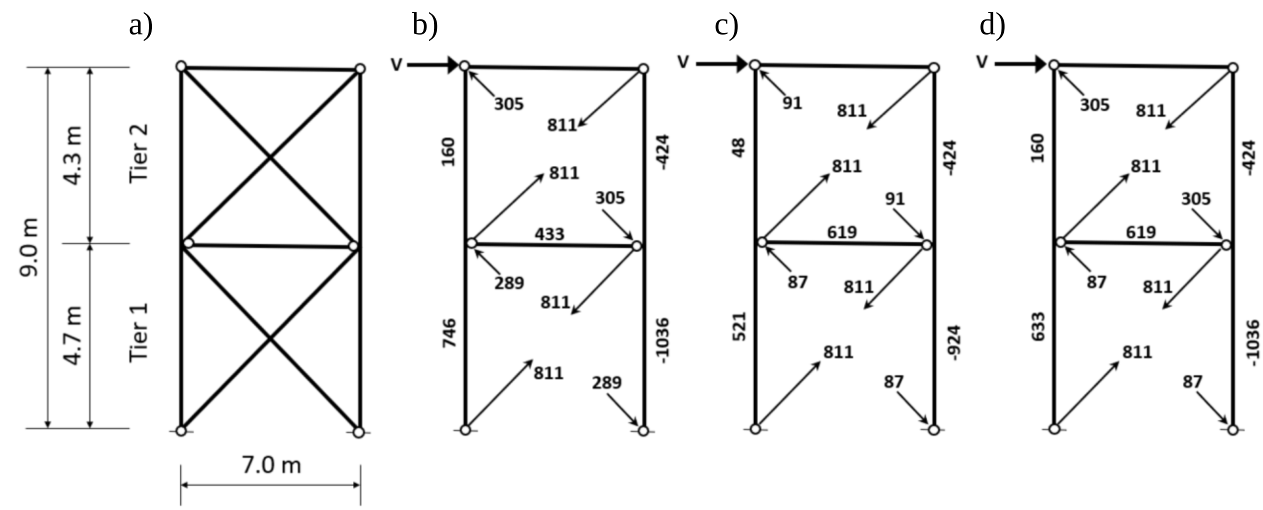


Figure 4: (a) Frame geometry; (b) Analysis case 1 under brace strength T_{exp} and C_{exp}; (c) Analysis case 2 under brace strength T_{exp} and C'_{exp}; (d) Analysis case 3 under T_{exp} and C_{exp} in the critical Tier 1 and T_{exp} and C'_{exp} in non-critical Tier 2 (forces are in kN)

4. Brace Frame Design

The design of members was performed in accordance with the AISC Specifications (AISC 2016b) and AISC 341 Seismic Provisions (AISC 2010; 2016a). This section summarizes the main design steps and member sizes for braces, columns and struts.

4.1 Brace Design

The braces were designed to resist the seismic load effects in tension and compression. The brace design force in compression is equal to $P_{r,b}$ = 199 kN, which includes the seismic induced axial force $P_{E,b}$ = 190 kN plus the gravity induced axial compression force $P_{G,b}$ = 9 kN. The braces are designed using square HSS sections. Such members are more efficient than single-symmetric sections as they have an identical radius of gyration about both principal axes of the section (Black et al. 1980). The braces are made of ASTM A1085 steel with a yield stress F_y = 345 MPa and an expected yield stress R_yF_y = 431MPa. Braces were designed such that they buckle out of the plane of the frame. An effective length of 0.45 times the total length of the brace, which is measured between the brace working points, was used in design to account for the lateral bracing provided by the brace acting in tension. The brace axial compression resistance obtained from AISC

Specifications is $P_{c,b}$ = 228 kN. Although the brace lengths are slightly different in tiers, an identical HSS 88.9x88.9x6.4 section was selected for the braces in both tiers. The selected section complies with the width-to-thickness ratio limit b/t < 14 as required for highly ductile members.

4.2 Column Design

The columns were first designed in accordance with the 2010 AISC Seismic Provisions. The frame with the selected columns is referred to as 2010 design. The columns were designed to resist the gravity loads $P_{G,c}$ = 287 kN plus the maximum axial load induced by the summation of the vertical forces due to the brace resistances in tension and compression. For the later, two analysis cases are prescribed by the 2010 AISC Seismic Provisions as shown in Figs. 4(b) and (c). The maximum axial compression force, $P_{E,c}$ = 1036 kN, was obtained under the first analysis case. The columns are made of ASTM A992 steel with yield stress F_y = 345 MPa and effective length factors of K_x = 0.84, K_y = 0.80, and K_z = 1.0 were used in design. The full frame height was used to check the column strength and stability in the strong-axis direction, whereas, the weak-axis buckling strength was computed using the height of the first tier as the strut provide lateral support for the columns in the plane of the frame. Effective length smaller than unity was used to account for the distributed axial load applied on the MT-CBF column segments (Dalal 1969). A W410x67 section was selected for the columns in the 2010 design. The column axial resistance is equal to $P_{c,c}$ = 1391 kN.

The columns were redesigned according to the AISC 341-16 Seismic Provisions, which is referred to as 2016 design, under the most critical brace loading scenario (Fig. 4(d)) that leads to 1) axial compression force due to the brace resistances plus the gravity load; 2) in-plane bending moment caused by a non-uniform yielding of the braces between two adjacent tiers and is obtained based on the brace resistances; and 3) an additional out-of-plane bending moment demands as mentioned in Section 2. The critical tier is identified as the tier with the least shear resistance, which is Tier 1 for the selected frame ($V_{exp,1}$ = 913kN < $V_{exp,2}$ = 950 kN). The shear resistance is obtained from the summation of the horizontal components of the brace resistances in tension and compression V_{exp} = (T_{exp}+ C_{exp}) cos(θ), where θ is the angle between the brace and the horizontal plane. The column in-plane bending demand is calculated using unbalanced brace story shear force (Imanpour et al. 2016b), $\Delta V_{br}.h_1$ / 2(1+(h_1/h_2)), where ΔV_{br} is the unbalanced brace story shear force and is computed as follows:

$$\Delta V_{br} = \left(T_{exp} + C_{exp}\right)_2 cos\theta_2 - \left(T_{exp} + C'_{exp}\right)_1 cos\theta_1 \qquad (1)$$

For the frame shown in Fig. 4(d), ΔV_{br} = 205 kN and the corresponding in-plane bending moment in the columns is M_{ry} = 231 kN-m (60% of M_{py}), assuming that the column is pinned in the plane of the frame at the top and base levels. The out of plane bending moment demand on the column has two components, the moment induced by applying an out-of-plane horizontal notional load at the strut level that is, 0.006 times the vertical load contributed by the compression brace (amplified by multiplier B_1 = 1.16 to account for the P-δ effect), and the out-of-plane moment induced by buckling of braces that is 1.1$R_y Mp/\alpha_s$, where R_y is the ratio of expected yield stress to the specified minimum yield stress, M_p is the plastic bending moment of the brace, and α_s is the LRFD force level adjustment factor and is taken equal to 1.0. The total out-of-plane bending moment demand is equal to 32 kN-m (3.9%).

A W310x143 column was selected to carry the gravity and seismic-induced forces described here. The column resistance was verified using the interaction equation H1-1 in the 2016 AISC Specifications. It is worth noting that the presence of combined axial compression force and bi-axial bending moment demands in MT-CBF columns leads to a complex loading scenario, which may not properly be predicted by the interaction equation prescribed by the AISC Specifications for doubly and single symmetric members subjected to flexure and axial force and further studies are needed to investigate the validity of such interaction equation for the MT-CBF columns.

4.3 Strut Design

For both frame designs, a horizontal strut was placed between tiers to resist the unbalance load that is developed after brace buckling and yielding. Fig. 4(c) shows the strut unbalanced load equal to $P_{r,s}$ = 619 kN that occurs under the second analysis case for the 2010 and 2016 designs when the tension braces reach T_{exp} and C'_{exp} is developed in the compression braces. As required by 2016 Seismic Provisions, additional torsional moment induced by brace out-of-plane buckling of braces was also considered in design. The struts were designed with W-shape conforming to ASTM A992 steel with yield stress $F_{y,}$ = 345 MPa. A W250x67 strut was selected in both designs to carry the design loads. For the 2010 design, the strut was oriented such that the web is in the vertical plane; however, the strut web is placed in the horizontal plane for the 2016 design so that it can provide torsional bracing at the strut-to-column connections (Imanpour et al. 2016a; Stoakes and Fahnestock 2014) as needed by 2016 Seismic Provisions. Figs. 5(a) and (b) show the members selected for both designs.

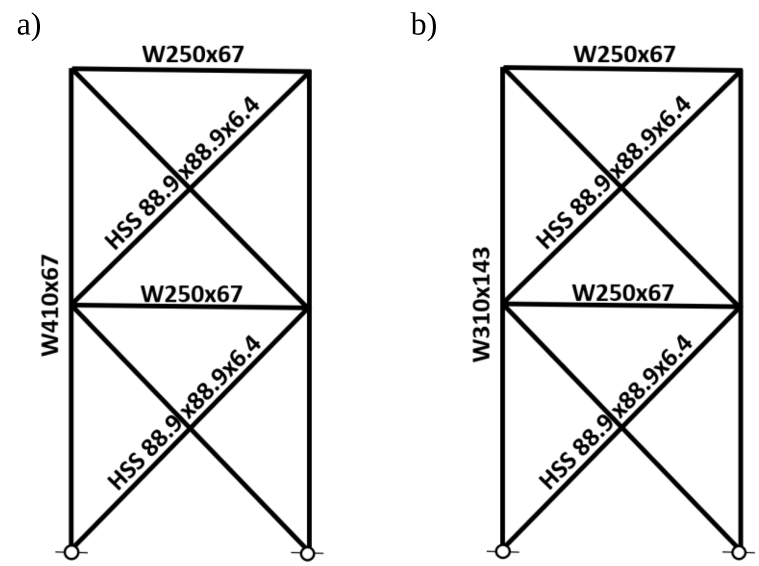


Figure 5: Selected members for MT-SCBFs (a) designed in accordance with the 2010 Seismic Provisions; (b) designed in accordance with the 2016 Seismic Provisions

5. Finite Element Analysis

5.1 Numerical Model

Two numerical finite element models of the two-tiered concentrically braced frame designed in accordance with the 2010 and 2016 requirements were constructed using the *ABAQUS* program (2014). The seismic response of the frames was evaluated using a nonlinear cyclic-pushover analysis procedure. 3D deformable shell elements were used to simulate braces, columns, struts, and connections. The numerical model of the frame is shown in Fig. 6. As illustrated, a finer mesh was applied in the connection zones to better capture the plastic deformations and stress concentrations. The material properties, including Young's modulus E = 200 MPa, Poisson's ratio υ = 0.3, and yield stress R_yF_y = 431 MPa for braces and F_y = 345 MPa for other members were assigned in the models. The plastic material from the ABAQUS material library was used to simulate the material inelastic response. A combined hardening model was used to capture the behavior of the steel material under cyclic loading. The parameters used for the combined hardening response were obtained from Suzuki and Lignos (2015) and include initial kinematic hardening modulus C_1= 3378 MPa, rate at which C_1 decreases γ = 20, the maximum change in size of the yield surface Q_∞ = 90 MPa, and the rate at which the size of the yield surface changes with plastic deformation b = 2.

Initial geometric imperfections corresponding to the members' first buckling mode, which were obtained from an Eigenvalue Buckling analysis, were assigned to columns and braces. The amplitude of the initial imperfection was taken equal to 1/1000 (AISC 2016c) times the unbraced length of the member in the direction of buckling, i.e. the total height of the frame in the out-of-plane direction and the tier height in the plane of the frame were considered for columns. For braces, the imperfections were only applied in the out-of-plane direction. Initial residual stresses were also incorporated in the column and strut sections. Residual stresses were applied based on the pattern proposed by Galambos and Ketter (1958). A leaning column was included in the model to account for P-Δ effects. The leaning column was simulated using a 3D deformable wire element and pin connected at its base and top.

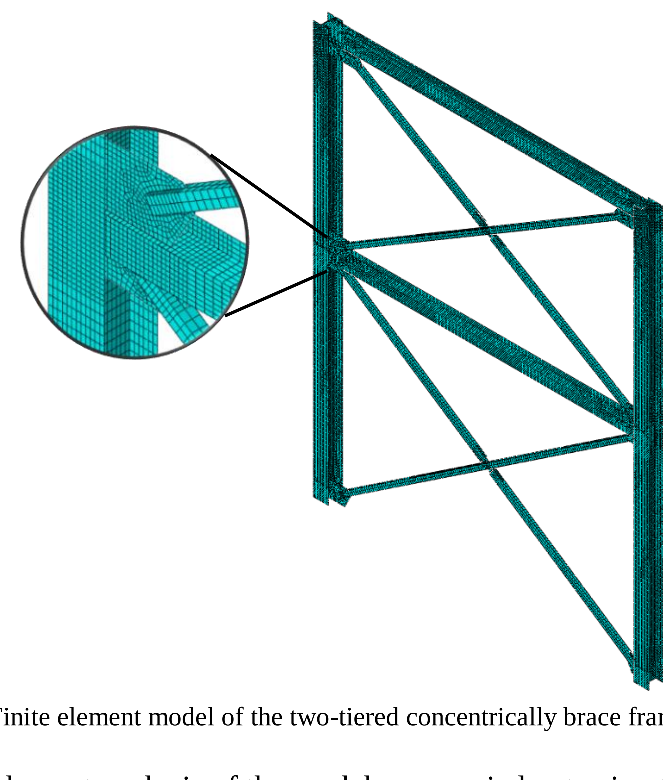


Figure 6: Finite element model of the two-tiered concentrically brace frame (leaning column omitted for clarity)

The finite element analysis of the model was carried out using two analysis steps in the ABAQUS program. The first step applied the gravity loads at the top of each column using a static/general procedure. Gravity loads were also applied at the top of an adjacent leaning column. A cyclic displacement history was then applied at the roof level using a similar static/general procedure. The horizontal displacement applied features 14 cycles based on the loading protocol proposed by Commentary Section K3 of the 2016 Seismic Provisions for experimental testing of buckling restraint braces. The selected loading protocol was modified in the last 4 cycles by applying higher displacement demands to ensure that the 2% tier drift limit, as permitted by 2016 Seismic Provisions, was captured during analyses of the frames. The loading protocol is shown in Fig. 7, where Δ_{by} and Δ_{bm} are the ratios of brace yield deformation and the design story drift, respectively.

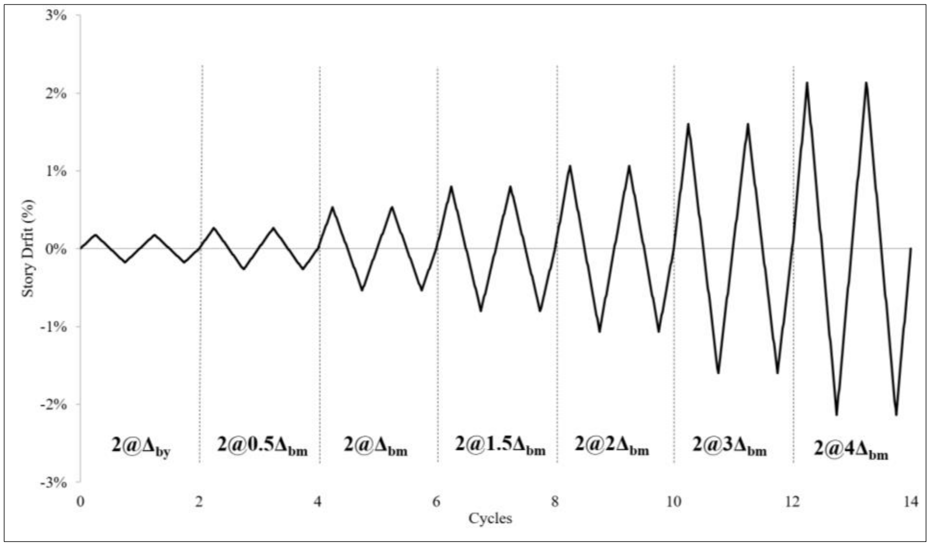
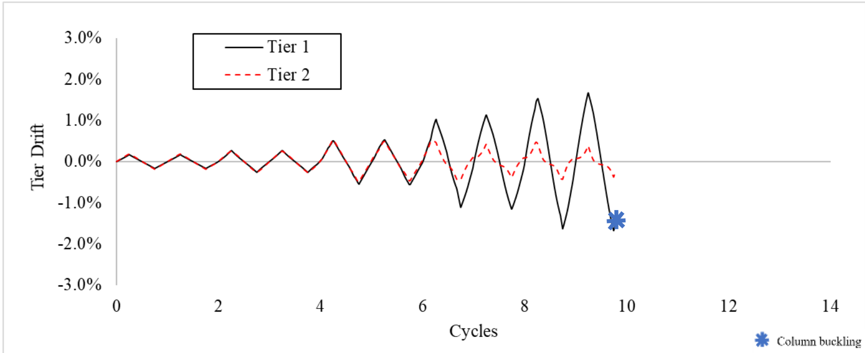


Figure 7: Loading test protocol

5.2 Analysis Results

Figs. 8(a) and (b) show the drift demands in both tiers for the 2010 and 2016 designs, respectively. The drift is plotted against the loading cycle to ease comparison between the drift that each tier experiences. For both designs, the drifts in the two tiers are nearly identical through the first 6 cycles until the story drift reaches 0.6. Brace yielding is then initiated in Tier 1 (critical tier) as expected due to lower story shear capacity of the tier, which led to significantly larger drift in this tier compared to Tier 2. This concentration of inelastic deformation remains unchanged until column instability occurs in the first-tier segment of the 2016 design. Fig. 9(a) shows the frame deformed shape at the initiation of column buckling. The analysis stopped after the initiation of column buckling due to numerical convergence issues. The response of the 2016 design was significantly different than the 2010 counterpart. A uniform deformation response was observed after and the inelastic displacement demands were more evenly distributed between tiers at story drifts that corresponds to inelastic frame deformations, as shown in Fig. 8(b). No column buckling was observed in the 2016 design up to the end of the analysis where the frame reached 2.1% story drift. The frame deformed shape at the maximum story drift is shown in Fig. 9(b).

a)



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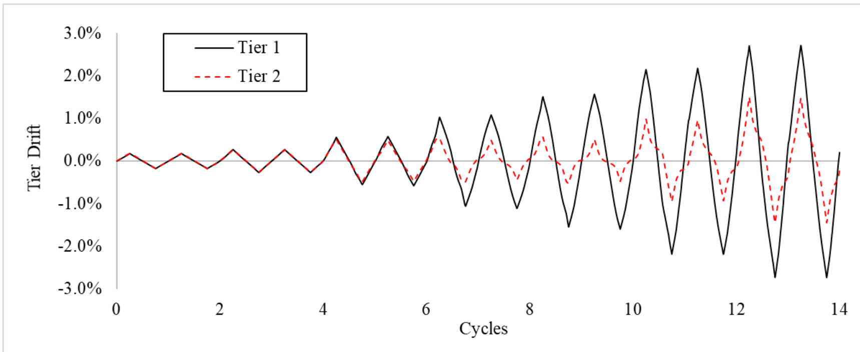
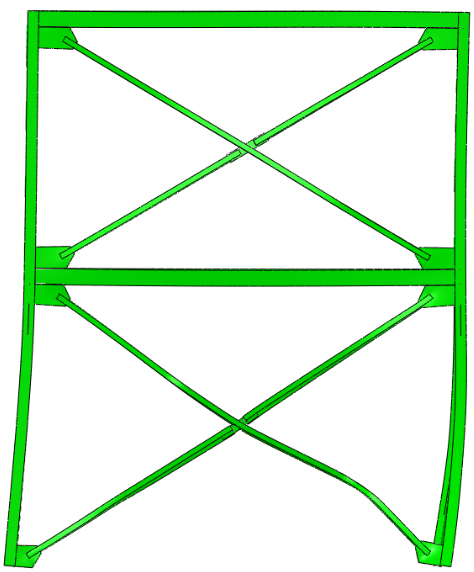


Figure 8: Tier drift of frames designed using the (a) 2010 Seismic Provisions; and (b) 2016 Seismic Provisions

a) b)



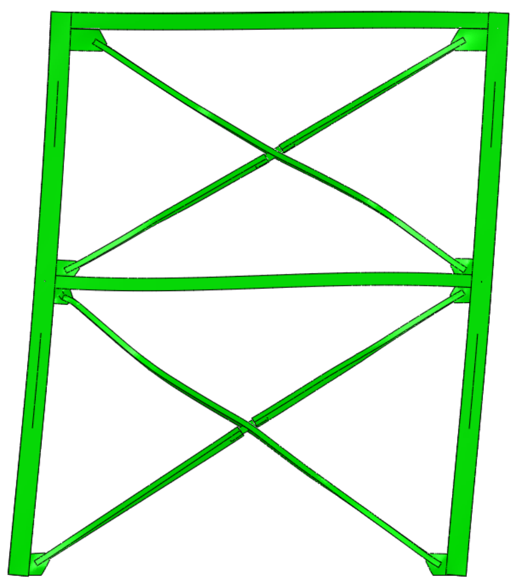


Figure 9: Frame deformed shape: (a) the 2010 design at column buckling (story drift = 1.1%); and (b) the 2016 design at maximum story drift (2.1%)

The brace axial forces are plotted against the tier drift in Figs. 10(a) to (d) for continuous and of discontinuous braces in both tiers. The brace axial forces for the 2010 design are shown in Figs.

10(a) and (b). As shown, the tension brace of Tier 2 remains nearly elastic during the application of the displacement history, although the compression force of the brace in this tier is slightly reduced. However, severe inelastic deformations are induced in the first tier due to severe buckling and yielding of the braces. In contrast, the braces in both tiers of the 2016 design (Figs 10(c) and (d)) contributed to the inelastic response of the frame and undergo yielding and buckling. The results observed confirm that the introduction of the special requirements in the latest edition of the AISC Seismic Provisions for MT-SCBFs, significantly improve the lateral response of the frame and result in a more uniform distribution of the inelastic demands, allowing an efficient seismic dissipation system for MT-SCBFs.

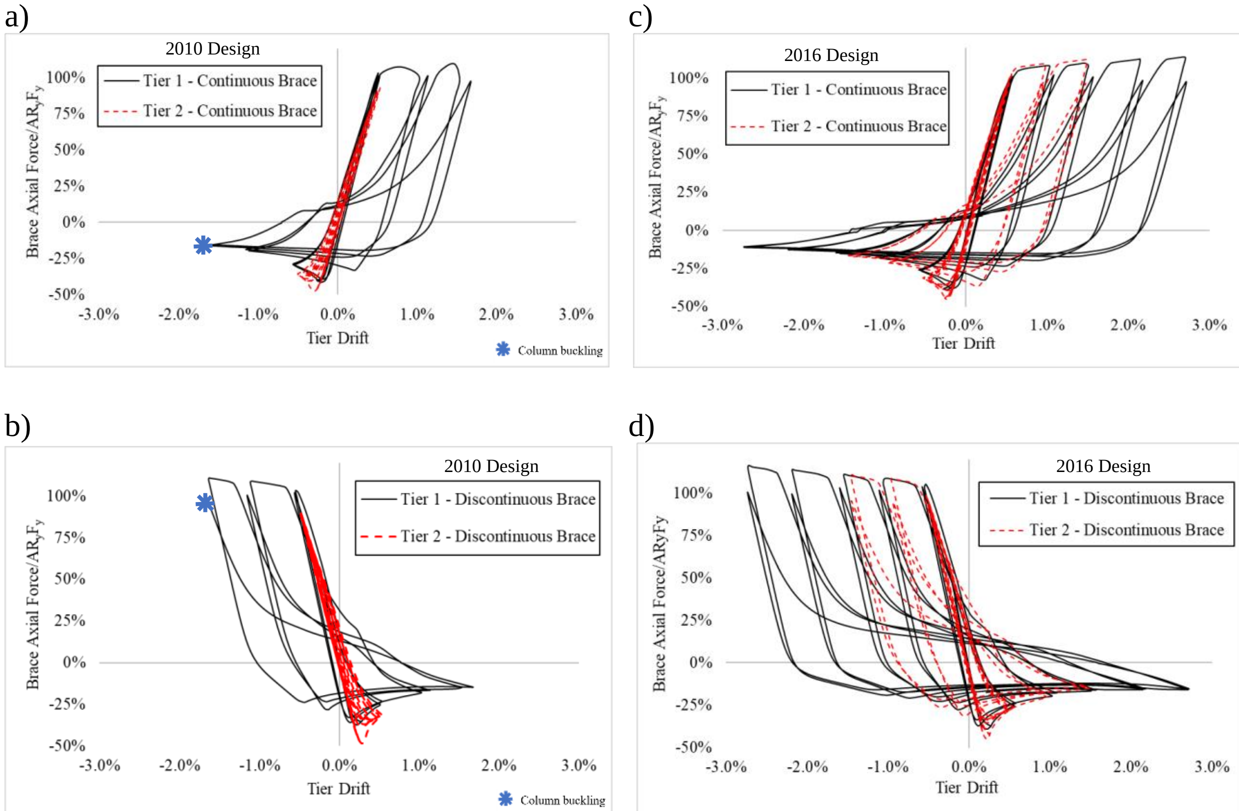


Figure 10: Normalized brace axial force: (a) continuous braces of the 2010 design; (b) discontinuous braces of the 2010 design; (c) continuous braces of the 2016 design; (d) discontinuous braces of the 2016 design

Column in-plane bending moments were plotted for both frames in Fig. 11(a) to determine how the differential tier drift affects the bending demand on the column. Note that the results of one of the columns are presented here. The moments were normalized by the plastic moment of the corresponding section about its minor axis M_{py}. The maximum normalized moment demand in the frame designed using the 2010 and 2016 Seismic Provisions are 34% and 33%, respectively. Larger in-plane moments are induced in the columns of the 2016 design compared to those of the 2010 design. The reason being that the higher stiffness of the column section in the 2016 design attracts higher moments to compensate for the unbalanced shear forces required to initiate yielding in non-critical Tier 2.

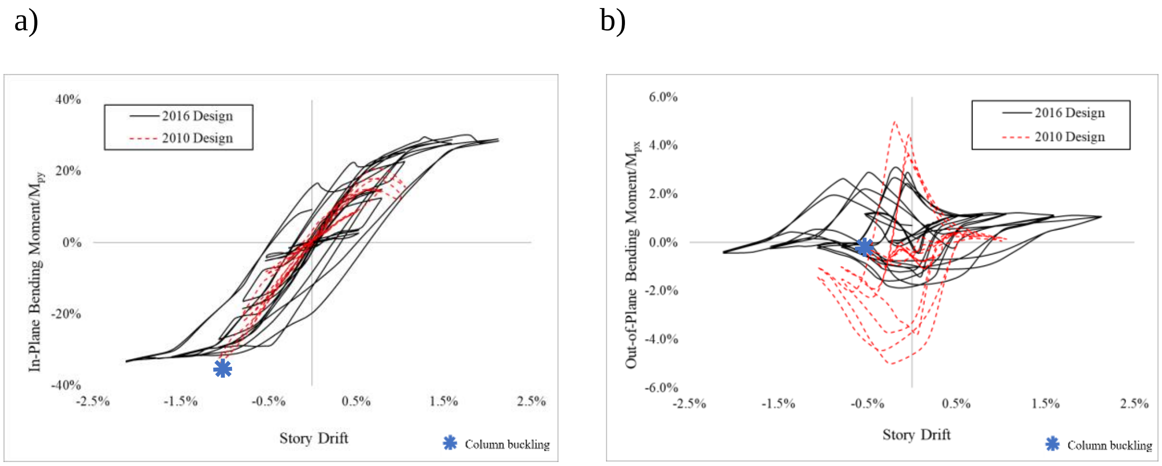


Figure 11: Column bending demands for the 2010 and 2016 designs: (a) in-plane bending moment; and (b) out-of-plane bending moment

Out-of-plane bending moment of the columns are plotted against total story drift in Figs. 11(b). The moments were normalized by the corresponding plastic section modulus about the section strong axis M_{px}. The columns of the 2010 and 2016 designs experienced an out-of-plane demand of 23 kN-m (5.5% M_{px}) and 25 kN-m (3.1% M_{px}), respectively. The maximum of out-of-plane bending moment obtained from the numerical model for the 2016 design match well the design values provided by the 2016 AISC Seismic Provisions as presented in Section 2. Furthermore, it was observed that the maximum out-of-plane moments do not occur at maximum story drifts, which may be attributed to the fact that compression brace forces reduce in higher story drifts that results in reduced out-of-plane bending moments imposed on the columns. Additional studies are needed to verify this observation in the frames with different number of tiers, heights, and configurations.

6. Conclusions

This paper describes the seismic response of and design procedure for steel multi-tiered special concentrically braced frames designed based on the 2010 and 2016 AISC Seismic Provisions. A detailed nonlinear finite element model of a two-tier special concentrically braced frame was developed and analyzed under cyclic displacement demands to evaluate the frame nonlinear response and validate the design requirements for MT-SCBF implicit in the current design standard. In particular, the column design demands were evaluated. The main findings of this study are summarized as follows:

- The numerical model of the frame can appropriately predict the brace inelastic cyclic response.
- Inelastic frame deformations are concentrated in one of the tiers in the frame designed in accordance with the 2010 Seismic Provisions. Such non-uniform lateral response led to column bi-axial buckling in the first tier for the frame case study.
- Lateral response of the 2016 frame, where the columns were sized to resist additional in-plane and out-of-plane bending moments, was significantly improved. No column buckling occurred and more uniform distribution of inelastic lateral deformations was observed at high story drifts.
- The results of preliminary numerical analyses on the frame case study showed that the current AISC approach may overestimate in-plane bending moment demand in multi-tiered

braced frame columns. This observation should be treated carefully because the numerical simulation was limited to one frame under cyclic displacement demands. Further numerical simulations on large number of frame configurations using the dynamic response history analysis procedure are required to further verify the findings of this study. The results of such dynamic analyses should be used to evaluate the column torsional and out-of-plane moment demands and propose improvements if necessary.

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