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EXPERIMENTAL STUDY OF TOMORROW'S STEEL BRACED FRAMES IN BUILDING STRUCTURES

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ABSTRACT

This paper briefly summarizes on-going experimental studies about the type of steel concentrically braced frames that are widely used in North America. Experimental results are presented here for a NEES small group project entitled "International Hybrid Simulation of Tomorrow's Braced Frames". Four large-scale two-story braced frames with different bracing member cross sections and gusset plate-to-beam connection details are designed and tested at the NEES facility at UC Berkeley. Test results are primarily used to refine the analytical models, confirm the current design practice and also validate the seismic performance of steel concentrically braced frames. Future research directions and experiments of the project are also discussed.

Introduction

Over the past few decades, the steel braced frame has thought to be one of the most efficient and economical seismic load resisting systems to control the deformation of structures. After several severe earthquakes strike major cities around the world, some anticipated and unanticipated damages (AIJ, 1995; Bonneville and Bartoletti, 1996; Kelly et al., 2000) were found in this kind of system. This reminds researchers and engineers to think about new ways to improve the behavior of braced frame systems. Several approaches have been proposed to enhance braced frame system behavior by making changes at the component level by using innovative devices such as buckling restrained braces (Watanabe et al., 1988) and self-centering braces (Christopoulos et al., 2008). Some research focuses on reducing deformation concentration by better distributing inelastic demand along the full height of the structure (Khatib et al., 1988; Tremblay and Merzouq, 2004). Although many experimental studies of the conventional buckling brace components and some braced frame specimens have performed in the past three decades (Black et al., 1980; Ballio and Perotti, 1987; Lee and Goel, 1987; Bertero et al., 1989; Tremblay, 2002; Yang and Mahin, 2005; Uriz and Mahin, 2008; Clark et al., 2008), the number of studies on the large-scale concentric braced frames is still limited. This paper briefly describes test results for two nearly full-scale, two-story, one bay concentric braced frame specimens tested at the NEES Berkeley site.

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Objectives and Scope

The main objectives and scope of the test program are as follows:

1. To obtain the experimental data on the behavior of key components.
2. To validate the analytical models.
3. To identify the improved design concepts and structural details.
4. To confirm the improvements by quasi-static cyclic loading tests and hybrid simulations of large-scale braced frame specimens.

Experimental Program at UC Berkeley

Test Setup

The experiments are conducted in the NEES facility at UC Berkeley. The overview of the test setup is shown in Fig. 1. Thirty reconfigurable reaction blocks are post-tensioned and grouted together horizontally and vertically to create an integrated reaction wall. The maximum base shear capacity of the test setup is 900 kips (600 kips at upper level and 300 kips at lower level) under the loading configuration illustrated in Fig. 1. This represents the loading condition of a typical braced bay at the corner of a building. Two 1.5 M-lb actuators with ±12 inch stoke are used at each floor level, which can impose around 5% inter-story drift on each test specimen. A heavy built-up floor beam is provided between the frame specimen and the strong floor to spread out the concentrated forces between the interfaces. Lateral stability frame is also provided in the test setup as illustrated in Fig. 1.

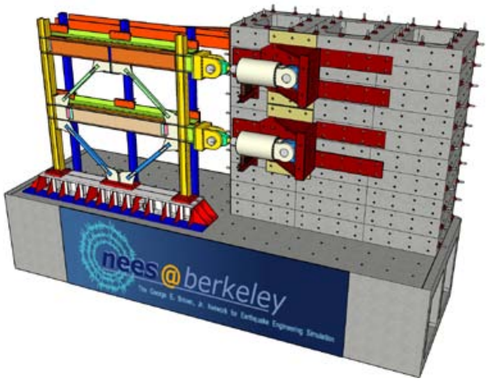


Figure 1. Overview of the test setup

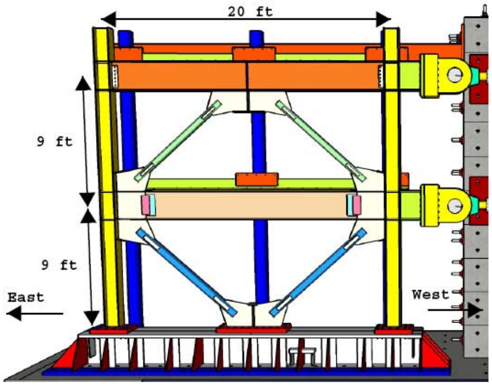


Figure 2. Dimension of test specimen

Description of Test Specimens

There are four braced frame specimens in the special concentric braced frame test program. Each test specimen consists of a two-story, single bay concentric braced frame which is designed and detailed in compliance with the AISC Seismic Provisions for Structural Steel Buildings (AISC, 2005). The story height is 9 feet tall measured from beam center line to center line distance, and the bay width is 20 feet long measured from column center line to center line distance as shown in Fig. 2. The name, member size, material type and test method of each specimen are shown in Table 1. Figs. 3 and 4 show the photo of specimen TCBF-B-1 and TCBF-B-2 before testing. It should be noted that the roof beam, two columns and base plates are re-used after the testing of TCBF-B-1 specimen.

Table 1. Name, member size, material type and test method of the specimens.

Name	**Column & Beam**	**Brace**		**Test Method**
TCBF-B-1	W12 x 96 (Column) (ASTM A992) W24 x 117 (Roof Beam) W24 x 68 (Lower Beam) (ASTM A992)	HSS 5 x 5 x 5/16 HSS 6 x 6 x 3/8	ASTM A500B	Cyclic Loading
TCBF-B-2		HSS 5 x 1/2 HSS 6 x 1/2	ASTM A500B	Cyclic Loading
TCBF-B-3		W 8 x 21 W 8 x 28	ASTM A992	Cyclic Loading
TCBF-B-4		HSS 5 x 5 x 5/16 HSS 6 x 6 x 3/8	ASTM A500B	Hybrid Simulation

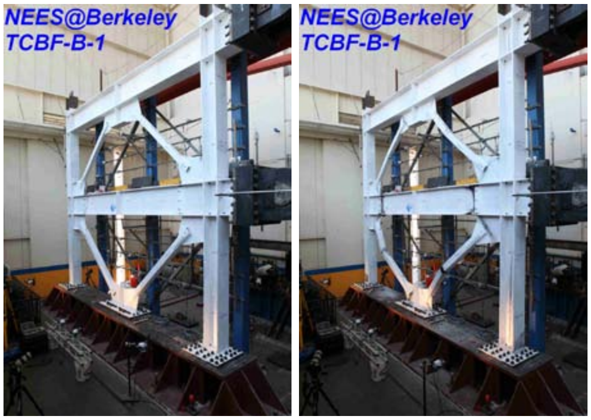


Figure 3. Specimen TCBF-B-1, before test (left), after test (right)

Figure 4. Specimen TCBF-B-2, before test (left), after test (right)

Figure 5. The one-piece gusset plate

Figure 6. Detail view of reinforcing plate

The gusset plates which connect braces from both stories in each specimen are 3/4” thick, one-piece gusset plates with two finger plates welded on them and spliced to the W24 x 68 lower beam (Fig. 5). The 2t brace-to-gusset plate recommended separation for out-of-plane buckling of the bracing system is used in each specimen to represent the typical detail in the design practice. Reinforcing plates at the net section of the braces are provided to prevent premature failure of bracing components (Fig. 6). More detail descriptions of test specimens are presented in the companion test report (Lai, 2009).

Loading Sequence and Instrumentation

The displacement of the roof beam is monitored and controlled during the entire test process. The upper level actuator is displacement controlled and the lower level actuator is force controlled with one-half of the force feedback from the load cell in the upper level actuator used as the command signal. This makes the lateral force pattern an inverted triangular distribution through out the entire experiment. The test protocol is modified from the Appendix T of AISC Seismic Provisions (AISC, 2005) in order to compare with the results of future BRBF testing in the research project. Additional eight cycles corresponding to one half of the elastic design drift (6@0.5D_{be}) and one elastic design drift (2@D_{be}) are added to the test protocol. Fig. 7 shows the cyclic loading protocol for the first three specimens. During the entire test process, the loading is paused when major observations are found and the loading is terminated following the cycle where both braces at a particular story (typically the first story) completely fracture. Each specimen is instrumented with displacement transducers, sticks, slip gages and wire pots to measure the inter-story drifts, relative displacements and local deformation of members or connections. Linear type strain gages and rosettes are also glued on the specimen to recover the internal force distributions during the tests. The whole specimens are painted with whitewash to observe the yield pattern (Figs. 3 and 4).

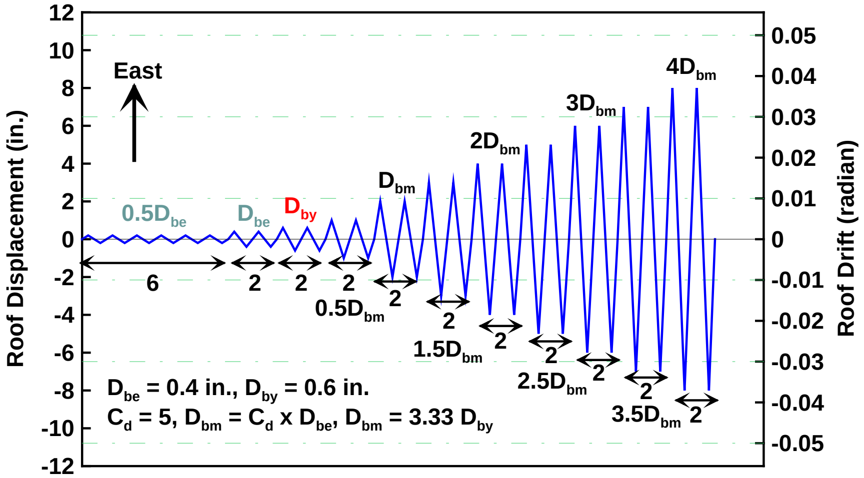


Figure 7. Cyclic loading protocol for TCBF-B-1, TCBF-B-2 and TCBF-B-3 specimens

Test Results of the First Two Specimens

For both experiments, the inter-story shear vs. inter-story drift data is obtained from the data acquisition system and monitored through the entire test process. The measured hysteretic loops provide valuable data to compare the behavior of the specimens. The following paragraphs describe the main observations for each specimen.

Specimen TCBF-B-1 (Square HSS Braces)

The base shear vs. controlled roof displacement of the specimen is shown in Fig. 8. The relationship between story shear and inter-story drift for specimen TCBF-B-1 is shown in Fig. 9. Table 2 illustrates and lists the major observations on the testing protocol with brief descriptions.

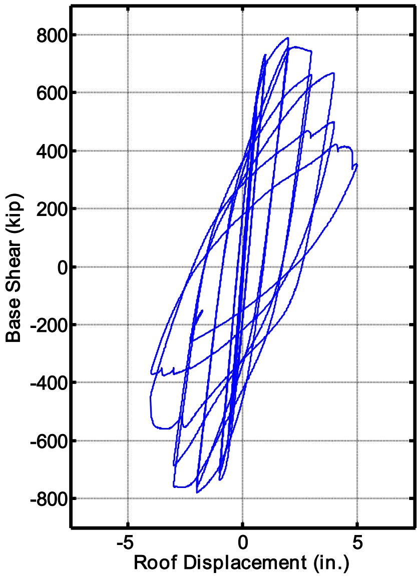


Figure 8. Base shear vs. roof displacement relationship for TCBF-B-1

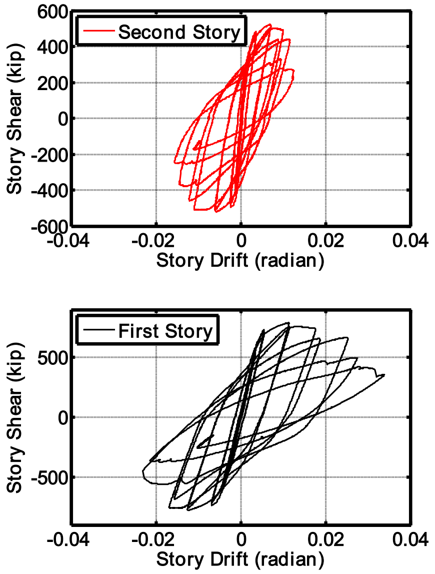
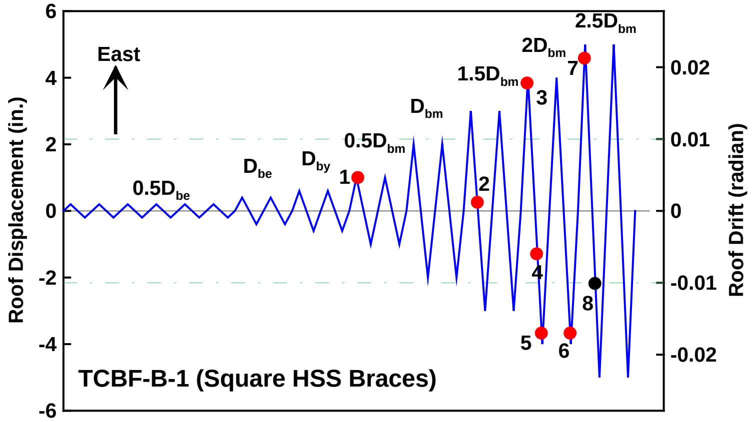


Figure 9. Story shear vs. inter-story drift relationship for TCBF-B-1

Table 2. The major observations for TCBF-B-1 specimen.


Major Observations

1	The braces at both stories start global buckling. (all out-of-plane to the north side)
2	The braces start local buckling at the middle portion of the brace. The top flange and web of lower beam at western side start local buckling. The bottom flange and web of lower beam at eastern side start local buckling (Fig. 10).
3	Cracks initiated in the outside corner of HSS brace at first story for both braces.
4	The cracks in the first story western brace propagated from outside corners to the center of the HSS brace section during the first cycle of the load step (Fig.11).
5	The western brace at ground story completely fractured during the first half cycle of loading to a roof displacement of 3.8 inch to the west. Crack initiated at the outside corner of HSS brace section at second story for both braces.
6	The bottom flange at eastern side of lower beam fractured at the CJP weld line during the second cycle of 2.0 D_{bm} load step at roof displacement about 3.8 inch to the west (Fig. 12).
7	The eastern brace at first story completely fractured during the first cycle of 2.5 D_{bm} load step at roof displacement corresponded to 4.7 inch to the east (Fig. 13).
8	The cracks in the second story eastern brace propagated from outside corners to the center of the HSS brace section during the first cycle of 2.5 D_{bm} load step. Test stopped at about 2.2 inch of roof displacement to the west.

Figure 10. The beam-to-gusset plate splice at east side of lower beam

Figure 11. The brace at first story (West)

Figure 12. The beam to one-piece gusset plate connection at east side

Figure 13. The brace at first story (East)

Specimen TCBF-B-2 (Round HSS Braces)

This specimen re-uses the roof beam, the columns and base plates from the previous test. Before the first trial, some welding repairs are conducted at the column bases. During the first trial at about 2-in. roof displacement (the second cycle of 1.5 D_{bm} load step), the west side column base fractures (Fig. 14) and then the test pauses to repair the column base (Fig. 15) to continue the subsequent loading. The base shear vs. controlled roof displacement of the specimen is shown in Fig. 16. The relationship between story shear and inter-story drift for specimen TCBF-B-2 is shown in Fig. 17. Table 3 illustrates and lists the major observations on the testing protocol with brief descriptions.

Figure 14. Fracture of the column base flange welding at west side

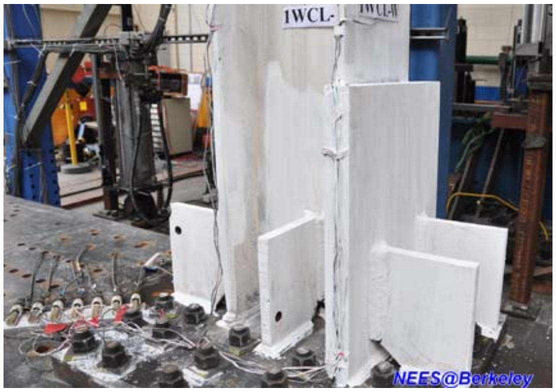


Figure 15. The west side column base after repair with stiffener plates

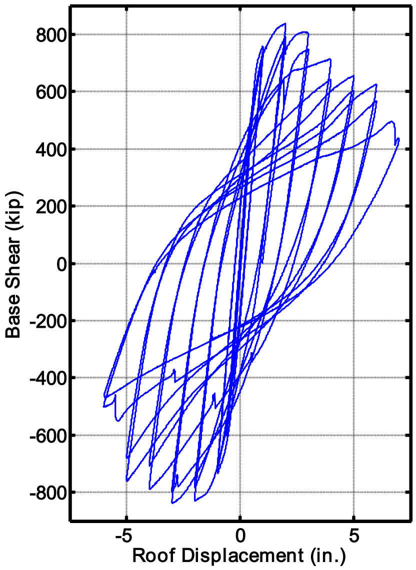


Figure 16. Base shear vs. roof displacement relationship for TCBF-B-2

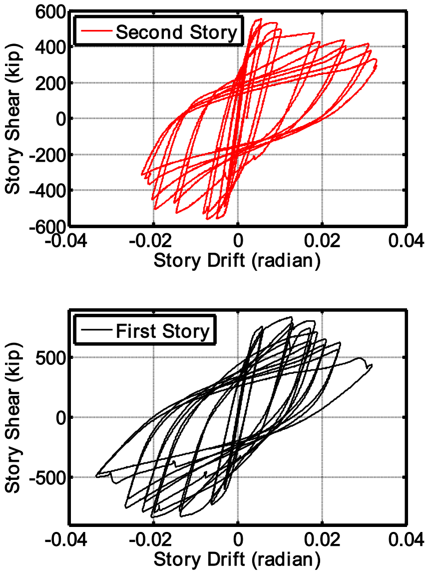
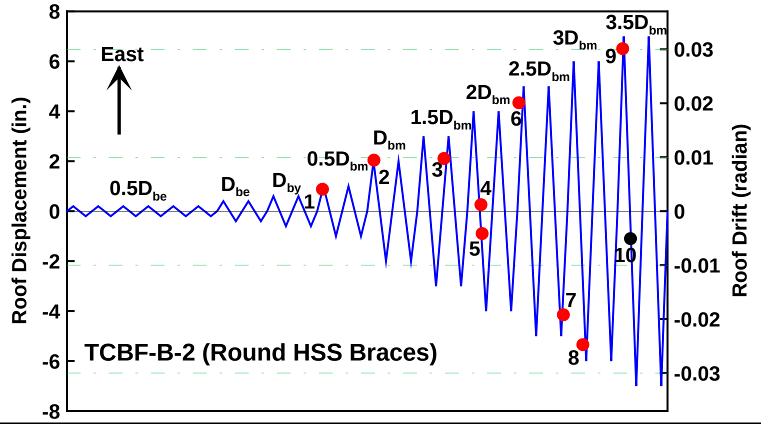


Figure 17. Story shear vs. inter-story drift relationship for TCBF-B-2

Table 3. The major observations for TCBF-B-2 specimen.



	Major Observations
1	The brace at first story west side starts global buckling. (out-of-plane to the south side)
2	The brace at second story east side starts global buckling. (out-of-plane to the north side)
3	The flange-to-base plate weld at the west side of column base completely fractured. Test paused to repair column base. (Figs. 14 and 15)
4	The top flange at western side of lower beam splices completely fractured at the CJP weld line (Fig. 18). The bottom flange at eastern side of lower beam splices partially fractured at net section of finger plate.
5	The bottom flange at eastern side of lower beam splices completely fractured at net section of finger plate (Fig. 19).
6	The braces start local buckling at the middle portion of the brace.
7	Cracks initiated in the middle of round HSS brace at first story western brace. (Fig. 20)
8	The western brace at ground story completely fractured during the first cycle of 3.0 D$_{bm}$ load step at roof displacement of 5.3 inch to the west. (Fig. 21)
9	The eastern brace at first story completely fractured during the first cycle of 3.5 D$_{bm}$ load step at roof displacement corresponded to about 7 inch to the east.
10	No local buckling or cracks in the second story braces were found. Test stopped at about 1.0 inch of roof displacement to the west.

Figure 18. The beam splice at west side

Figure 19. The beam splice at east side

Figure 20. The cracks in the middle of western side of round HSS brace

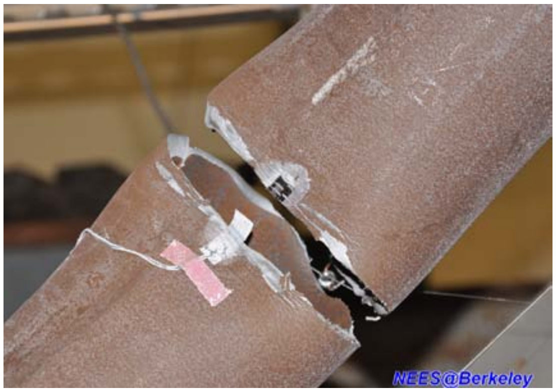


Figure 21. Completely fracture of the brace at first story (West)

Discussions and Conclusions from Two Experimental Results

From the results shown above, we can see that under similar base shear capacity, the framed specimen with round HSS braces exhibits better ductility capacity than the one with square HSS braces. The peak base shear degrades slower and the local buckling of the braces occurred later in TCBF-B-2 specimen under the same test sequence. Besides, the story drift at each story of TCBF-B-2 specimen tends to be more uniform than TCBF-B-1 specimen. The story shear vs. story drift relationship at each story for both specimens is not symmetric once the brace fractured in certain floor level. The brace and gusset plate configuration of the specimen tends to amplify the rotation demand at both ends of lower beam-to-gusset splices similar to an EBF link beam. From the observation during the tests, once the plastic hinges formed at both ends of the lower beam, even if top or bottom flanges partially (or completely) fracture, the frame specimen redistribute the internal force to resist larger roof lateral displacement which implies that a pinned beam-to-gusset details might be used to avoid local damage (flanges or web buckling and fracture) at this region. In specimen TCBF-B-2, the net section failure mode (Fig. 19) happened in the finger plate of one-piece-gusset which indicates a modified finger plate to gusset plate details should be used to avoid this failure mode. Yield patterns (photos not show here) on the column flanges also indicate that significant torsion and biaxial bending occurred in the column under large deformation which can cause unexpected local damage such as column base failure. Frame action also observed in the tests but not sure how much it affects the results, further research is needed.

Note that this paper only shows the SCBF experimental part of the research, the analytical part and the BRBF experimental part will be presented more detail in the project progress report in the future.

Acknowledgments

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