NUMERICAL AND EXPERIMENTAL ASSESSMENT OF CHEVRON BRACED FRAMES WITH WEAK BEAMS


ABSTRACT

Steel concentrically braced frames have been commonly employed as lateral force resisting systems in the United States for many decades. A wealth of past research has been dedicated to the improved seismic performance of new special concentrically braced frames (SCBFs), which are designed using capacity design principles. However, much of the existing braced frame infrastructure in seismic regions was designed prior to the codification of such principles. These older frames are classified as non-seismic concentrically braced frames (NCBFs) and many likely possess undesirable failure mechanisms and limited ductility. In particular, chevron braced frames are subjected to large flexural beam demands caused by post-buckling unbalanced brace forces. NCBFs in such configurations largely contain beams that cannot develop these unbalanced forces elastically, causing significant beam deflections that may accelerate brace buckling. A series of two-story tests of full-scale chevron NCBFs with weak beams was conducted at the National Center for Research on Earthquake Engineering laboratory in Taiwan and results show that the presence of a weak beam was not detrimental to system level performance. Detailed finite element analyses confirm this behavior and will lead towards developing a solid framework for modeling NCBFs.

1 Graduate Student, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, adsen@uw.edu
2 Doctoral Candidate, Dept. of Structural Engineering, Tongji University, Shanghai, China, livia0408@gmail.com
3 Graduate Student, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, dansloat@uw.edu
4 Professor, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, croeder@uw.edu
5 Assoc. Professor, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, delehamn@uw.edu
6 Assoc. Professor, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, jwberman@uw.edu
7 Professor, Dept. of Civil Engineering, National Taiwan University, Taipei, Taiwan, kctsai@ntu.edu.tw
8 Asst. Researcher, National Center for Research on Earthquake Engr., Taipei, Taiwan, chli@ncree.narl.org.tw
9 Asst. Researcher, National Center for Research on Earthquake Engr., Taipei, Taiwan, acwu@ncree.narl.org.tw

Numerical and Experimental Assessment of Chevron Braced Frames with Weak Beams

A. D. Sen¹, L. Pan², D. Sloat³, C. W. Roeder⁴, D. E. Lehman⁵, J. W. Berman⁶, K. C. Tsai⁷, C. H. Li⁸, and A. C. Wu⁹

ABSTRACT

Steel concentrically braced frames have been commonly employed as lateral force resisting systems in the United States for many decades. A wealth of past research has been dedicated to the improved seismic performance of new special concentrically braced frames (SCBFs), which are designed using capacity design principles. However, much of the existing braced frame infrastructure in seismic regions was designed prior to the codification of such principles. These older frames are classified as non-seismic concentrically braced frames (NCBFs) and many likely possess undesirable failure mechanisms and limited ductility. In particular, chevron braced frames are subjected to large flexural beam demands caused by post-buckling unbalanced brace forces. NCBFs in such configurations largely contain beams that cannot develop these unbalanced forces elastically, causing significant beam deflections that may accelerate brace buckling. A series of two-story tests of full-scale chevron NCBFs with weak beams was conducted at the National Center for Research on Earthquake Engineering laboratory in Taiwan and results show that the presence of a weak beam was not detrimental to system level performance. Detailed finite element analyses confirm this behavior and will lead towards developing a solid framework for modeling NCBFs.

Introduction

Special concentrically braced frames (SCBFs) represent the current standard for concentrically braced frames in seismic applications in the United States and follow the capacity design principles set forth by the American Institute of Steel Construction (AISC) Seismic Provisions for Structural Steel Buildings, AISC 341 [1]. In an SCBF, the braces are designed to buckle in compression and yield in tension and the surrounding frame is designed to permit this inelastic behavior while the framing elements remain essentially elastic. Braces are typically

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¹ Graduate Student, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, adsen@uw.edu
² Doctoral Candidate, Dept. of Structural Engineering, Tongji University, Shanghai, China, livia0408@gmail.com
³ Graduate Student, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, dansloat@uw.edu
⁴ Professor, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, croeder@uw.edu
⁵ Assoc. Professor, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, delehamn@uw.edu
⁶ Assoc. Professor, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195, jwberman@uw.edu
⁷ Professor, Dept. of Civil Engineering, National Taiwan University, Taipei, Taiwan, kctsai@ntu.edu.tw
⁸ Asst. Researcher, National Center for Research on Earthquake Engr., Taipei, Taiwan, chli@ncree.narl.org.tw
⁹ Asst. Researcher, National Center for Research on Earthquake Engr., Taipei, Taiwan, acwu@ncree.narl.org.tw

connected to the frame using gusset plate connections with adequate clearance that is intended to localize rotational demand to the gusset plates. Several advancements in SCBF design have recently been made by Roeder et al. [2] aimed at improving the ductility of braced frames through a balanced design procedure that establishes ductile yielding and failure hierarchies.

Concentrically braced frames in seismic regions that were designed prior to the adoption of the 1988 Uniform Building Code [3] do not possess many of the attributes of SCBFs. These are termed non-seismic concentrically braced frames (NCBFs) and their perceived deficiencies are both diverse and numerous when evaluated with capacity design standards. Critical issues associated with NCBFs, such as non-ductile connection details and non-seismically compact brace sections, were found in an infrastructure review that is described in greater detail in the companion paper by Sloat et al. [4].

The seismic performance of NCBFs in the V and inverted-V, or chevron, configurations is especially concerning. In chevron configurations, two braces are used in each bay that concurrently resist lateral load through axial compression and tension. These braces span from the frame corners to the middle of the beam, where they intersect. If the braces remain elastic, then the connections are potentially the limiting components. SCBF design requires that the connections be sized such that they fully develop the expected brace capacity, but some NCBFs may have undersized connections. If the braces buckle, then unbalanced forces develop at the middle of the beam. These forces arise from the post-buckling strength degradation of the brace in compression and the relatively stable force of the opposing brace in tension. The AISC Seismic Provisions [1] prescribe that the unbalanced load be computed using the expected tensile strength of one brace and 30 percent of the expected compressive strength of the other.

In SCBFs, the beam is sized to develop the unbalanced force, but this was not a design consideration in NCBFs. As a result, many NCBFs with braces in chevron configurations may subject beams to large vertical deflections. In turn, this behavior may increase the amount of brace deflection at a given drift level and accelerate brace local buckling, which is a primary precursor to fracture. In a post-brace fracture frame, strength and stiffness are significantly reduced and collapse prevention relies upon the presence of secondary and often unintended mechanisms. To study the behavior of concentrically braced frames with weak beams, numerical and experimental studies were conducted on three full-scale two-story NCBF specimens shown with braces in the inverted-V configuration.

**Specimen Design**

Three specimens were designed as a part of this study. The first specimen, designated NCBF-INV-1 and shown in Fig. 1, was designed with characteristics representative of actual NCBF systems observed in the aforementioned infrastructure review, with a few practical exceptions. The top floor beam was designed to meet SCBF unbalanced force requirements and included a fully composite slab. The upper second story gusset plate connections were designed with the balanced design procedure. These measures were implemented to prevent damage to the frame, which would be reused for multiple tests in the experimental study.

Non-seismically compact HSS7x7x1/4 braces were selected for both stories, since many
NCBF braces do not meet the current compactness requirements in the AISC Seismic Provisions. At the first story and lower second story gusset plates, a horizontal or vertical clearance of 25mm was provided, which does not satisfy either the AISC linear clearance or balanced design elliptical clearance requirements. The lower second story brace-beam-column connections utilized a continuous shear tab along the gusset plates and beam web, which is uncommon in current design. The inset in Fig. 1 shows a magnified view of this connection. Net section fracture and brace base metal fracture were the only failing limit states in a capacity analysis of all gusset plate connections, excluding the second story top beam gusset plate connections. In addition, AWS E71T-7 weld wire was used at the existing connection locations; this material has no Charpy V-notch toughness requirements, as would have been common in older construction.

Figure 1. NCBF-INV-1 specimen overview with (inset) typical brace-beam-column gusset plate connection (units in milimeters).

The bottom beam, a W16x45 section, was weak in flexure relative to the expected brace unbalanced load demands. The contribution of the slab towards composite action was unknown but presumed to be minimal; the deck and beam were connected via tack welds only in order to simulate older frames without composite beam design. Lateral bracing was present at the third-points of the beam and violated SCBF design, where lateral bracing is required at the brace intersection in most cases.

Both the second and third specimens, NCBF-INV-2 and NCBF-INV-3, represented repairs of the first specimen. NCBF-INV-2, pictured in Fig. 2, utilized seismically compact HSS5x5x3/8 braces on the first story designed for in-plane buckling using knife plate connections with $3.25\ t_{kp}$ as-built clearances, where $t_{kp}$ is the thickness of the knife plate. A typical knife plate connection from NCBF-INV-2 is inset in Fig. 2. In-plane buckling connections may prove useful in NCBF retrofit scenarios where the rotational capacity of the existing gusset plates is limited. Using knife plates to induce in-plane brace buckling, the
rotational demands on the gusset plates may be significantly reduced and potentially poor gusset plate interface connections may be protected. However, for NCBF-INV-2, the first story gusset plates were damaged in the first test and therefore replaced in the repair. The new components were designed using the balanced design procedure and a clearance of 50mm was provided between the end of the knife plates and column or beam flanges to allow for ease of fabrication. It was assumed that this small clearance would provide adequate fixity against out-of-plane rotation.

![Figure 2. NCBF-INV-2 specimen overview with (inset) typical brace-base plate-column knife plate connection (units in millimeters).](image)

The third specimen, NCBF-INV-3, shown in Fig. 3, represented a repair using wide flange H175x175x7.5x11 (metric) braces designed for out-of-plane buckling on the first story. As in NCBF-INV-2, the connections were designed using the balanced design procedure. Following this procedure, the corner gusset plates were designed with an $8t_p$ elliptical clearance and the middle gusset plates were designed with a $6t_p$ vertical clearance, where $t_p$ is the thickness of the gusset plate. Two and three story SCBF tests previously conducted at NCREE by Clark [5] and Lumpkin [6], respectively, showed that wide flange braces can achieve greater ductility than their HSS counterparts. Thus, the wide flange braces were expected to place large flexural demands on the weak beam.

The experimental specimens were constructed and tested at the National Center for Research on Earthquake Engineering (NCREE) laboratory in Taiwan. High fidelity finite element models of each specimen were developed in Abaqus [7] using shell elements for the steel members and solid elements for the concrete slabs. The steel material properties used in the models were derived from coupon tests of each component used in the experiments. Brace buckling was seeded using initial out-of-plane geometrical imperfections in the braces not exceeding $L/500$, where $L$ is the brace length. In both the experimental and numerical cases, the
frame was subjected to the same increasing amplitude cyclic loading protocol at the top slab using a displacement controlled procedure.

Figure 3. NCBF-INV-3 specimen overview with (inset) typical brace-base plate-column gusset plate connection (units in millimeters).

**Experimental Results**

The experimental behavior and failure mode of each of the three specimens was highly varied, as the roof drift-base shear hystereses in Fig. 7 indicate. The plots also show that numerical models have been able to accurately simulate the global behavior of the specimens.

**NCBF-INV-1: Existing Building**

The overall response of NCBF-INV-1 was highly non-ductile. The first story south brace was the only brace to buckle, which occurred at -0.48 percent first story drift. Local buckling near the midpoint of the brace coincided with brace buckling and is shown in Fig. 4(a). In the tension half-cycle immediately following buckling, tearing of the brace was observed at the locally buckled corners of the brace. Brace fracture initiated one cycle later at +0.49 percent story drift, seen in Fig. 4(b). This is a relatively low drift level to correlate with brace fracture, but there is little evidence suggesting that the weak beam was responsible for this behavior. Instead, this highly non-ductile behavior can be associated with the compactness of the brace; non-compact braces experience severe local buckling, which can lead to earlier brace fracture according to Goel [8]. Such behavior was confirmed in this test.

Following brace fracture, the low flexural capacity of the beam relative to the brace axial capacity heavily influenced behavior. This weak beam-strong brace condition prevented buckling of the remaining first story brace. Thus, the bottom beam south of the midpoint became
a large flexural link in an eccentrically braced frame. The brace itself did not fracture, but partial fracture of the brace-beam connection occurred later in the test. Failure initiated from cracking in the brace net section, followed by brace base metal and splice weld fracture on the west side of the connection, as is seen in Fig. 4(c). This result is consistent with the capacity analysis conducted prior to testing. It should be noted that throughout the test, the second story braces remained essentially elastic, though some net section cracks formed at the unreinforced lower brace ends.

Figure 4. NCBF-INV-1. (a) Initial local buckling of first story south brace, (b) brace fracture of first story south brace, and (c) partial fracture of first story north brace-beam gusset plate connection.

Figure 5. NCBF-INV-2. (a) Peak first story north brace out-of-plane displacement, (b) bottom (weak) beam flange yielding and gusset plate elliptical yielding from out-of-plane rotation, and (c) partial fracture of first story north brace.
NCBF-INV-2: In-Plane Buckling Repair

Although the second specimen was designed such that the first story braces would buckle in-plane, out-of-plane buckling of the south and north first story braces occurred at -0.28 and +0.43 percent story drift, respectively. Despite this unintended mechanism, the frame was able to achieve considerable ductility. Fig. 5 illustrates the peak out-of-plane displacement of the north first story brace and beam yielding caused by large vertical deflections. Local buckling in both braces occurred at just over 1.74 percent story drift in either direction, and the braces partially fractured around 2.40 percent story drift in either direction. This level of story drift at brace fracture is comparable to SCBFs with tubular braces tested by Clark [5] and Lumpkin [6].

Much of the observed deformation capacity of the system can likely be attributed to the weak beam. Torsional rotation of the beam is evident through the analysis of both visual and measured data. This torsion, in addition to ductile tearing of the gusset plate interface welds, helped accommodate out-of-plane brace end rotations in the test without brittle connection failure. On the other hand, the weak beam likely influenced the prevalence of the unintended out-of-plane buckling mode, along with the lack of lateral support at the brace intersection and the gusset plate flexural stiffness relative to the knife plate. These factors work concurrently to decrease the fixity of the out-of-plane rotational restraint, increasing the effective length of the brace. A minor amount of inelastic behavior in the second story developed near the end of the test, which is qualitatively confirmed through increased net section cracking and slab spalling.

Figure 6. NCBF-INV-3. (a) Wide flange brace out-of-plane deflection, (b) second story south brace connection fracture, and (c) bottom beam beam-to-column connection partial fracture.

NCBF-INV-3: Wide Flange Brace Repair

The third test was characterized by significant nonlinear behavior at the second story, where the brace-beam-column gusset plate connections failed. This occurred at -0.53 percent drift on the north end and +0.86 percent drift on the south end. The failure of the south connection can be seen in Fig. 6(b). As has been previously documented, these braces accumulated a moderate amount of net section cracking in the previous two tests. The net
sections of these braces were unreinforced in NCBF-INV-1, where only minor cracking was observed. They were retrofitted with net section reinforcement plates in NCBF-INV-2 in an attempt to impede the development of further damage, but these efforts did not prevent connection failure. As in the connection failure in NCBF-INV-1, the net section cracks propagated towards the brace end, and ultimate failure was therefore a combination of net section, brace base metal, and splice weld rupture.

At the first story, the wide flange braces behaved as anticipated, as they buckled and placed significant flexural demands on the weak beam. However, the weak beam’s deformation capacity proved to be limited by the beam-to-column shear tab connection pictured in Fig. 1. The connection was repaired following both NCBF-INV-2 and NCBF-INV-3 due to cracking along the shear tab-to-beam web weld. In the third test, the weld cracked severely and the upper beam-shear tab erection bolts failed in shear as shown in Fig. 6(c). This is an undesirable brittle failure mechanism, and many older connections may possess rotationally limited connection details.

Numerical Validation

Three non-linear finite element analyses were performed in Abaqus to attempt to numerically simulate the experimental behavior of the specimens. As it can be seen from Fig. 7, the global responses of the experiments and analyses match reasonably well. To achieve this level of similarity, some test events, such as brace and connection fracture, were manually initiated in the model. In the instance of fracture, certain elements of the model were deactivated at the corresponding drift levels in the test. However, in reality, failure progresses along a damage continuum that is difficult to capture in numerical modeling. This is particularly evident when comparing the responses of NCBF-INV-3, where connection failure was gradual in the experiment but instantaneous in the model.

![Figure 7. Roof drift-base shear hystereses for (a) NCBF-INV-1, (b) NCBF-INV-2, and (c) NCBF-INV-3.](image)

Conclusions

These experiments represent some of the first performed on older braced frames in the inverted-V configuration with weak beams, and detailed numerical modeling has supported
many of the findings from these tests. NCBF-INV-1 demonstrated that NCBFs have the potential to behave in highly non-ductile manners. However, this poor performance is mostly attributed to the use of non-seismically compact braces, which undergo severe local buckling that is nearly coincident with brace buckling. NCBF-INV-2 has provided data suggesting that chevron braced frames with weak beams may be able to achieve relatively high ductility levels in the presence of seismically compact braces. Further, the low torsional stiffness of the beam may enhance performance by alleviating rotational demands on the gusset plates. Despite these apparent advantages, NCBF-INV-3 proved the importance of the in-plane rotational capacity of the beam-to-column connections. Connections with low rotational capacities may result in a brittle failure mechanism that precedes brace fracture and beam plastic hinging.

The numerical investigation has resulted in the development of NCBF models that successfully capture much of the observed experimental behavior. As a result, NCBF models using the same characteristics may be used to predict the behavior of future experimental tests. The long-term goal of determining modeling parameters to apply to the evaluation of existing infrastructure has not yet been accomplished by this research. Further numerical and experimental studies will be important in developing models to evaluate the need for retrofit of the many NCBFs still in service today.

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