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Seismic Performance Assessment of Steel Multi-Tiered Ordinary Concentrically-Braced Frames

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Abstract
Multi-tiered braced frames (MT-BFs) are created when a tall single-story bay is divided into multiple bracing panels over the height, with no diaphragms or out-of-plane column supports between the base and roof. The unique conditions in MT-BFs make the column susceptible to instability from combined axial and bending moment demands during nonlinear seismic response. The present research is investigating the seismic behavior of multi-tiered ordinary concentrically-braced frames (MT-OCBFs), which are designed with a relatively simple procedure and are expected to provide limited inelastic deformation capacity. The baseline for the study is the 2010 AISC *Seismic Provisions*, which require column design for an amplified axial demand. The new 2016 AISC *Seismic Provisions*, which are based on a limited initial evaluation, stipulate an additional amplified axial demand to approximately account for moment in the MT-OCBF column. This approach is being more comprehensively studied, and the interaction effects of axial force, in-plane and out-of-plane moments are being thoroughly assessed. This paper presents the results from nonlinear static (pushover) and response history (dynamic) analyses for a set of prototype MT-OCBFs with X bracing. Significant drift concentration and column buckling is observed in all the baseline frames. The larger column sections resulting from the new design provisions possess adequate strength to delay column buckling in most cases and tend to improve the inelastic drift distribution. However, the columns do not necessarily have sufficient stiffness to eliminate the potential for brace fracture. For OCBFs, a simple but effective design approach, which does not require rigorous capacity-based calculations, is desired to control drift concentration in a single tier and maintain column stability.

1. Introduction
Steel braced frames are commonly used as the lateral force-resisting system in tall single-story buildings such as industrial warehouses, airplane hangars, stadiums, and performing arts and convention centers. A multi-tiered braced frame (MT-BF) is created when the tall single-story is divided into multiple bracing panels or tiers using intermediate struts, with no out-of-plane supports or floor diaphragms between the base and the story level. MT-BFs may be designed with X, V (chevron) or split-X bracing configurations, and with uniform or varying tier heights depending on the project constraints. MT-BFs are a practical and economical choice for tall single-story buildings, or tall stories within multi-story buildings. Example MT-BFs which two bracing configurations are shown in Fig 1.

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(a) (b)

Figure 1: (a) 2-tiered X-braced frame in a single-story steel building (Impanpour et al. 2016); (b) chevron MT-BF at the AT&T Park in San Francisco, California (scientika.blogspot.com).

The MT-BF configuration creates a unique condition in which large flexural demands are imposed on the columns, which along with the axial forces make the columns susceptible to instability. Prior research, focused on multi-tiered special-concentrically braced frames (MT-SCBFs), has shown that column flexural buckling is the primary mode of failure. A similar comprehensive investigation is needed for multi-tiered ordinary concentrically-braced frames (MT-OCBFs). This paper presents results from an ongoing assessment of the seismic performance of MT-OCBFs. The results from nonlinear static and dynamic analyses from a subset of MT-OCBFs proportioned per the baseline (AISC 2010a) and new (AISC 2016) versions of the AISC *Seismic Provisions* are evaluated and compared.

1.1 Behavior of MT-BFs

Similar to multi-story concentrically-braced frames, the primary seismic energy dissipation mechanism in MT-BFs is expected to be brace inelastic axial response. In an MT-BF with equal tier heights, the nominal shear strengths of all the tiers are identical, but differences are likely to develop in the inelastic range of response. Unlike multi-story frames, inertial forces cannot develop at the tier levels and unbalanced horizontal forces that develop due to differences in capacities of adjacent tiers must be redistributed within the braced frame itself. Prior research (Imanpour et al. 2013; Imanpour and Tremblay 2016; Imanpour et al. 2016a; Imanpour et al. 2016b) has shown that inelastic deformations tend to concentrate in the critical tier in which brace tension yielding is initiated. Strength degradation of the compression brace in the post-buckling range further reduces the shear strength of the critical tier that leads to increasing inelastic drift concentration in this tier. In addition to imposing excessive ductility demands on the braces, which can lead to low-cycle fatigue fracture, this phenomenon prevents tension yielding from occurring in adjacent tiers and imposes additional flexural demands on the columns. These flexural demands are not directly considered in the 2010 *Seismic Provisions*, and combined with axial demands, they can result in column instability. Column stability is further compromised due to the lack of out-of-plane supports at the tier levels.

1.2 Research Motivation

Previous assessment of the seismic performance of MT-BFs in the United States has been focused on isolated column studies (Stoakes and Fahnestock 2012; Stoakes and Fahnestock 2016) and high-ductility or special concentrically-braced frames (SCBFs). The results from extensive nonlinear static and dynamic analyses have been used to provide design guidance for MT-SCBFs in the new 2016 AISC *Seismic Provisions* (AISC 2016) that impose minimum strength and stiffness requirements to achieve satisfactory seismic performance. However, the guidance for MT-OCBFs is based on a limited evaluation and a more comprehensive assessment is needed to determine the effectiveness of the provisions and to study the need for any changes, while maintaining the simplified design procedure used for limited ductility systems.

2. Seismic Design Framework

The AISC *Seismic Provisions* contain systems expected to provide a range of inelastic behavior from high to limited ductility, depending on the proportioning and detailing. Systems that are intended to exhibit high ductility have stringent capacity-design and seismic detailing requirements as compared to those intended to exhibit limited ductility. In the braced-frame category, ordinary concentrically-braced frames (OCBFs) and special concentrically-braced frames (SCBFs) are intended to exhibit modest and high levels of inelastic response, respectively. As such, SCBFs possess extensive capacity-design and detailing requirements, while OCBFs are designed using a relatively simple procedure with few such requirements. Since this study is intended to compare and contrast the seismic performance of MT-OCBFs designed per both the 2010 AISC *Seismic Provisions* (AISC 2010a), the currently-used standard, and the 2016 AISC *Seismic Provisions* (AISC 2016), the new standard, a brief description of both is provided.

The 2010 *Seismic Provisions* (AISC 341-10) do not contain specific design requirements for MT-BFs, thus the requirements prescribed for multi-story OCBFs are used, whereas the new 2016 AISC *Seismic Provisions* (AISC 341-16) detail a design procedure for MT-OCBFs. Since a rigorous capacity design procedure is not required, AISC 341-10 employs a simple approach to account for additional demands in the columns and connections that are designed for an amplified seismic load effect given by $\Omega_0 Q_E$, where Ω_0 is the system overstrength factor and Q_E is the horizontal earthquake effect. No specific strength requirements are imposed for the struts, which are designed to resist the full frame base shear in compression. AISC 341-16 imposes an additional factor of 1.5 on the horizontal earthquake effect ($1.5\Omega_0$) considered for the columns, struts, and the connections. The column design is further enhanced by including out-of-plane moment. Braces are required to be moderately ductile per both provisions, and their required strength is based on the appropriate load combinations without the system overstrength factor.

3. Design Matrix

The frames presented in this study are used as the lateral force-resisting systems in a single-story, industrial steel building assumed to be located in coastal California. The building has a rectangular plan with dimensions 460 ft x 180 ft with long roof trusses spanning its width. Four MT-BFs are used along the perimeter of the building in each of the two orthogonal directions. Two frame heights are considered, 40 ft and 60 ft, and the bay width is constant at 20 ft, yielding aspect ratios of 2:1 and 3:1. The frames are designed as OCBFs with a response modification coefficient R = 3.25, deflection amplification factor C_d = 3.25, and system overstrength factor Ω_0 = 2.0. Seismic design is per ASCE 7-10 (ASCE 2010), and members and connections are proportioned per the provisions of AISC 360-10, the AISC *Specification* (AISC 2010b), and AISC 341-10 or AISC 341-16. Each frame is assigned a unique label that describes its geometry and design. For example, the components of the label OCBF10-3-3X-U describe: (1) the seismic provisions used to design the frame, where 10 and 16 refer to AISC 341-10 and AISC 341-16, respectively; (2) frame aspect ratio, where 2 and 3 refer to the 40 ft and 60 ft frames, respectively; (3) the number of tiers that varies between 2 and 4 for the frames presented in this study, followed by an X that indicates the bracing configuration; and (4) uniformity of tier height ratios, where U and NU refer to uniform tier heights and non-uniform tier heights (tall first tier), respectively.

The gravity loads on the structure include a roof dead load of 25 psf (live loads are not included in these designs) and exterior wall weight of 25 psf. For seismic design, the building is assigned to Site Class D with Seismic Design Category D (SDC D), Risk Category II, and an importance factor, I_e, of 1 for earthquakes. All frames are designed for the design basis earthquake (DBE) hazard which is 2/3 of the maximum considered earthquake (MCE). Thus, the design spectral accelerations in the short period range and at 1 second are S_{DS} = 1.0g and S_{D1} = 0.6g, respectively. The equivalent lateral force procedure within the ASCE 7-10 framework is used to determine the seismic design base shear, and a 10% amplification is included for accidental eccentricity between the centers of mass and stiffness. The design fundamental period (C_uT_a), seismic response coefficient (C_s), seismic weight of the building, and design base shear are summarized in Table 1 for the two frame heights.

Table 1: Seismic design parameters.

Parameter	Aspect ratio = 2	Aspect ratio = 3
C_uT_a	0.44 seconds	0.60 seconds
C_s	0.308	0.308
Seismic Weight	2710 kips	3030 kips
Design Base Shear	230 kips	257 kips

The braces are designed to resist combined gravity and seismic loads in compression using an effective length factor of 0.45 (Imanpour et al. 2013) on the workpoint length. HSS sections with ASTM A1085 material specification are used for the braces. The columns of the braced frames are designed to resist the axial load effect from gravity and seismic loading, including the vertical earthquake effect. The horizontal seismic effect is appropriately amplified depending on the version of the seismic design provisions used. The columns are assumed to be continuous over the full frame height and effective length factors of 0.80 and 0.79 (Imanpour et al. 2013; Dalal 1969) are used for in-plane and out-of-plane flexural buckling, respectively. W sections, oriented for weak-axis bending in-plane, and ASTM A992 material specification are used for both the columns and the struts. Both the columns and struts are assumed to have pinned end connections, and the struts are assumed to provide in-plane and torsional restraints to the columns. The frame geometries used in this study are shown in Fig 2. The section sizes determined from the described design process are summarized in Table 2. Additionally, the design story drift is computed for each frame and checked against an allowable drift of 2% imposed by ASCE 7-10. The design story drift estimate is given by $C_d\Delta_e/I_eh$.

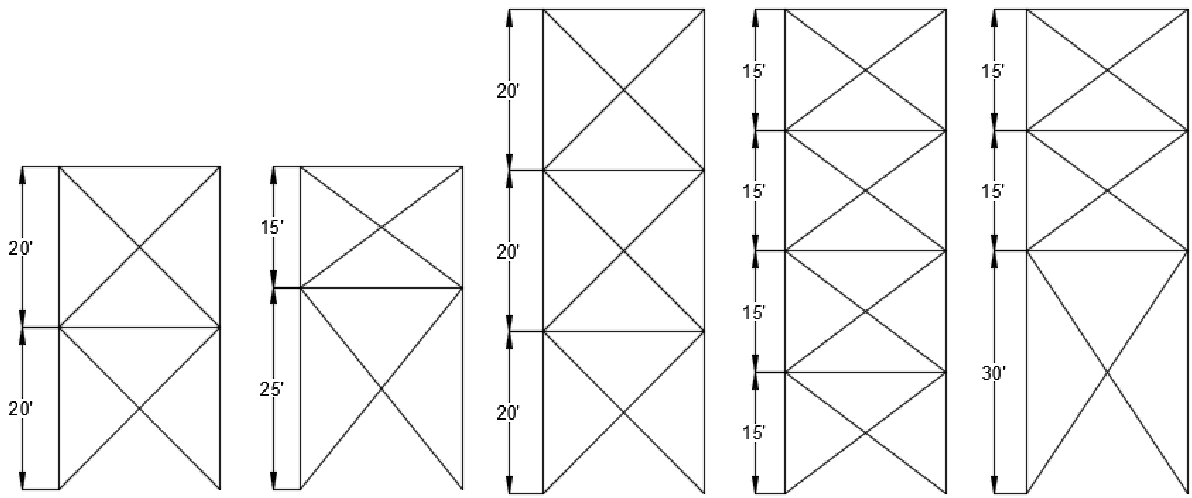


Figure 2: Frame geometry and brace configuration.

Table 2: Member sizes.

Frame	Brace	Column	Strut	Design Story Drift (%)
OCBF10-2X-U	HSS 5x5x3/8	W18x86	W8x48	0.81
OCBF16-2X-U	HSS 5x5x3/8	W21x122	W12x87	0.73
OCBF10-2X-NU	HSS 5.5x5.5x3/8 HSS 4.5x4.5x3/8	W18x97	W8x48	0.80
OCBF16-2X-NU	HSS 5.5x5.5x3/8 HSS 4.5x4.5x3/8	W24x131	W12x87	0.74
OCBF10-3-3X-U	HSS 5.5x5.5x5/16	W24x146	W10x49	0.74
OCBF16-3-3X-U	HSS 5.5x5.5x5/16	W30x211	W12x96	0.69
OCBF10-3-4X-U	HSS 5x5x5/16	W24x146	W10x49	0.75
OCBF16-3-4X-U	HSS 5x5x5/16	W27x194	W12x96	0.72
OCBF10-3-3X-NU	HSS 6x6x1/2 HSS 5x5x5/516	W27x178	W10x49	0.70
OCBF16-3-3X-NU	HSS 6x6x1/2 HSS 5x5x5/516	W30x235	W12x96	0.67

4. Numerical Model

A three-dimensional numerical model was created using the OpenSees (McKenna and Fenves 2006) simulation platform. A schematic of a four-tiered frame, along with the reference coordinate axes is shown in Fig 3(a). Non-linear beam-column elements are used to model the braces and columns, along with fiber-discretized cross-sections to capture the buckling response of these members. An example of the brace cross-section is shown in Fig 2(c). The uniaxial Giuffré-Menegotto-Pinto (Steel02) model is used to define the material stress-strain response. The nominal yield strength F_y = 50 ksi is specified for the columns, and the expected yield strength R_yF_y = 62.5 ksi is specified for the braces. In frames with identical tier heights, the expected yield strength of the braces in the first tier is reduced to $0.95R_yF_y$ to account for inherent material variability, and to initiate brace inelastic response in the tier in which the column segment carries the maximum axial load. In addition, a residual stress pattern, with $F_{RC} = 0.3F_y$ in compression, is also included for the column sections (Galambos and Ketter 1958) as shown in Fig 3(d). Elastic beam-column elements are used to model the roof beam and struts. The P-Δ effects due to gravity loads are modeled using an elastic beam-column element for the leaning column. Bi-directional initial out-of-straightness, with a half-sine profile and maximum amplitude of 1/1000 of the unsupported length of the member, is specified for the braced frame columns, while only out-of-plane imperfections are included for the braces. Imperfections for the braced frame columns are illustrated in Fig 3(b). Pinned supports are simulated for the column bases, and torsional restraint is provided at the two ends of the braced frame columns. The roof beam and struts are pin-connected to the columns for in-plane flexure. The effect of the gusset plate connections on the buckling response of the braces is also simulated. Stiff elastic beam-column elements with ten times the stiffness of the adjacent element are used at the member ends to model inelastic response occurring outside the connection region, and zero-length nonlinear rotational springs that capture the flexural response of the gusset plate are used at the brace ends (Hsiao et al 2012). Similar rotational springs and stiff elements are used at the intersection zone of the braces. Point masses are assigned at the top of the braced frame columns, and tributary gravity loads are applied to the braced frame and leaning column. Mass-proportional damping corresponding to 2% of critical damping in the primary mode of vibration is employed for the nonlinear dynamic analyses.

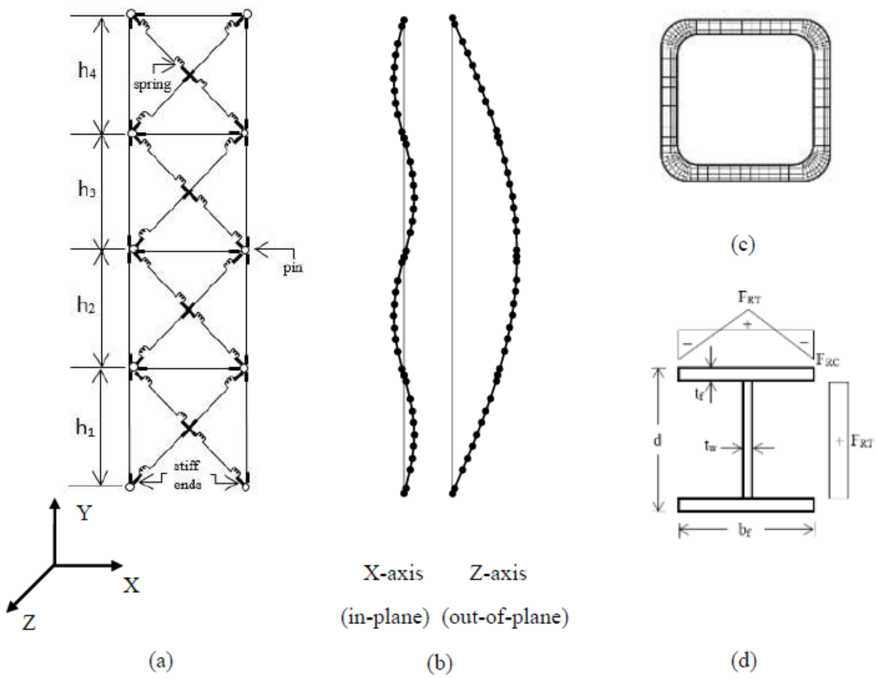


Figure 3. Numerical model (a) frame (leaning column not shown); (b) initial imperfections in the columns; (c) fiber-discretization of HSS brace section; (d) residual stress pattern in W column section.

5. Nonlinear Static Analysis

As mentioned previously, in MT-BFs the seismic mass is concentrated at the story level (roof) and inertial forces cannot develop at the tier levels. As such, the dynamic response of the frame under the first large inelastic displacement cycle of an earthquake can be reasonably estimated using a nonlinear static analysis. In particular, this analysis provides insight into the evolution of axial and flexural demands in the columns due to inelastic drift concentration, and the sequence of limit states that may occur in an MT-BF during an earthquake event.

The relative contribution of the axial, in-plane moment, and out-of-plane moment demands to the stability of the MT-BF columns is assessed using the following demand-to-capacity ratios:

$$p = \frac{P_u}{P_y} \tag{5.1}$$

$$m_z = \frac{M_{uz}}{M_{pz}} \tag{5.2}$$

$$m_x = \frac{M_{ux}}{M_{px}} \tag{5.3}$$

where P_u, M_{uz} and M_{ux} are the axial, in-plane moment, and out-of-plane moment demands at a given location in the MT-BF column. The demands are normalized by the respective cross-sectional plastic capacities. Additionally, inelastic drift concentration is evaluated by computing the drift concentration factor, *DCF*, as follows:

$$DCF = \frac{\theta_{tier,\max}}{\theta_{story}} \tag{5.4}$$

where θ_{story} is the story drift value at a particular point in time and $\theta_{tier,\max}$ is the maximum of the corresponding drifts in any of the tiers at the point in time.

The responses of OCBF10-3-3X-U and OCBF16-3-3X-U to static monotonic lateral loads are shown in Figs 4 and 5, respectively. The annotated pushover curve for OCBF10-3-3X-U is shown in Fig 4(a). The frame exhibits an initial linear elastic response until point [1] when brace buckling is initiated in Tier 1, closely followed by initiation of tension yielding in Tier 1 (critical tier). At this point, the frame reaches its maximum lateral resistance. Brace buckling occurs in the adjacent tiers (at the same time in this case) as seen by the small drop in the base shear or "kink" in the pushover plot. Following this, the lateral resistance of the frames decreases due to loss in strength of the compression brace in the post buckling range. As shown in Fig 4(b), Tier 1 drift diverges from the other tiers and increases rapidly as the tension brace is stretched in the inelastic region. The difference in tier drifts grows further until combined axial and flexural demands imposed on the column initiate buckling as indicated by point [2] on the pushover plot. The drift concentration factor at a story drift of 2% is 2.48 when the corresponding Tier 1 drift is 4.96%. With column buckling, the lateral resistance of the frame decreases significantly as the story drift continues to increase. Drifts in the non-critical tiers are similar and the top two tiers of the frame move as a rigid body on top of Tier 1 as the column buckles. At the estimated design story drift, drifts in all tiers remain below the brace fracture estimate of 2%, which is used as an indicator of the potential for premature brace fracture. The described brace behavior is shown in Fig 3(c). All braces buckle in compression, but tension yielding is limited to the brace in Tier 1. The Tier 1 compression brace also loses strength in the post-buckling region, while other compression braces unload after column buckling. The deformed shape of the frame after column buckling is shown in Fig 3(d).

The evolution of demands in the compression column with increasing story drift is examined more closely in Figs 3(e) and 3(f), which show the normalized demands at the mid-height and top of Tier 1, respectively. The peak axial demand in the column occurs when the braces have buckled and tension yielding has been initiated. At the mid-height of Tier 1, the normalized in-plane bending moment, m_z, is close to its peak value at the time of column buckling. Following this, both m_z and m_x increase gradually. Similar response is seen at the tier level, however, m_z reverses sign due to curvature reversal and increases in the opposite direction.

A similar pushover response curve for OCBF16-3-3X-U is shown in Fig 4(a). This frame is designed per the new seismic design provisions and has larger columns. It exhibits an initial linear elastic response up to point [1] when the compression brace in Tier 1 buckles. The Tier 2 compression brace buckles at point [2] while the Tier 3 brace remains elastic. The base shear decreases slightly as each brace buckles due to loss in strength of the respective brace. In contrast to OCBF10-3-3X-U, tension yielding is also triggered and sustained in the adjacent tier, which can be attributed to the larger column size. The improved distribution of inelastic tier drifts is also evident from Fig 4(b). All tier drifts remain below the brace fracture estimate of 2% at the design story drift estimate. The *DCF* at a story drift of 2% is 1.93 which is lower than the *DCF* in OCBF10-3-3X-U at the same story drift. Thus, there is some improvement in the distribution of inelastic tier drifts over the frame height, and column buckling does not occur. The axial response of the braces, Fig 4(c), clearly illustrates that tension yielding occurs in the adjacent tier as well. Column buckling does not occur in this frame as seen in the deformed shape, Fig 4(d), of the frame at 2% story drift. The imposed demands in the columns are also relatively lower compared to the column in OCBF10-3-3X-U and the column remains stable.

For the given frame configuration, the enhanced column design in the new provisions prevents buckling and subsequent loss in the lateral resistance of the frame. However, the same is not true for other frame configurations. Fig 6 shows the pushover plots for each of the two frame designs for the remaining four configurations. Column buckling still occurs, albeit at larger story drifts, in three of the four additional configurations. In particular, the worst case of column buckling is seen in the two-tiered configurations in which the maximum in-plane and out-of-plane moments tend to occur at the same location along the frame height leading to earlier onset of column buckling. Despite little difference in the section size, the column (W18x97) in OCBF10-2-2X-NU buckles before the column (W18x86) in OCBF10-2X-U, likely due to the larger in-plane unbraced length in the frame with the taller first tier. Further, the larger columns allows tension yielding to occur in adjacent tiers and make the distribution of inelastic deformations over the height of the frame more favorable, but do not necessarily provide sufficient stiffness to reduce the potential for other limit states such as brace fracture.

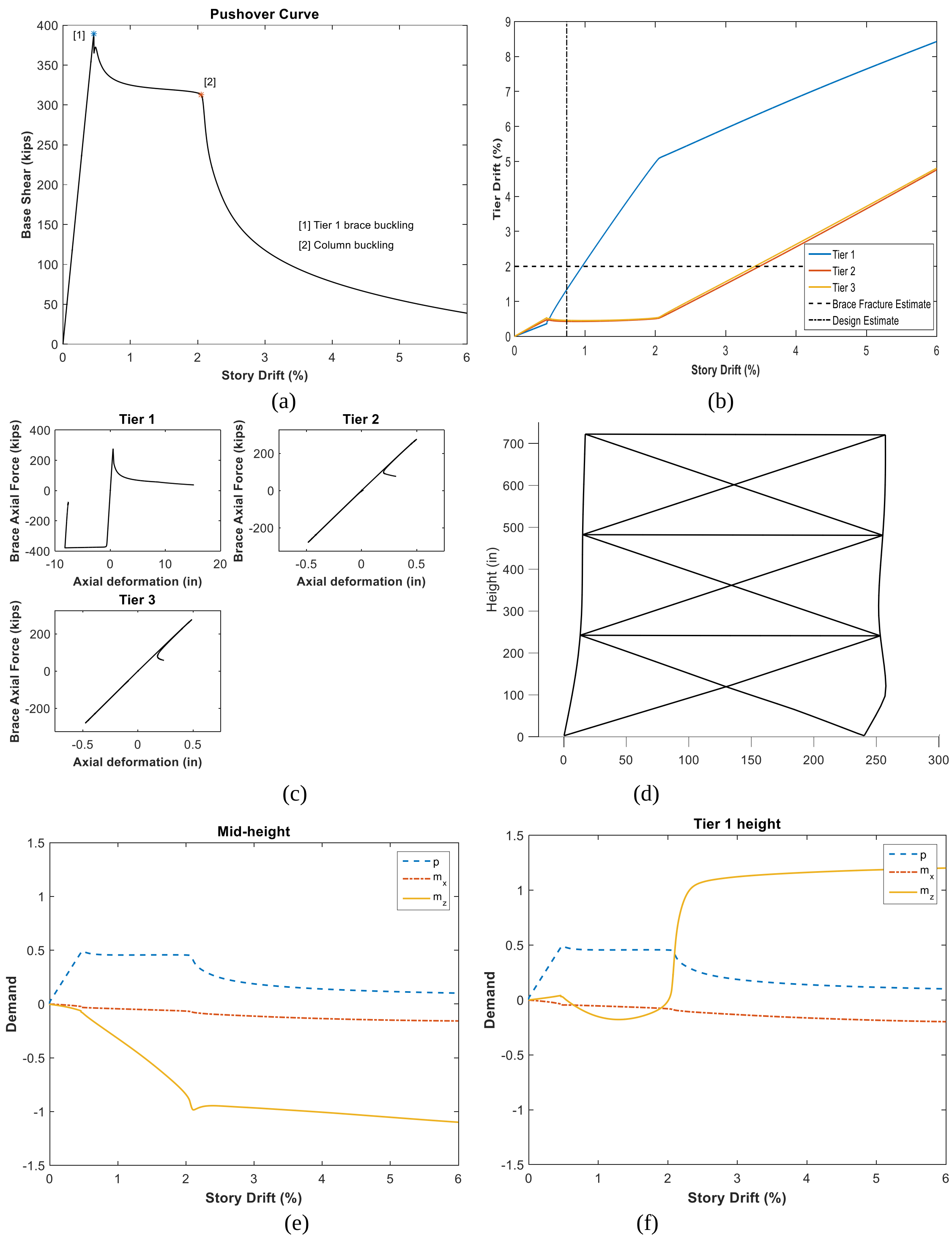


Figure 4: Static response of OCBF10-3-3X-U (a) base shear vs. roof drift (pushover curve); (b) tier drift vs. story drift; (c) brace axial response; (d) deformed shape; (e) normalized column demands at the mid-height of Tier 1; (f) normalized column demands at Tier 1.

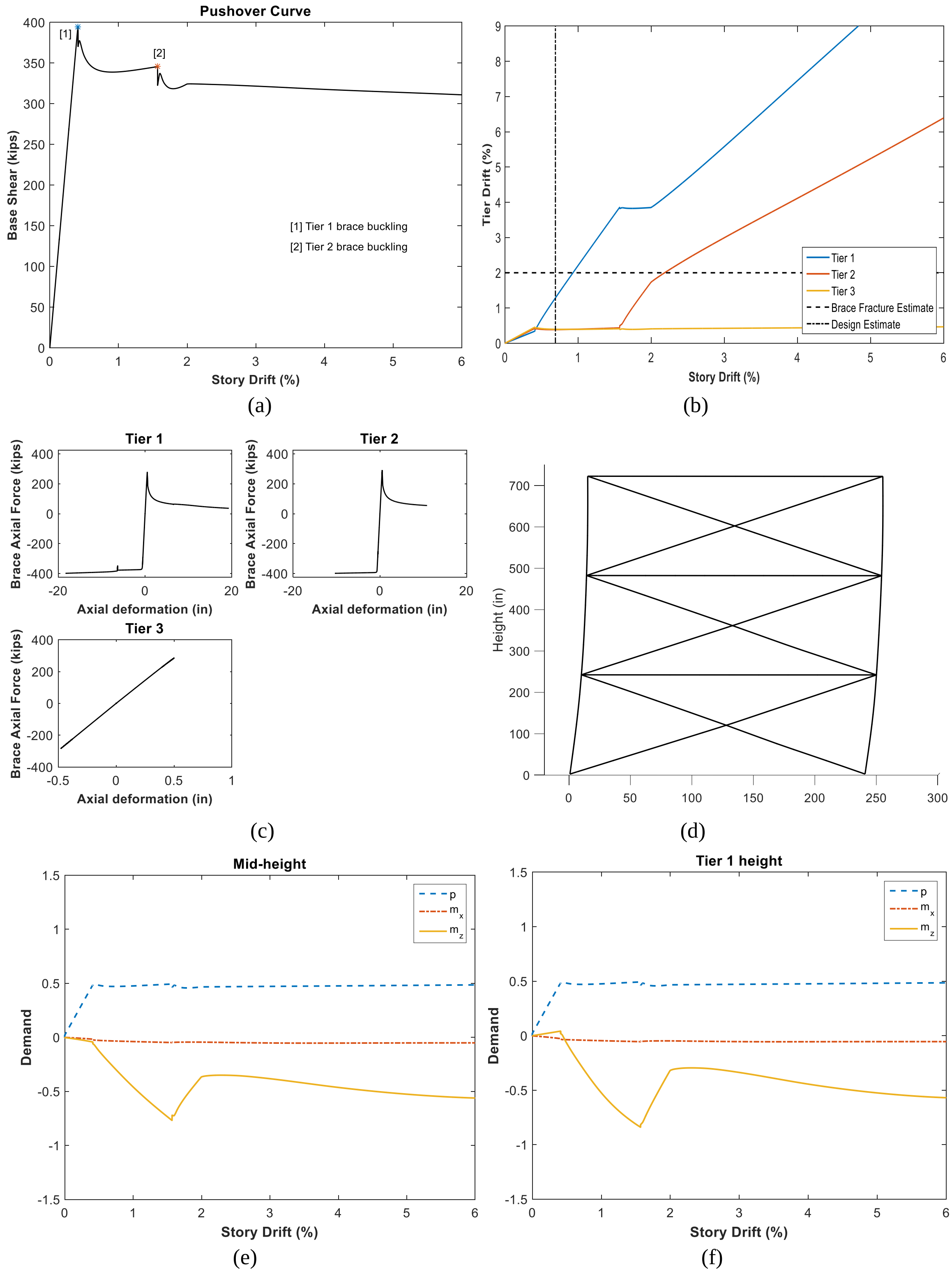
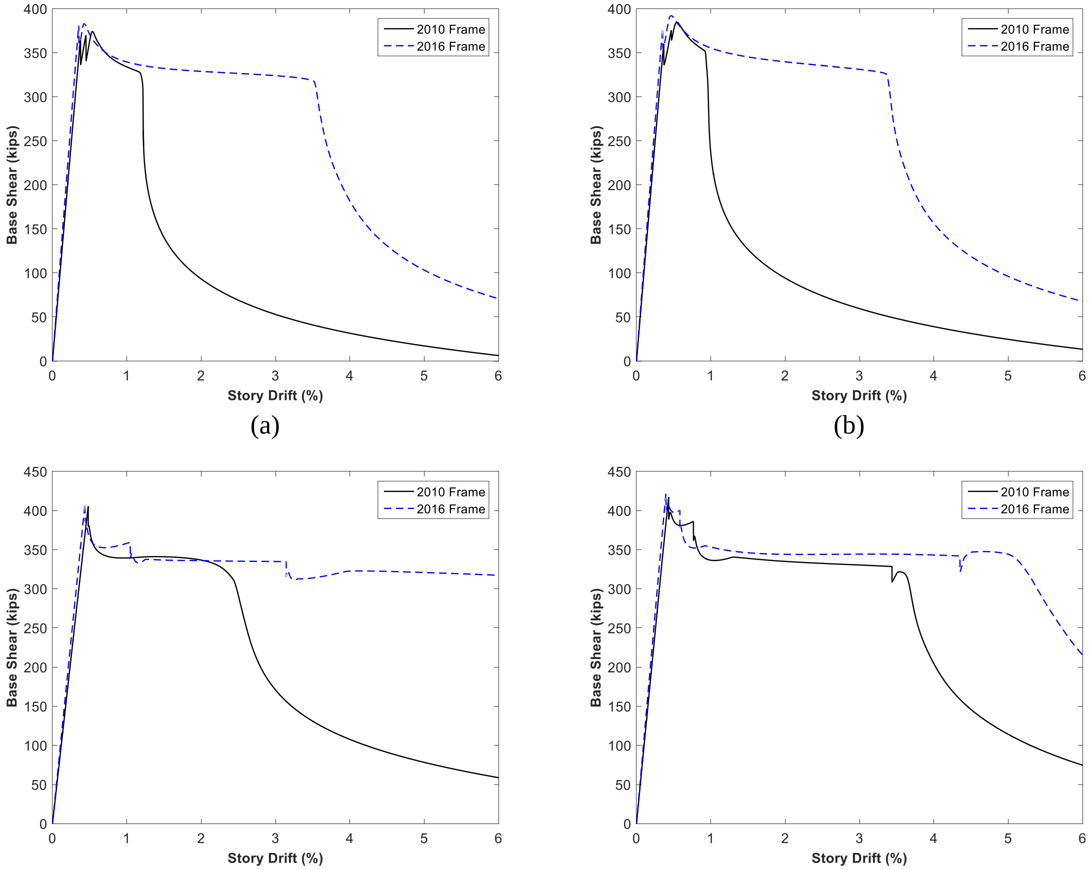


Figure 5: Static response of OCBF16-3-3X-U (a) base shear vs. roof drift (pushover curve); (b) tier drift vs. story drift; (c) brace axial response; (d) deformed shape; (e) normalized column demands at the mid-height of Tier 1; (f) normalized column demands at Tier 1.



(c) (d)

Figure 6: Pushover curves (a) OCBF10-2-2X-U and OCBF16-2-2X-U; (b) OCBF10-2-2X-NU and OCBF16-2-2X-NU; (c) OCBF10-3-4X-U and OCBF16-3-4X-U; (d) OCBF10-3-3X-NU and OCBF16-3-3X-NU.

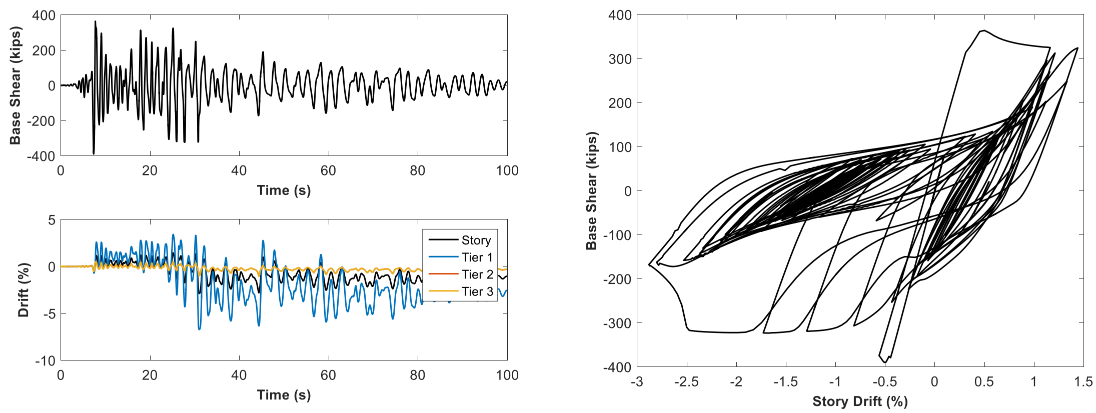
Thus, the AISC 341-10 design provisions underestimate the demands in the columns and as such do not provide an adequate minimum strength requirement. Column buckling is still observed in frames designed with the new provisions. In addition, the enhanced column sections do not necessarily provide adequate stiffness to reduce drift concentration and eliminate the potential brace fracture (due to excessive ductility demand). Thus, the flexural demands imposed on the columns during the inelastic range of behavior, and their interaction with axial loads, are key design consideration in MT-BF columns.

6. Nonlinear Dynamic Analysis

A series of nonlinear response history (dynamic) analysis were also performed to assess the seismic performance of the frames. The ground motions records were selected per the recommendations from FEMA P695 (FEMA 2009) for the collapse assessment of buildings. A total of 22 ground motions, each with two components, from the “Far-Field” record set are used in this study. The ground motion records are scaled to the MCE hazard level to assess the response characteristics of the frames at a single ground motion intensity. Each ground motion record is normalized and scaled (per FEMA P695) so that the median of the scaled ground motion suite matches the target MCE-level spectrum at the computed fundamental period of the structure.

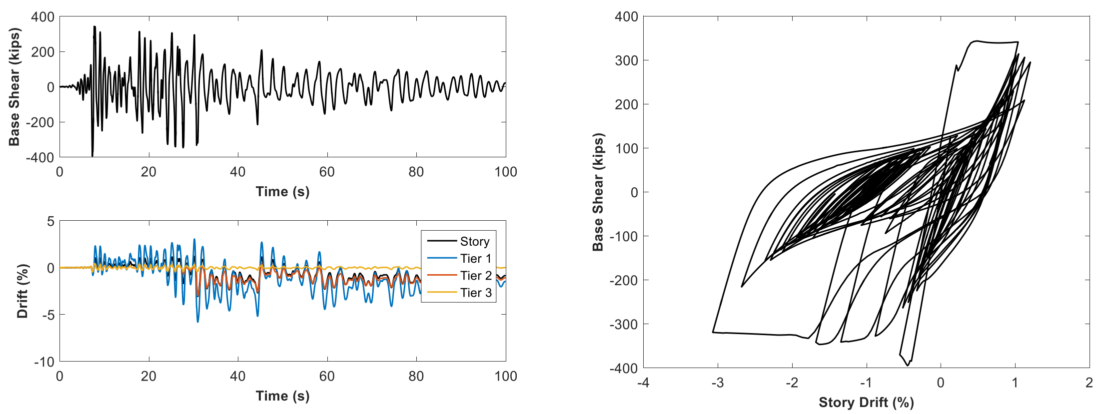
Results from the dynamic analysis of OCBF10-3-3X-U and OCBF16-3-3X-U for a single record scaled to the MCE event are discussed here. Fig 7(a) shows base shear and drift histories of OCBF10-3-3X-U to the second component of GM#5 (1979 Imperial Valley, Delta). The peak story drift under this ground motion is 2.88%, which is about 3.9 times the design story drift estimate. Significant drift concentration, with a *DCF* of 2.33, is seen in Tier 1, which has a peak drift of 6.72%, while drifts in other tiers remain well below 2%. The inelastic drift concentration prevents tension yielding in the adjacent tier and leads to buckling of the left-hand side column, followed by a rapid decrease in the base shear with increasing story drift (similar to the pushover curve at column buckling) as shown in Fig 7(b). Following column buckling, the overall stability of the frame is compromised and the potential for global collapse increases.

Fig 8(a) shows base shear and drift histories of OCBF16-3-3X-U. The peak story drift is 3.07%, which is slightly higher than the peak story drift for OCBF10-3-3X-U. However, the peak Tier 1 and Tier 2 drifts in this frame are 5.81% and 3.00% with a *DCF* of 1.89. This illustrates the improvement in inelastic drift distribution achieved by using a larger column section. Tension yielding also occurs in the Tier 2 brace and column buckling is prevented. However, the tier drifts impose excessive ductility demand that are likely to cause low-cycle fatigue fracture in the braces and loss of the primary energy dissipation mechanism (brace inelastic response) in the frame.



(a) (b)

Figure 7: Dynamic response of OCBF10-3-3X-U to GM#5, component 2 (a) base shear vs. time and drift vs. time; (b) base shear vs. story drift.



(a) (b)

Figure 8: Dynamic response of OCBF16-3-3X-U to GM#5, component 2 (a) base shear vs. time and drift vs. time; (b) base shear vs. story drift.

Nonlinear time history analysis was completed for all frames using both components of the suite of 22 ground motions (44 total analyses per frame). Although not all analyses completed successfully due to numerical convergence issues, each frame had a minimum of 35 full analyses. The median and 84th percentile of the peak seismic response quantities from the converged analyses are summarized in Tables 3 and 4, respectively. The total number of column failure cases that may lead to frame collapse are also noted. In general, the peak story drifts are slightly higher in the current frame designs. While the *DCF* is not consistently higher in the baseline frame designs, all column failure cases at the MCE-level occur in the baseline frames while none occur in the new frames with larger columns. In particular, the maximum number of column failure cases are observed in the current two-tiered configurations in the 40 ft frame. The two-tiered frame with a taller first tier is more prone to column instability (16 cases of buckling at the MCE-level event) due to the larger in-plane unbraced length of the columns. No column buckling is observed in the current four-tiered uniform and three-tiered nonuniform 60 ft frames.

Table 3: Median of peak seismic response quantities.

Frame	Story Drift (%)	Tier 1 Drift (%)	Tier 2 Drift (%)	Tier 3 Drift (%)	Tier 4 Drift (%)	*DCF*	Column failure
OCBF10-2X-U	1.09	1.48	0.59	-	-	1.31	10
OCBF16-2X-U	1.00	1.63	0.48	-	-	1.63	0
OCBF10-2X-NU	1.04	1.24	0.63	-	-	1.17	16
OCBF16-2X-NU	1.01	1.33	0.52	-	-	1.40	0
OCBF10-3-3X-U	1.08	2.35	0.49	0.53	-	2.14	1
OCBF16-3-3X-U	0.99	2.14	0.43	0.46	-	2.10	0
OCBF10-3-4X-U	1.01	2.53	0.54	0.53	0.55	2.50	0
OCBF16-3-4X-U	0.96	2.41	0.50	0.48	0.49	2.35	0
OCBF10-3-3X-NU	1.00	0.35	1.45	2.40	-	2.18	0
OCBF16-3-3X-NU	0.94	0.32	1.26	2.09	-	2.15	0

Table 4: 84th percentile of peak seismic response quantities.

Frame	Story Drift (%)	Tier 1 Drift (%)	Tier 2 Drift (%)	Tier 3 Drift (%)	Tier 4 Drift (%)	*DCF*	Column failure
OCBF10-2X-U	2.22	2.74	2.35	-	-	1.67	10
OCBF16-2X-U	1.66	2.95	0.52	-	-	1.79	0
OCBF10-2X-NU	2.41	2.45	2.64	-	-	1.40	16
OCBF16-2X-NU	1.64	2.36	0.56	-	-	1.48	0
OCBF10-3-3X-U	1.74	4.26	0.59	0.55	-	2.36	1
OCBF16-3-3X-U	1.61	3.74	1.46	0.46	-	2.30	0
OCBF10-3-4X-U	1.56	4.19	0.61	0.54	0.56	2.79	0
OCBF16-3-4X-U	1.75	4.01	2.12	0.48	0.50	2.60	0
OCBF10-3-3X-NU	1.57	0.35	2.34	3.43	-	2.66	0
OCBF16-3-3X-NU	1.66	0.33	2.49	3.55	-	2.25	0

7. Conclusions and Future Work

The seismic performance of steel multi-tiered ordinary concentrically braced frames (MT-OCBFs), with X bracing, is evaluated in this study. A set of five frames with two aspect ratios (2 and 3), and varying number of tiers and tier height ratios were designed in accordance with the 2010 AISC *Seismic Provisions* (AISC 341-10), which is currently used in many jurisdictions, and the new 2016 AISC *Seismic Provisions* (AISC 341-16). AISC 341-10 does not contain guidelines for the design of MT-BFs and employ a simple axial force amplification to determine the minimum required column strength. AISC 341-16, on the other hand, contains specific guidelines for MT-BFs. In particular, the results from extensive research have been used to define the minimum strength and stiffness requirements for multi-tiered special concentrically-braced frames (MT-SCBFs) to achieve satisfactory seismic performance. In contrast, the research on MT-OCBFs is limited. An additional amplification of the axial force demands, which leads to larger column sections, is used to approximately account for the flexural demands imposed on the MT-OCBF columns during inelastic seismic response. This study assesses the effectiveness of the enhanced column in improving the seismic performance of MT-OCBFs.

The results of the nonlinear static (pushover) analyses show that column buckling occurs in all frames designed in accordance with AISC 341-10. All frames exhibit an initial linear elastic response until brace buckling and tension yielding occur in the braces. Differences in the shear strength of adjacent tiers develop due to the brace inelastic response (tension yielding typically contained in Tier 1) and lead to the development of unbalanced horizontal forces at the tier levels that must be resisted through column bending. The additional flexural demands imposed on the columns are not directly considered in the design, which results in inadequate column sizes. Although brace fracture is not modeled in this study, the significant inelastic drift concentration at the tiers increases the potential for brace fracture. The worst case for column instability is observed in the two-tiered frames, where buckling occurs relatively early in the story drift history. In general, column buckling is delayed to larger story drifts in the new frame designs. The larger column sections allow tension yielding to propagate to adjacent tiers and tend to improve the distribution of inelastic tier drifts over the frame height. The potential for brace fracture is also reduced although not eliminated. The overall performance of the new frames is better that that of the corresponding baseline frames.

A series of nonlinear response history (dynamic) analysis was also conducted. Both components of a suite of 22 ground motions, scaled to the maximum considered earthquake (MCE) intensity, were used to assess the seismic response of the frames. The overall dynamic analysis results are in good agreement with the pushover analysis results. The median peak story drifts were larger in the current frames, with a few exceptions in the 84^{th} percentile of the peak story drifts. Tension yielding was triggered in adjacent tiers and no column buckling occurred in the new frames. Similar to the pushover analysis results, the statistics of the peak tier drifts still show potential for brace fracture during an inelastic seismic event.

This study demonstrates the key seismic response characteristics of MT-OCBFs for both sets of designs. The larger column sections possess adequate strength to prevent buckling in most cases, but inelastic drift concentration still remains an issue. Future work on MT-OCBFs should evaluate minimum stiffness requirements to address this issue. As mentioned previously, brace fracture is also an important limit state that must be considered due to the influence of brace loss on column demands and the stiffness of the frames. An ongoing study is also aimed at assessing the influence of brace configuration on the seismic performance of MT-OCBFs. Analysis techniques that both reasonably assess the imposed flexural demands on the columns and maintain the relatively simple design procedure for OCBFs are required.

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