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Parametric Evaluation of Multi-Tiered Ordinary Concentrically Braced Frames under Seismic Loading

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Abstract

Steel braced frames are commonly used to provide resistance to lateral loads in structures across the United States. In tall, single-story buildings, it is more beneficial to use a multi-tiered braced frame (MT-BF) configuration that is formed by dividing the tall story into multiple bracing panels using horizontal struts. However, there are no out-of-plane supports at the intermediate tier levels and the majority of the seismic mass is located at the top of the frame (typically the roof level) that has important implications in seismic design and performance. The unique conditions in MT-BFs have been shown to cause significant inelastic drift concentration and eventual column instability. This study employs nonlinear analysis of multi-tiered ordinary concentrically braced frames (MT-OCBFs) designed in accordance with two versions of the AISC *Seismic Provisions*. The responses of frames with tension/compression as well as tension-only bracing are compared. The results show column stability is delayed to larger roof drifts in frames designed with enhanced design provisions. Further, the response of MT-BFs with tension-only bracing is also better than the corresponding frame with traditional tension/compression brace pairs.

1. Introduction

In single-story construction with tall, open spaces such as in industrial warehouses, airplane hangars, convention centers and stadiums, using a multi-tiered braced frame (MT-BF) is a practical and economical alternative to a single brace or pair of braces between the base and the roof. Horizontal struts divide the tall, single-story frame into multiple bracing panels or tiers, and there are no intermediate floor diaphragms or out-of-plane supports at the tier levels. The struts also provide in-plane bracing for the columns that can reduce the column size and the amount of steel used. MT-BFs are amenable to easy customization to meet project-specific needs: uniform or irregular tiers, and different bracing configurations including X, split-X, V, or inverted-V (chevron). The multi-tiered configuration can also be used within a tall story of a multi-story structure. In addition to system geometry, the type of bracing may also be varied: conventional tension/compression (T/C) members, buckling restrained braces (BRBs) or tension-only (TO) members. The columns are typically wide-flange sections oriented for strong-axis bending out-of-plane over the full frame height. Reduced brace lengths in MT-BFs are beneficial for ductile

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seismic design since it is easier to meet strict slenderness requirements. Smaller brace sections can be used that also lower seismic design forces in traditionally capacity-designed components of the frame such as columns, beams/struts, and connections.

In concentric MT-BFs, seismic energy is primarily dissipated through brace inelastic axial response. During a seismic event, unbalanced horizontal forces are likely to develop at the tiers due to differences in brace post-buckling response between tiers. Since MT-BFs do not have seismic mass at the tier levels, no inertial forces can counteract the force unbalance, and the force must be redistributed within the braced frame system itself. Prior research has shown that brace inelastic deformations tend to concentrate in the critical tier in which tension yielding is initiated (Imanpour et al 2013; 2016a; 2016b; Imanpour and Tremblay 2016). Rapid strength degradation of the compression brace in the post-buckling range reduces the shear strength of the critical tier and results in increasing drift concentration in this tier. This phenomenon tends to impose excessive ductility demands on the braces and increases the potential for premature low-cycle fatigue fracture. Additionally, brace tension yielding is commonly confined to the critical tier and flexural demands are imposed on the braced frame columns. Since braced frame members are considered to be only axially-loaded, seismic design provisions prior to the 2016 AISC *Seismic Provisions* do not directly consider the additional in-plane flexural demands on the columns, leaving them susceptible to instability. Column stability in MT-BFs is further compromised due to the lack of out-of-plane supports at the tier levels.

Prior research on this topic includes work that focuses on evaluating isolated MT-BF columns (Stoakes and Fahnestock 2012; Stoakes and Fahnestock 2016). Extensive research has also been conducted on high-ductility or special concentrically braced frames, MT-SCBFs (Imanpour et al 2013; 2016a; 2016b; Stoakes and Fahnestock 2014; Imanpour and Tremblay 2016) that provides the basis for the minimum strength and stiffness requirements introduced for multi-tiered special concentrically braced frames in the 2016 AISC *Seismic Provisions*. These provisions also contain the first generation of design guidance for multi-tiered ordinary concentrically braced frames or MT-OCBFs, however this is based on a limited evaluation. A comprehensive assessment is needed to determine the effectiveness of the provisions and to ascertain the need for any modifications to achieve satisfactory seismic performance for MT-OCBFs. A portion of the extensive design matrix of MT-OCBFs developed for a larger study on this topic is presented here and the seismic response is compared for MT-OCBFs (both with T/C and TO bracing) designed in accordance with the 2010 AISC *Seismic Provisions* (AISC 2010a) –the standard that is still widely used in practice – and the 2016 AISC *Seismic Provisions* (AISC 2016) guidance – the newest standard that is gradually being implemented more as jurisdictions adopt the latest codes. Results from nonlinear static pushover (NSP) and response history analysis are discussed.

2. Frame Design Matrix and Numerical Model

The 2010 AISC *Seismic Provisions*, which classifies MT-BFs as K-braced frames and prohibits their use, do not contain special treatment of MT-BFs. As such, the requirements prescribed for multi-story OCBFs are used in this study. In contrast, the 2016 AISC *Seismic Provisions* provide the first generation of design guidance for MT-BFs. For OCBFs, the 2010 AISC *Seismic Provisions* employ a simple approach to account for additional demands in the columns and connections. These elements 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 on the struts, as such they are designed to resist the full frame base shear in compression. The newest 2016 AISC *Seismic Provisions* specify an additional factor of 1.5 on the amplified horizontal earthquake effect ($1.5\Omega_0 Q_E$) for the columns, struts, and the connections to approximately account for in-plane flexural demands. In frames with T/C X-bracing, the axial forces in the struts are determined in the absence of the compression brace. Nominal out-of-plane moment demands are also included for the columns. One of the few seismic detailing requirements for OCBFs imposes a local slenderness limit on the brace cross-section that is consistent with a *moderately ductile* classification. No local slenderness constraints are imposed on the beams or the columns, however, this study considers only non-slender sections. The 2016 provisions also address MT-OCBFs with tension-only bracing: for columns and struts, the additional amplification of required strength by 1.5 is eliminated if the braces have a global slenderness ratio greater than 200 (advantageous for TO bracing), and in-plane moment arising from unbalanced tier-level forces is included in column design.

The theme building in the study is a single-story, industrial building with a 460ft x 180ft rectangular footprint that has been used in prior research on MT-BFs (Imanpour et al 2013). It has 180 ft trusses spanning its width and is located in a high seismic region in coastal California. Two braced frames, designed as OCBFs (R= 3.25; Ω_0 =2.0), are located on each side along the perimeter of the building. A subset of 35 ft tall frames with 20 ft wide bays (aspect ratio of 1.75) are presented here. Structural loads include a roof dead load (no live load) and exterior cladding weight of 25 psf each. The building is assigned to Site Class D and Risk Category II, with an importance factor for earthquakes, I_e, of 1. The design spectral accelerations, S_{DS} and S_{D1}, are 1.0g and 0.6g, respectively. The maximum fundamental period of the frame that is permissible for strength design, C_uT_a, is 0.40 seconds. The static design base shear per frame is 223 kips and includes a 10% amplification for torsion. Seismic design is in accordance with ASCE 7-10 (ASCE 2010), and the frame members and connections are proportioned as per the requirements of either 2010 AISC *Seismic Provisions* (AISC 2010a) or 2016 AISC *Seismic Provisions* (AISC 2016), as well as the AISC 360-10 *Specification* (AISC 2010b). The selected member sizes are listed in Table 1. Each frame is also assigned a unique label: (1) MT-OCBF and MT-TOCBF refer to multi-tiered ordinary concentrically braced frames with T/C and TO bracing, respectively; (2) aspect ratio, which equals 1.75 for the frames presented here; (3) number of tiers followed by the brace configuration; and (4) tier height uniformity where U refers to frames with equal tier heights.

Table 1: Frame member sizes.

Frame	Brace[1]	AISC 2010		AISC 2016	
		Column[2]	Strut/Beam[2]	Column[2]	Strut/Beam[2]
MT-OCBF-1.75-2X-U	HSS5x5x5/16	W12x72	W8x48	W14x99	W12x87
MT-OCBF-1.75-3X-U	HSS4.5x4.5x5/16	W14x74	W8x48	W18x97	W12x87
MT-TOCBF-1.75-2X-U	L5x5x3/4	W14x90	W8x48	W14x120	W12x65
MT-TOCBF-1.75-3X-U	L5x5x5/8	W18x86	W8x48	W14x120	W12x65

1. ASTM A1085 for HSS sections and ASTM A529 for equal-leg, single angles
2. ASTM A992 W sections with weak-axis bending in-plane for columns and struts, and strong-axis for the roof beam.

A three-dimensional, fiber-based numerical model was developed using the *OpenSees* (McKenna and Fenves 2006) simulation platform. Key model details are summarized in Table 2. The braced frame columns were modeled as continuous with pinned bases and torsional end restrains. Out-of-plane restraints were included for the roof strut. Point masses representative of the portion of the structure being modeled were applied to the top of the braced frame columns, and tributary gravity loads were assigned to the frame and the leaning column (P-Δ). Mass-proportional damping corresponding to 2% critical damping in the first mode was specified for the dynamic analysis. Ground motions were specified in the plane of the frame.

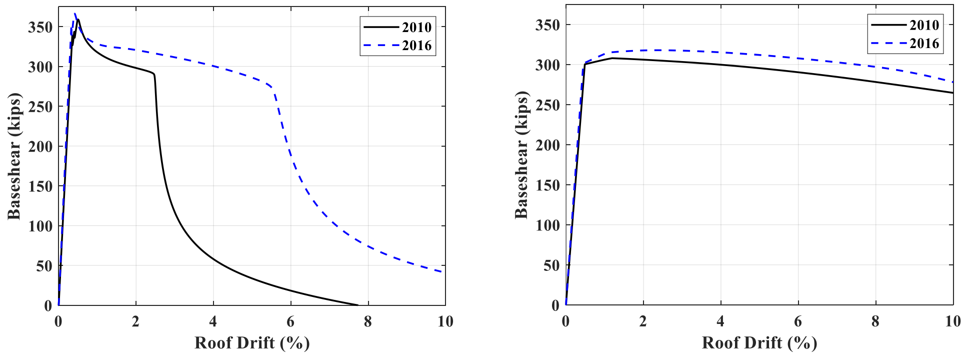
Table 2: Summary of numerical model details.

Member Type	Element Type	Section Type	Material Model	Yield Stress	Other
T/C Braces	Displacement beam-column	Fiber	Steel02	62.5 ksi[2]	Out-of-plane imperfections
TO Braces	corotTruss	-	Elastic, perfectly-plastic	60 ksi[2]	-
Braced Frame Columns	Force beam-column	Fiber	Steel02 with residual stress[1]	50 ksi	In-Plane and Out-of-plane imperfections
Struts/Beams	Force beam-column	Fiber	Steel02 with residual stress[1]	50 ksi	-
Leaning Column	Beam-column	Elastic	-	-	-
Connections	Beam-column	Elastic	-	-	-
	Zero-length	-	Steel02	Mgp[3]	

1. Lehigh residual stress pattern (Galambos and Ketter 1958)
2. Expected yield stress of the brace material. A 5% reduction was applied for first-tier braces to initiate inelastic response
3. Expected yield moment of gusset plates (Hsiao et al 2012)

3. Nonlinear Static and Response History Analysis

A nonlinear static pushover (NSP) analysis is employed to provide insight into the dynamic response of the frames under the first significant inelastic displacement cycle of an earthquake record. Fig 1(a) shows that the 2-tiered T/C frames have an initial linear elastic response until brace buckling is triggered in the first tier. With brace post-buckling response and yielding of the first-tier tension brace, inelastic drift concentration, which is characteristic of MT-BFs, begins to occur. The 2010 frame column buckles at a roof drift of 2.4%, and the first plastic hinge at the mid-height of the Tier 1 segment occurs at a roof drift of 2.5%. Any location along the column where the value of a stress resultant yield surface equation (Ziemian and McGuire 2002), based on the nominal section capacities, exceeds 1 is designated as a plastic hinge location. Since MT-BF columns have pinned end conditions with no intermediate out-of-plane supports, a single plastic hinge along the height can initiate buckling. In contrast, the first plastic hinge in the compression column of the 2016 frame appears at a drift of 3.8%, but it remains stable until a larger drift of 5.5% when the plastic hinge region expands to an adjacent node. This result shows the beneficial impact of the enhanced column design provisions. A similar plot for the 2-tiered TO frames is shown in Fig 1(b). The frames exhibit initial linear elastic response, followed by brace tension yielding. Braces in both tiers of the two frames yield and the columns also remain stable. A plastic hinge occurs near the Tier 1 level of the compression column in the 2016 frame at a drift of 8.8%, but the rapid and sudden loss in lateral resistance due to instability occurs at a drift beyond 10%. Plastic hinges also develop in the tension columns of the frames. Similar observations were made for the 3-tiered frames in this study where column buckling was delayed to larger roof drifts in the 2016 frames. No buckling was observed in the 3-tiered TO frames (up to 10% drift) despite the occurrence of plastic hinges.

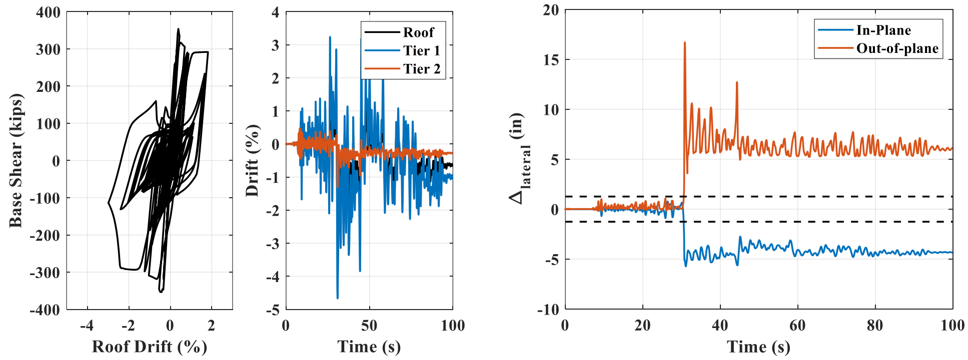


(a) (b)

Figure 1: Static pushover response: (a) MT-OCBF-1.75-2X-U; (b) MT-TOCBF-1.75-2X-U.

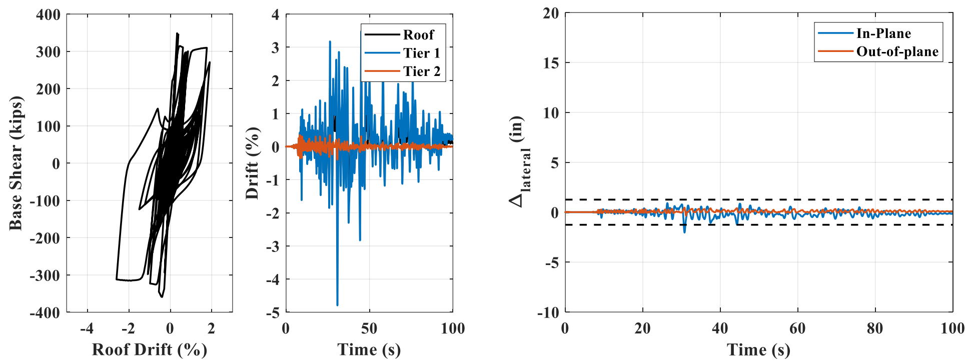
Nonlinear response history analyses were conducted using a suite of 44 “Far-Field” ground motion records (22 earthquake scenarios, each with 2 components) recommended in FEMA P695 (FEMA 2009). The record set was scaled such that the median of the suite matched the maximum considered earthquake (MCE) hazard at the design fundamental period of the frame.

The responses of MT-OCBF-1.75-2X-U, 2010 and 2016 designs, to the second component of the 1979, Imperial Valley, Delta ground motion record are shown in Figs 2 and 3, respectively. For the 2010 frame, a plastic hinge forms near the middle of the Tier 1 segment of the left-side column at a drift of 2.49%. Fig 2(a) shows that the lateral resistance of the frame decreases rapidly with increasing drift (similar to NSP) that is representative of column buckling. Under this ground motion, the frame attains a peak roof drift of 2.99% and a drift concentration factor, *DCF*, of 1.56, which is the ratio of the maximum local drift in any of the tiers and the corresponding roof drift. In contrast, the 2016 frame has a slightly lower peak roof drift of 2.56% but higher *DCF* of 1.85. Plastic hinges also occur in the columns of this frame at different times, but stability is maintained. Thus, the enhanced column section can prevent buckling, but the issue of large inelastic demands on the braces that increases the potential for brace fracture is unresolved. In addition to the plastic hinge criterion, the lateral displacements, away from a chord, at the mid-point of the first-tier column segment are useful to assess the severity of the column response. A threshold of three times the amplitude of the initial imperfection amplitude in the column is chosen (Houreh and Imanpour 2020) as a limit indicating significant damage due to the column becoming unstable in the simulation. Fig 2(b) shows that both the in-plane and out-of-plane residual displacements far exceed the threshold of 1.26 in. for the 2010 frame. Much smaller residual displacements develop in the 2016 frame, and these do not exceed the threshold as shown in Fig 3(b).



(a) (b)

Figure 2: Response of MT-OCBF10-1.75-2X-U to component 2 of the 1979, Imperial Valley, Delta record: (a) base shear vs. roof drift and drift history; (b) lateral displacement at mid-height of lowermost segment of LHS column.



(a) (b)

Figure 3: Response of MT-OCBF16-1.75-2X-U to component 2 of the 1979, Imperial Valley, Delta record: (a) base shear vs. roof drift and drift history; (b) lateral displacement at mid-height of lowermost segment of LHS column.

Median statistics from the full set of analysis are provided in Table 3. Peak roof drifts tend to be lower in all frames designed with the enhanced column design requirements of the 2016 *Seismic Provisions*. However, the *DCF* is indicative of significant inelastic drift concentration in the 2016 frames as well. This excessive ductility demand on the braces increases the potential for premature brace fracture. Frames with TO bracing had lower peak roof drifts than the corresponding T/C frames. The same is true for the *DCF* and is likely due to the lack of post-buckling response in TO braces. The table also lists the total number of analyses where at least one (instantaneous) plastic hinge occurred in the columns; however, the prior discussion shows that while this does not necessarily imply a buckled column, it is undesirable to exceed the column cross-section capacity during a seismic event. The column demands in the TO frames did not lead to a plastic hinge at MCE hazard, while several cases of plastic hinge were identified in the T/C braced frames. This may also be due to larger differences in tier shear strength in the latter during a seismic event that increase the magnitude of the unbalanced horizontal force and in-plane flexural demands on the braced frame columns.

Table 3: Median statistics from NLRH analysis (MCE hazard).

Frame	AISC 2010			AISC 2016		
	Peak Roof Drift (%)	DCF	Plastic Hinge	Peak Roof Drift (%)	DCF	Plastic Hinge
MT-OCBF-1.75-2X-U	1.27	1.66	1	1.14	1.69	6
MT-OCBF-1.75-3X-U	1.22	2.14	12	1.15	2.35	20
MT-TOCBF-1.75-2X-U	0.88	1.52	0	0.86	1.53	0
MT-TOCBF-1.75-3X-U	0.83	2.13	0	0.68	1.95	0

4. Conclusions

The seismic stability of columns in multi-tiered ordinary concentrically braced frames, MT-OCBFs, is of significant importance due to the unique conditions during a seismic event that impose additional flexural demands on the braced frame columns. The MT-OCBFs in this study were designed in accordance with the 2010 AISC *Seismic Provisions*, as well as the latest 2016 AISC *Seismic Provisions* that contain special guidance for the design of multi-tiered frames to approximately account for additional demands. Nonlinear static and dynamic (at MCE seismic hazard) analyses were employed to compare the performance of a subset of MT-OCBFs developed for a larger study; both tension/compression and tension-only bracing were evaluated. Column stability was evaluated by considering both the overall section capacity (plastic hinge criterion) as well as the residual lateral deformations. The results showed that column buckling in frames designed in accordance with the enhanced criteria in the 2016 AISC *Seismic Provisions* guidance is delayed to larger roof drifts, but not necessarily eliminated. Significant inelastic drift concentration is also observed that can lead to brace fracture. Frames with tension-only bracing tend to have better performance and columns remain stable up to larger drifts due to smaller unbalanced forces that can be resisted through column flexure.

Acknowledgments

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