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FULL-SCALE CYCLIC TESTING OF A LOW-DUCTILITY CONCENTRICALLY-BRACED FRAME

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Abstract. *Steel concentrically-braced frames (CBFs) are used extensively as lateral-force-resisting systems for low to mid-rise buildings in moderate seismic regions of the United States, such as the East Coast and Midwest. Although good structural performance of CBFs in moderate seismic regions for typical gravity and wind loading is well-established, there is essentially no data for earthquake loading. As a result of this situation, a research project was initiated to investigate the seismic performance of CBFs in moderate seismic regions. This paper summarizes one aspect of the project: a full-scale cyclic test of a one-bay two-story CBF designed assuming R=3 and not specifically detailed for seismic resistance – focusing on the sequence of limit states and associated system behavior. The frame experienced brittle brace buckling in both upper story braces at ±0.35% frame drift. Brace-to-gusset weld fracture was subsequently induced in the lower story to observe the influence of brace re-engagement on system strength.*

1 INTRODUCTION

Ductile lateral-force-resisting systems, which were developed through research documenting their nonlinear behavior, are used extensively in high seismic regions. These systems – such as special moment resisting frames (SMRFs) or special concentrically-braced frames (SCBFs), are created using capacity-based design procedures with comprehensive detailing requirements such that brittle behavior is avoided. In contrast, lateral-force-resisting systems (LFRSs) used in moderate seismic regions typically have modest or no ductile detailing requirement and capacity-based design procedures, and there is little experimental data related to the nonlinear behavior of these more brittle structural systems. For example, steel concentrically-braced frames (CBFs) are used widely for low to mid-rise buildings in the East Coast and Midwest of the United States because of their high stiffness-to-weight ratio in the elastic range, but their inelastic seismic performance is essentially unstudied.

A research project was initiated to investigate the seismic performance of CBFs in moderate seismic regions with focus on the influence of reserve capacity: lateral-force-resisting mechanisms outside of the primary load path. This project includes a comprehensive integration of analyses and full-scale tests for components, connections and systems. Two full-scale tests were conducted in 2014 at the Network for Earthquake Engineering Simulation (NEES) facility located at Lehigh University. These tests aimed to provide a better understanding of the type and hierarchy of damage mechanisms that occur in low-ductility CBFs. This paper summarizes one of the tests, a CBF not specifically detailed for seismic resistance.

2 PROTOTYPE BUILDING

A three-story prototype building was used to contextualize the design of the braced frame studied in this experiment [1]. The rectangular floor plan of the prototype building is 45.7 m by 53.3 m [150 ft by 175 ft] with five equal bays in both directions. A pair of single-bay braced frames is placed in both directions. Story heights are uniformly 4.57 m [15 ft]. The prototype building was assumed to be located in Boston, MA with soil Site Class D and Seismic Design Category B. The seismic load effects were determined by the equivalent lateral force procedure in ASCE 7 [2].

For the experiment discussed here, the LFRS in the prototype building was designed as a CBF in a chevron (inverted-V) configuration using a response modification factor R equal to 3. US seismic design procedures allow for steel frames to be designed using R=3 without imposing any seismic detailing requirements [2, 3]. As a result, the failure mechanisms of these frames are typically brittle in nature. The seismic weight of the prototype building was 28.5 MN [6,400 kips] and in accordance with the equivalent lateral force procedure [2], the design base shear, V_B', was = 1.5 MN [336 kip]. Here V_B' represents the design base shear of half the building (one braced frame) and V_B represents the experimental base shear.

3 EXPERIMENTAL SETUP

An elevation of the experimental setup is shown in Figure 1. Story heights were 4.57 m [15 ft], and the bay width was 10.7 m [35 ft]. The actuators were aligned 15.2 cm [6 in] above the beam centerlines at Levels 2 and 3, and fixed to a pair of HSS members, which were used to load the test unit. Fixtures at the bases of the columns simulated a pin boundary in plane and a pair of linkage fixtures simulated a horizontal roller boundary at Level 1.

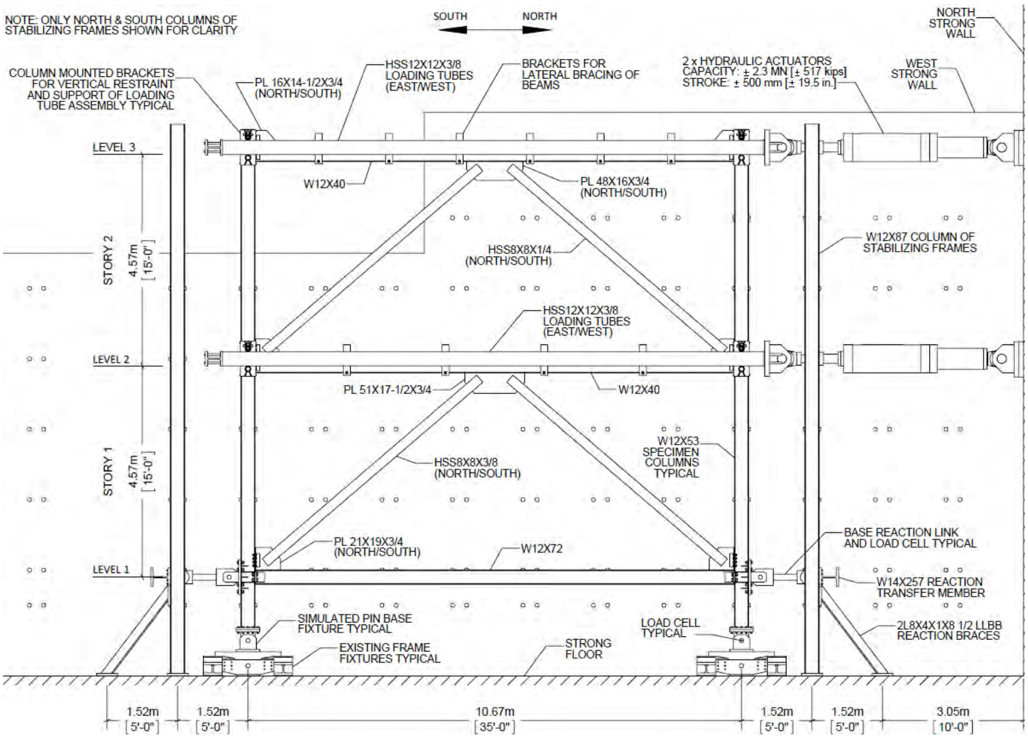


Figure 1. Experimental setup elevation.

When viewing the frame as in Figure 1, North is to the right and South is to the left. For drift measurements, the sign convention was positive to the North and negative to the South. Actuator displacements and forces were negative when the test unit was drifting South and positive when drifting North.

4 LOADING PROTOCOL

The loading protocol for this experiment was quasi-static and cyclic. For the primary loading pattern, the Level 3 actuator (see Figure 1) was kept in displacement control while the Level 2 actuator was slaved by force to the Level 3 actuator. The ratio of applied forces $F_{A3} : F_{A2}$ was held constant at 3.42. Since only the bottom two stories of the three-story prototype frame were used in the experiment, the force ratio was determined by adding the Level 4 and Level 3 seismic design forces derived from the equivalent lateral force analysis [2]. In this way, the experimental story shears simulated the existence of a Story 3 to maintain consistency with design shears. The experiment advanced under this loading protocol in ±0.05% frame drift increments until buckling occurred in both Story 2 braces, resulting in a softened Story 2.

After the formation of a softened Story 2, it became evident that the original loading protocol could no longer transmit enough force into the frame to adequately exercise Story 1, which had remained primarily elastic up to this point in the test. As a result, a secondary loading pattern was developed to allow further exploration of Story 1 behavior. The secondary loading pattern delivered demand to the frame entirely through the Level 2 actuator (with $F_{A3} = 0$) to remove demands on the weakened Story 2.

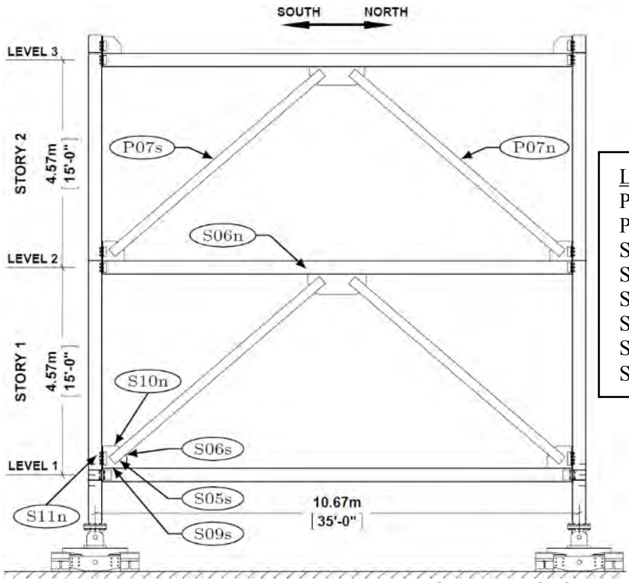
For clarity, *Loading Pattern P (Primary)* is used hereafter to identify the original loading pattern using a ratio of applied forces $F_{A3} : F_{A2} = 3.42$, and *Loading Pattern S (Secondary)* is used to identify a modified loading pattern using the Level 2 actuator only (with $F_{A3} = 0$). A summary of the overall loading history detailing when the two loading patterns were used is provided in Table 1. The loading history is divided into four phases during which the loading pattern is alternated from *Loading Pattern P* to *Loading Pattern S*. Key results and observations from the test are additionally summarized in Table 1.

Table 1. Summarized loading history for *R*=3 chevron test frame

Cycle	Target Frame Drift	Summary
Phase I: *Loading Pattern P (Primary)* – [Original loading pattern] – (Figure 3)		
P01-P06	±0.05% to ±0.30%	Elastic behavior
P07n	+ 0.35%	Story 2 North brace buckling
P07s	- 0.35%	Story 2 South brace buckling
Phase II: L*oading Pattern S (Secondary)* – [Attempt to weaken Story 1]		
S01n	N/A	Failed to damage Story 1
Phase III: *Loading Pattern P* – [Fatigue cycles on Story 2 braces] – (Figure 3)		
P08	±0.50%	Soft Story 2, Story 1 undamaged
P09	±1.00%	Drift limited - braces bearing on reaction frame
P10-P42	±1.00%	Brace degradation; Level 3 beam pull-down
Phase IV: *Loading Pattern S* – [Artificially weaken Story 1] – (Figure 4)		
S02s-S04s	N/A	Removing weld from Story 1 South brace
S05s	N/A	Story 1 South brace lower weld fracture
S06-S09	±0.50% to ±2.00% Story 1 Drift	Brace re-engagement; Level 2 beam hinging
S10n	+ 3.00% Story 1 Drift	Brace slip out of plane off gusset plate
S11n	+ 6.00% Story 1 Drift (maximum)	Story 1 column hinging at base

5 EXPERIMENTAL OBSERVATIONS

Figure 2 is an annotated elevation of the test frame that illustrates the cycle and location of the critical observations that were made during the test. For example, the label P07n at mid-height of the Story 2 North brace indicates an observation was made (brace buckling in this case) here in the North direction (indicator “n”) of the 7th cycle (indicator “07”) of the primary loading pattern (indicator “P”). Observation labels are additionally provided on a complete cyclic hysteresis plot of base shear, V_B, versus total frame drift, δ_T, for *Loading Pattern P* in Figure 3 as well as a hysteresis plot of base shear, V_B, versus Story 1 drift, δ_1, for *Loading Pattern S* in Figure 4. The maximum base shear under the standard loading pattern (*Loading Pattern P*) was 2060 kN [464 kip] and occurred at 0.35% total frame drift. The base shear was 641 kN [144 kip] at a total frame drift of 1.0%, the maximum drift achieved under *Loading Pattern P*. The horizontal dashed lines on Figure 3 mark the design base shear of 1500 kN [336 kip].



Legend
P07n – Brace buckling
P07s – Brace buckling
S05s – Brace-to-gusset weld fracture
S06s – Brace re-engagement on gusset plate
S06n – Beam yielding
S09s – Brace re-engagement on beam flange
S10n – Brace slip off gusset plate
S11n – Column yielding

Figure 2. Frame elevation with observation locations.

5.1 Phase I: *Loading Pattern P* [Original loading pattern]

The frame remained elastic during the first six cycles, P01-P06 (up to ±0.30% frame drift, or ±27.4 mm [1.08 in] Level 3 displacement). While pushing the frame North to a target total displacement, δ_T, of 32 mm [1.26 in] in cycle P07n (see Table 1), global buckling occurred in the Story 2 North brace (Figure 2). The buckling limit state was brittle, resulting in a sudden drop in base shear from 2060 kN [464 kip] to 665kN [150 kip]. This caused the frame to move forward past the target drift of 0.35% drift to nearly 0.45% drift. Upon cycling the frame back to zero load, permanent deformation was visible in the brace (Figure 5a). Significant flaking of whitewash was also apparent at the brace mid-span, identifying that it had yielded and buckled locally. This yielding and buckling were localized approximately to an area extending to one section width (203mm [8 in]) in each direction from the mid-span.

Upon reloading the frame and pushing towards the next target displacement (-0.35% drift), the overall stiffness of the frame increased as the buckled Story 2 North brace was pulled in tension, but the stiffness was noticeably smaller than the elastic range (Figure 3). In this half-cycle (P07s), the Story 2 South brace buckled (Figure 2) as the Story 2 North brace had in the half-cycle prior. Buckling occurred at a drift of -0.30% and base shear of -1935kN [-435 kip]. The brittle nature of the brace buckling resulted in a drop in base shear to -665kN [-150 kip] and an increase in drift to the target of -0.35%, ending the cycle.

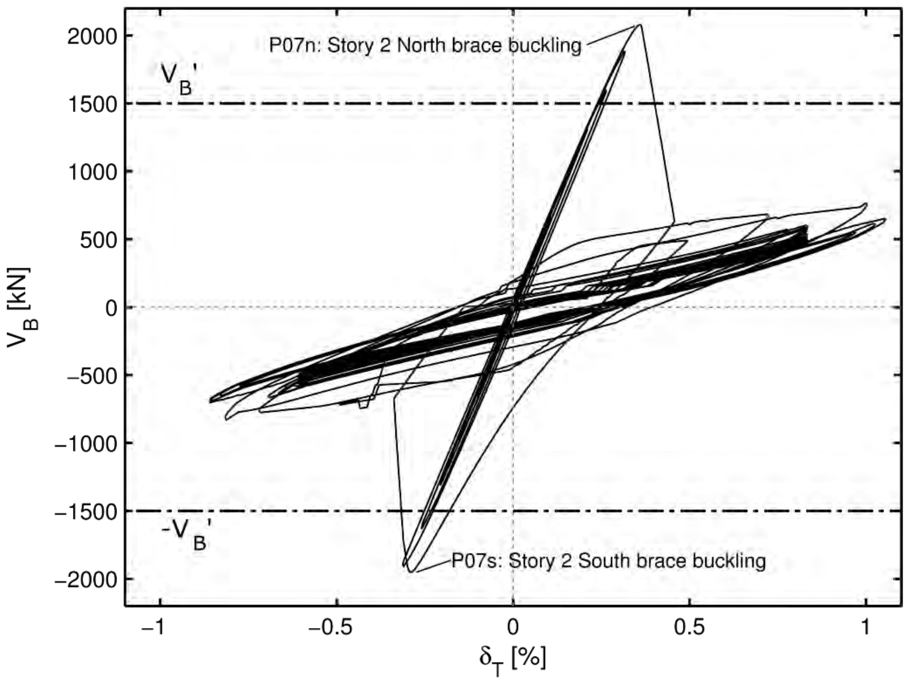


Figure 3. Base Shear vs. Total Frame Drift – *Loading Pattern P* (Phase I & III).

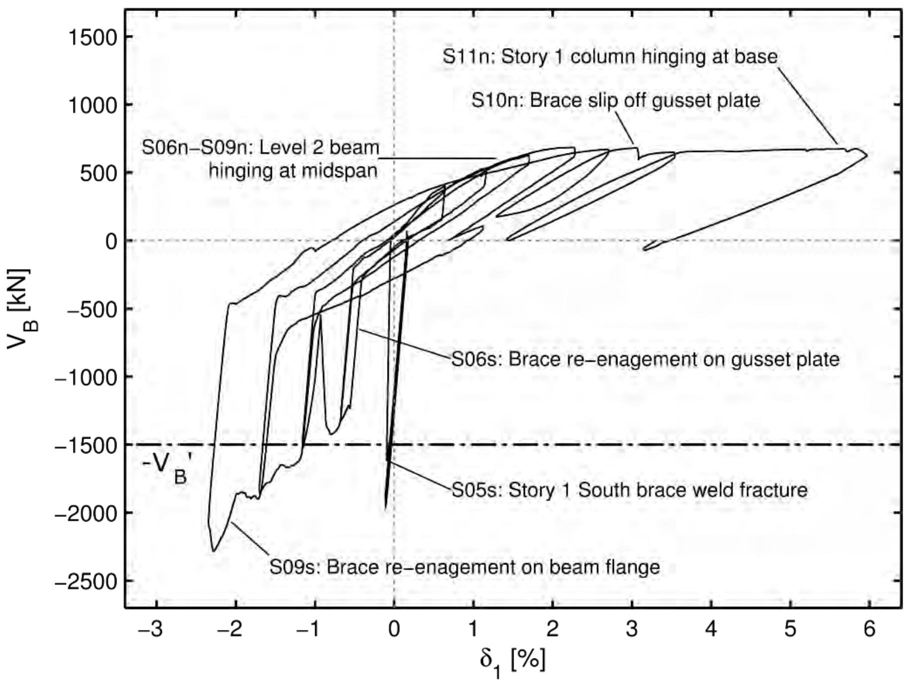


Figure 4. Base Shear vs. Story 1 Drift – *Loading Pattern S* (Phase IV).

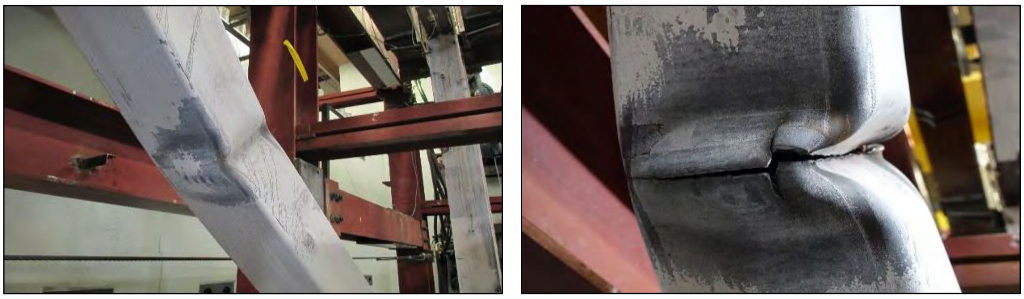
Buckling in both Story 2 braces resulted in a soft-story mechanism, as the story stiffness dropped 97% from 117kN/mm [625kip/in] to 4 kN/mm [25kip/in]. Due to the formation of a soft Story 2, the 3.42:1 load ratio used by *Loading Pattern P* could no longer deliver enough shear into Story 1 to exercise it sufficiently. Thus, the loading pattern was modified to exercise Story 1 more extensively.

5.2 Phase II: *Loading Pattern S* [Attempt to weaken Story 1]

In an attempt to induce a weld fracture or brace buckling in Story 1, the Level 2 actuator alone was used to load the frame monotonically to the North. Shortly before reaching the capacity of the actuator (2300kN [520 kip]), the Level 2 beam began to yield and deform locally under the high loads, requiring unloading of frame to prevent further damage. Neither weld fracture nor brace buckling occurred in Story 1 during this cycle, and other than the aforementioned localized inelastic response resulting from the large compressive forces, the story remained essentially undamaged. Upon unloading, a residual increase in frame drift of 0.2% was observed due to the localized deformation.

5.3 Phase III: *Loading Pattern P* [Resume original loading protocol]

Next, the original loading protocol was resumed in an attempt to fracture one of the buckled Story 2 braces in tension. The Story 2 drift reached peak positive and negative values of +2.00% and -1.50% before the half-cycles needed to be stopped as the braces had buckled over 300 mm [12 in] out of plane and were beginning to laterally bear on the test unit's stabilizing frame (Figure 1). Unable to further increase the drift, the frame was subjected to 33 fatigue cycles under *Loading Pattern P*, causing the braces to degrade significantly to a point where only a single face of the original HSS section remained (Figure 5b). However, the braces never fractured through the full section. As the brace sections deteriorated from repeated cycling (P10-P42), the gusset-to-gusset length of the braces effectively decreased, resulting in a Level 3 beam midspan deflection of -51mm [-2 in] at the end of the fatigue cycles.



(a) (b)

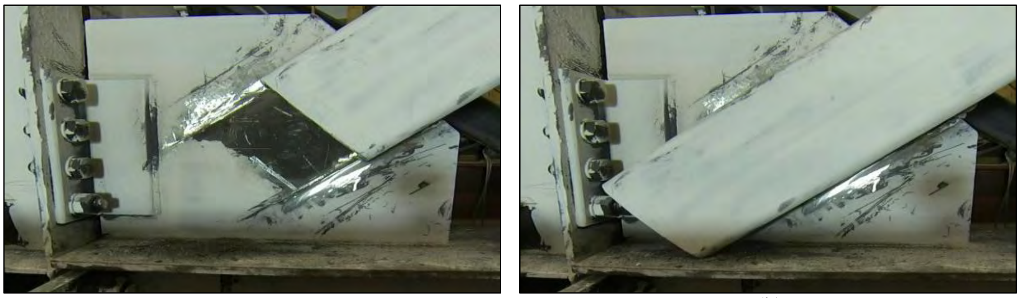
Figure 5. Story 2 North brace buckling: (a) initial buckling; (b) end of test.

5.4 Phase IV: *Loading Pattern S* [Artificially weaken Story 1]

In a final effort to exercise Story 1, portions of the brace-to-gusset welds at Level 1 on the South side were removed by incrementally grinding them down until weld fracture could be obtained. Multiple attempts were made to reduce the weld length and induce a weld fracture in subsequent loading. Each of the four welds in the gusset-to-brace weld group were ground off roughly 2 inches at a time before each attempt (P02s-P04s). Finally, on the 4th attempt (P05s), weld fracture occurred at V_B = 1675kN [375 kip]. The relationship between this capacity and the original weld capacity is uncertain, since it was difficult to identify exactly how much weld had been removed from the connection. The welds, which had partially filled the gap between HSS brace and gusset plate were especially hard to reach or remove completely with a grinding wheel. As expected, the brittle nature of the weld fracture resulted in a large instantaneous drop in base shear from 1670kN [375 kip] to approximately zero (Figure 4).

After weld fracture was induced, the frame was cycled in ±0.5% Story 1 drift increments, still from the Level 2 only (with $F_{A3} = 0$). In the positive (North) direction, the fractured Story 1 South brace was pulled in tension, only capable of transferring force through friction with the gusset plate (Figure 6a). As the fractured brace could not take much demand through friction, the Level 2 beam carried a significant amount of demand in these half cycles (P06n-P11n) through flexural hinging at the midspan. During these half cycles the frame exhibited behaviour similar to that of an eccentrically-braced frame (EBF); the half of the Level 2 beam corresponding to the side of the fractured brace underwent significant flexural deformations similar to those experienced by long "flexural links" found in EBFs. As shown in Figure 4, this behaviour was very ductile, with a stable strength plateau up to 6% Story 1 drift.

In the negative (South) direction, the frame remarkably retained nearly the entirety of its original stiffness, as the fractured brace re-engaged onto the gusset plate (Figure 6b), providing a mechanism for transferring load in compression. As the brace continued to bear on the gusset plate, the slot length grew from the gusset cutting through the brace. As the frame was pushed further and the gusset plate cut further through the brace, the bearing surface grew due to flattening of the deformed steel (Figure 7a), further increasing the capacity of the fractured brace in compression. Eventually, at large negative drifts (P09s), the gusset cut through the brace by such an amount that the brace began to bear on the flange of the beam, even further increasing the bearing surface. When Story 1 was pushed to a total of 3% positive drift (P10n), the brace slipped off the gusset plate and deflected out of plane, preventing any further studies on brace re-engagement. The frame was then pushed North the maximum amount possible, to δ_1 = 6%, ending the test. During this final cycle (P11n), significant column bending was observed, resulting in yielding at the base of the Story 1 South column (Figure 7b).



(a) (b)

Figure 6. Story 1 North brace connection following weld fracture: (a) brace detachment; (b) brace re-engagement.

6 SUMMARY AND CONCLUSIONS

Full-scale testing of a two-story *R*=3 chevron concentrically-braced frame (CBF) has provided valuable new data on the cyclic behaviour of braced frames not specifically detailed for seismic resistance. Common to moderate seismic regions and popular because of their economy, *R*=3 frames are more susceptible to experience brittle limit states due to lack of seismic detailing. During this test, both Story 2 braces experienced a brittle buckling mechanism with significant loss of strength. This is in contrast to the other full-scale test performed by the authors on an *R*=3.25 ordinary concentrically-braced frame (OCBF), which experienced more ductile brace buckling behavior as expected due to its modest seismic detailing requirements [4]. Despite the relatively brittle brace buckling in the *R*=3 test described here, the frame still maintained a capacity of 445kN [100 kip] to Story 2 drifts upwards of 2%, identifying the reserve capacity achievable from the connections, frame action and the buckled braces. The maximum base shear achieved under a loading protocol mimicking the equivalent lateral force distribution used in design was 2060 kN [464 kip], nearly 40% more than the design base shear of 1500 kN [336 kip].

Hysteretic behaviour of Story 1 following an induced brace-to-gusset weld fracture revealed that the story regained nearly all of its original stiffness when the brace was bearing on the gusset plate (Figure 4). The bearing capacity was not simply capped by the original surface area of the HSS thickness acting over the gusset plate, but rather increased substantially throughout the test as the effective bearing surface of the brace increased due to local deformation (Figure 7a) and contact with the gusset plate and the beam flange (Figure 6b). Another interesting observation from this test was the durability and longevity of the EBF-like behaviour following weld fracture. This mechanism withstood story drifts of up to 6% in this test, implying that there is significant reserve capacity through this combination of beam and column flexure.

(a) (b)

Figure 7. Post-test observations: (a) HSS bearing on the gusset plate; (b) column hinging from large story drift.

Overall, the R=3 frame performed reasonably well, achieving a capacity of nearly 1.4 times the design base shear and displaying appreciable reserve capacity. In the case of brace buckling, the frame exhibited reserve strength primarily from the post-buckling brace behavior. In the case of weld fracture, however, the frame exhibited reserve strength that varied based on the direction of load. In the positive direction, the frame exhibited reserve strength from friction between the fractured brace and gusset plate, beam flexural hinging, and column bending. In the negative direction, the frame exhibited reserve strength predominantly from the fractured brace re-engaging onto the gusset plate and bearing in compression. The combination of this EBF-like behaviour with brace re-engagement created a promising source of reserve capacity which seemed to outperform brace buckling on its own. However, insuring the formation of such a mechanism can be difficult, as shown in this experiment, and further studies are required before it can be suggested as a design strategy.

7 ACKNOWLEDGEMENTS

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