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SEISMIC VULNERABILITY AND REHABILITATION OF OLDER CONCENTRICALLY BRACED FRAMES

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Abstract

Concentrically braced frames (CBFs) are one of the most common lateral-force-resisting systems in steel construction. In the US, they have been used to resist earthquake loading extensively but many were built prior to 1988, before the codification of capacity-based design and other provisions which ensure ductility. These older CBFs are considered nonductile CBFs (NCBFs), and an infrastructure review of 12 NCBFs from the US was conducted to quantify the frequency and severity of global and local deficiencies relative to special CBF (SCBF) requirements, but the consequences of these deficiencies are unclear. An experimental research program was devised to advance the understanding of NCBF brace and connection deficiencies and potential retrofit schemes. This paper presents a case study of three (3) of the 18 single-diagonal-brace NCBF specimens tested; the case study specimens all had double-angle gusset-plate connections. The results demonstrate the poor behavior of a typical NCBF and two (2) partial retrofit schemes which enhanced the deformation capacity of the system. The exceptional performance of the retrofits highlights the importance of designing schemes which do not punish yield mechanisms while mitigating vulnerable failure modes.

Keywords: steel; braced frames; connections; nonductile; rehabilitation

1. Introduction

Steel concentrically braced frames (CBFs) are strong, stiff lateral force resisting systems. They have been used in regions with high seismic risk to resist earthquake-induced loading for many decades. Today, they are designed to special CBF (SCBF) standards defined by the AISC *Seismic Provisions* [1]. These ensure damage is concentrated in the braces and their connections and that these components can sustain large, inelastic cyclic deformations without significant strength degradation. However, this capacity-based, ductile design philosophy is relatively recent in the history of steel construction, and many CBFs on the West Coast of the US and other seismically active regions of the world do not meet requirements for new construction.

Current SCBF provisions are rooted in the capacity-based design principle introduced in the 1988 *Uniform Building Code* (UBC) [2] and have evolved significantly to address component- and system-level concerns. CBFs designed prior to the 1988 UBC, termed nonductile CBFs (NCBFs) due to their expected behavior, do not meet many of the following requirements:

- Design of the connections, beams, and columns for the expected brace capacities, including consideration of the post-buckling unbalanced load in chevron configurations;
- Configuration of the braces to balance tension and compression loading while avoiding undesirable plastic mechanisms;
- Limitation of the brace cross-sectional slenderness (width-to-thickness ratio) to delay plastic hinging and fracture of the brace;
- Accommodation in the connection for brace-end rotation due to buckling; and
- Use of demand critical welds for yielding elements.

Damage to NCBFs in prior earthquakes in the US and Japan has been well documented [3, 4, 5, 6] and can be traced to noncompliance with the above SCBF requirements.

Many questions remain about the seismic performance of NCBFs, and this has motivated an ongoing research program on NCBFs. The project experimentally evaluated the behavior of existing and retrofitted subsystems based on an inventory of US NCBFs. This paper presents the results of three specimens tested at the University of Washington which focus on the existing and retrofitted performance of NCBFs with double-angle gusset/beam-to-column connections.

2. Infrastructure Review

In seismic design, the yield and failure hierarchy of the lateral force resisting system is critical. To achieve good seismic performance, the yield mechanisms should be encouraged while the failure modes are suppressed. In SCBFs, the brace is sized for the reduced seismic loads and brace yielding and buckling are the primary yield mechanisms. Brace fracture eventually occurs due to low-cycle fatigue and is the primary failure mode. These actions are encouraged by sizing the connections, beams, and columns for the brace capacities and adhering to geometric limits which prevent or delay failure modes.

In NCBFs, connections, beams, and columns were sized for the same seismic loads as the braces, and therefore failure modes were not necessarily suppressed. For example, a weld connecting a brace to the gusset plate could resist the computed seismic load effects but not the yield force of the brace; hence, the weld would fracture before the brace yields. Using the same reasoning, NCBFs may also have additional yield mechanisms. Recent research on SCBFs suggests that an extended yield hierarchy can improve inelastic deformation capacity [7]. The inclusion of gusset-plate yielding as a secondary yield mechanism markedly improved the drift capacity, and this is a central premise of the balanced design procedure (BDP) [8]. Thus, the extended yield hierarchies present in some NCBFs may be beneficial for existing seismic performance or harnessed in retrofit.

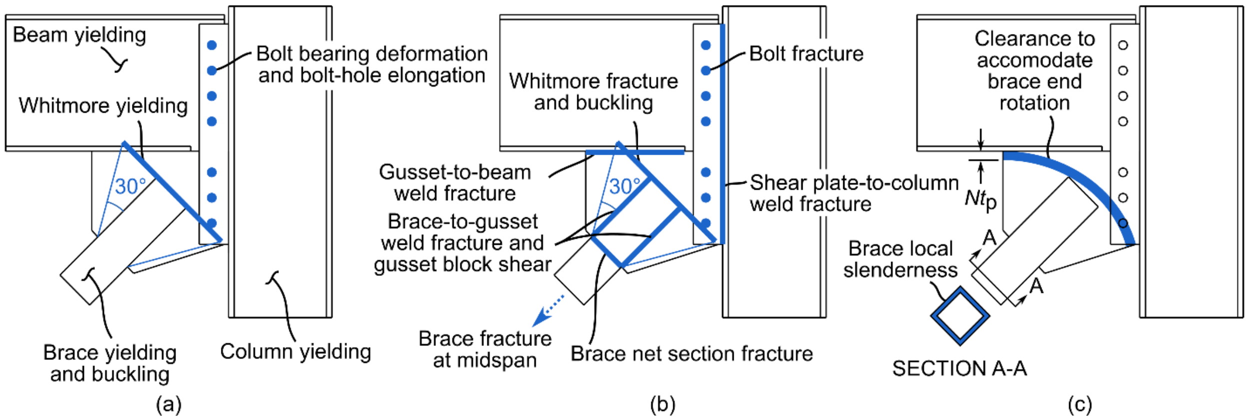


Fig. 1 – Concentrically braced frame (a) yielding mechanisms, (b) failure modes, and (c) geometric limits

A review of 12 NCBFs up to 9 stories in height was conducted to understand the deficiencies present in the existing building stock. The review showed that chevron and single-diagonal bracing configurations were most common. System deficiencies are not investigated in this paper, but deficient beams in chevron configurations were the focus of a companion experimental program [9, 10]. The braces were most often rectangular hollow structural sections (HSS). Gusset plates were typically used to connect braces to the frame but, as Fig. 2 shows, there is considerable variation in gusset plate configuration.

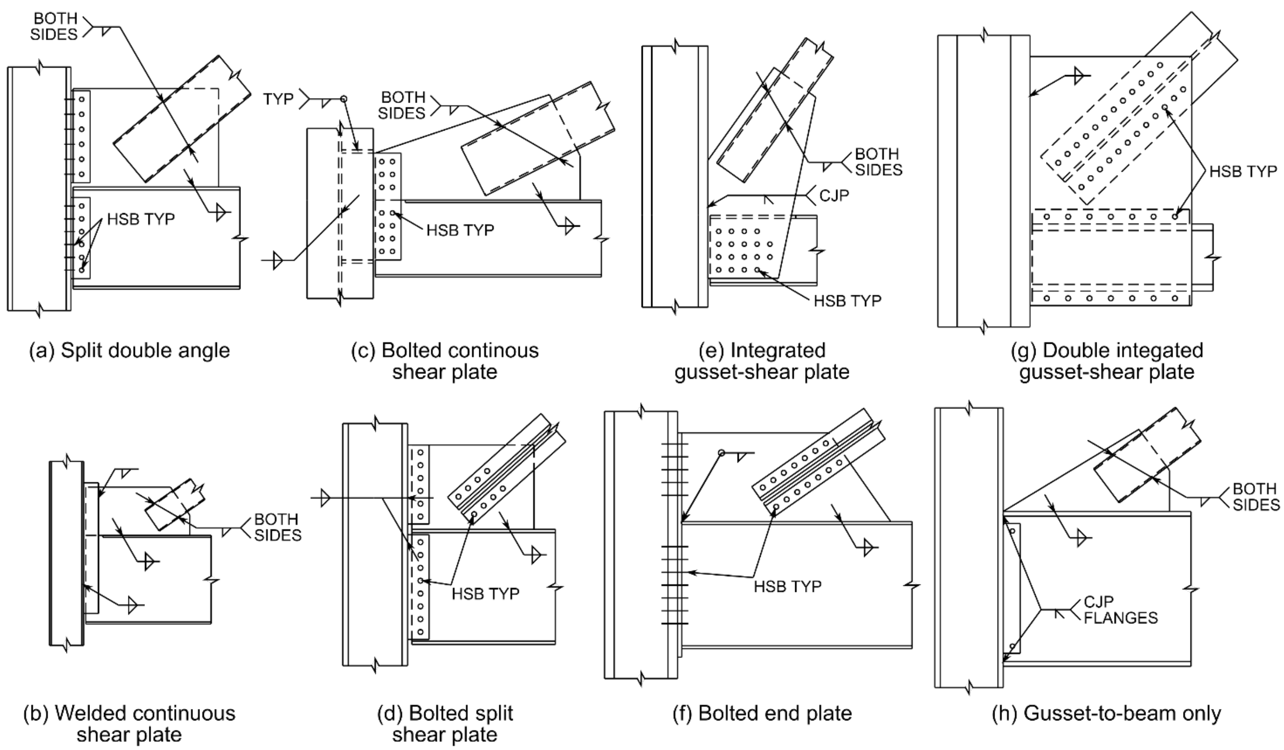


Fig. 2 – NCBF connection configurations

The connections were evaluated using SCBF requirements [1, 11] to identify potential vulnerabilities. For welded gusset-plate-to-frame connections, both the uniform force method (UFM) [12] and BDP [8] design expressions were considered. In the UFM, the welds are designed for the brace capacity in tension under an assumed distribution between the vertical and horizontal interfaces. In the BDP, the welds are designed for the gusset-plate capacity since plate yielding is expected in tension as part of a secondary yield mechanism and due to brace-end rotation.

The median of the maximum demand-to-capacity ratios (DCRs) for each building in the infrastructure review for important yield mechanisms, failure modes, and geometric limits are provided in Table 1. The DCRs do not consider resistance factors and are based on the expected material properties (i.e., applying R_y and R_t values provided in the *Seismic Provisions* [1]) because this was anticipated to more accurately assess vulnerability. The approximate DCR required by design (accounting for these factors) is shown for comparison. Table 1 shows that many buildings have:

- Potential gusset-plate yielding based on the Whitmore section;
- Vulnerable gusset-to-beam or -vertical-element welds, especially based on the BDP;
- Excessive bolt-fracture DCRs for bolted gusset-plate connections;
- High brace-to-gusset weld fracture DCRs; and
- Locally slender HSS braces.

While it is clear that NCBFs often do not meet SCBF requirements, especially considering the approximate design limit DCRs, the importance of their deficiencies is not well understood. This knowledge is critical to determining retrofit strategy and is a major focus of the research described here.

Table 1 – Demand-to-capacity ratios for infrastructure review buildings and test specimens

Classification	**Criterion**	**Approx. Design Limit**	**NCBF Median**	**Specimen**		
				1	**2**	**3**
Yield mechanisms	Gusset-plate Whitmore yielding	0.7	1.1	1.0	0.8	0.8
	Gusset-to-vertical-element bolt bearing	0.6	0.5	–	0.8	0.8
	Beam-to-vertical-element bolt bearing	0.6	0.8	–	1.0	1.0
Failure modes	Gusset-to-vertical-element weld fracture[a]	0.6 (0.8)	0.9 (1.7)	0.9 (1.3)	–	–
	Gusset-to-beam weld fracture[a]	0.6 (0.8)	0.9 (1.3)	0.6 (1.3)	0.5 (1.0)	0.3 (0.7)
	Gusset-to-vertical-element bolt fracture	0.8	1.3	–	0.6	0.6
	Beam-to-vertical-element bolt fracture	0.8	1.3	–	0.5	0.5
	Brace-to-gusset weld fracture	0.8	1.1	1.7	0.7	0.7
	Gusset block shear	0.6	0.7	0.9	0.7	0.7
Geometric limits	HSS brace local slenderness[b]	1.0	1.8	1.9	0.9	0.9

Note: A vertical element is a shear plate, end plate, angle, or similar element which connects to the column.

[a]UFM DCR (BDP DCR)

[b]DCR is λ/λ_{hd}, where λ is the width-to-thickness ratio (b/t) and λ_{hd} is the limit for highly ductile members in the *Seismic Provisions* [1]

3. Experimental Program

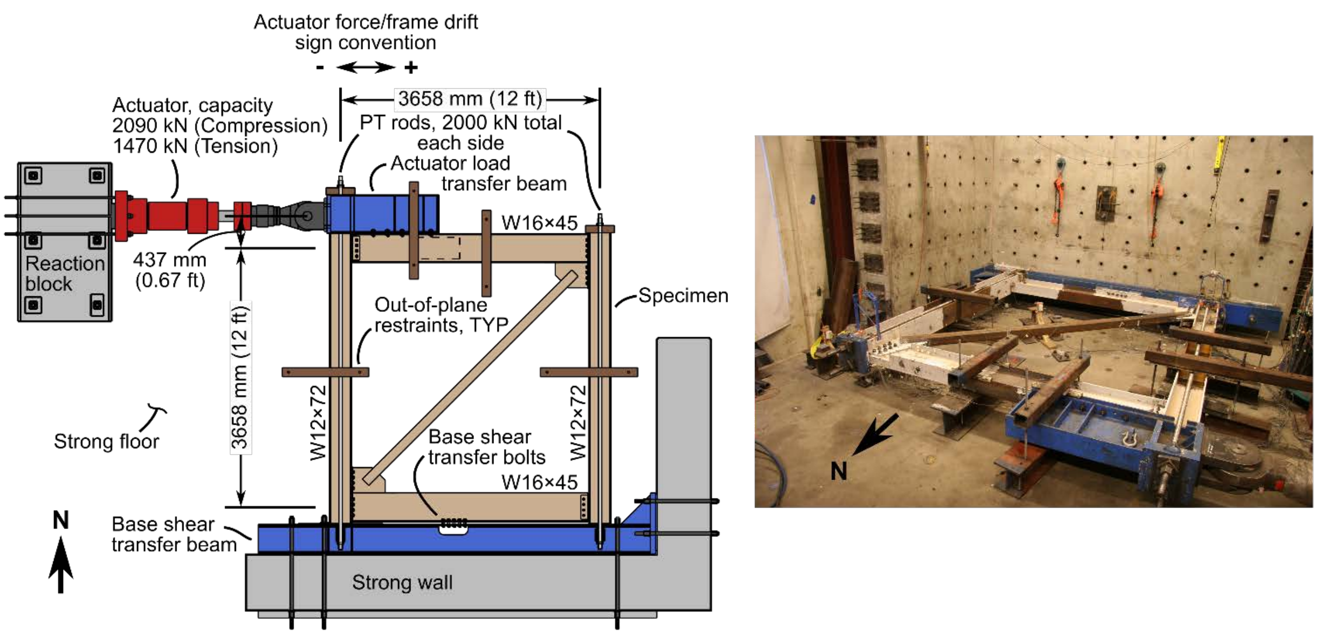
An experimental program was conducted at the University of Washington's Structural Research Laboratory to determine existing and retrofitted performance of NCBFs [13]. Each specimen consisted of a single diagonal brace within a square frame with W16×45 beams and W12×72 columns as shown in Fig. 3. They were subjected to quasistatic loading using a fully reversed, increasing-amplitude cyclic loading protocol based on ATC-24 [14]. Eighteen (18) specimens were tested, and these utilized gusset-plate connections resembling those shown in

Fig. 2a through 2f [15, 16, 17, 18, 19]. The specimens had DCRs which simulated salient limit states and geometric limits from the infrastructure review. The retrofit strategies tested included:

- Brace replacement with more compact out-of-plane or in-plane buckling HSS braces;
- Brace replacement with a buckling-restrained brace;
- Brace rehabilitation through concrete in-fill of a locally slender HSS brace;
- Connection reinforcement with bolts on previously weld-only connections; and
- Connection reinforcement with demand critical weld overlay on BDP-deficient, non-demand critical fillet welds.

This paper focuses on the testing of the specimens with split double-angle connections similar to that in Fig. 4a. The DCRs for these specimens are shown in Table 1. Specimen 1 represented an NCBF with a locally slender HSS6×6×1/4 brace (near the median DCR from the infrastructure review) and deficient gusset-plate welds for both the brace and frame connections [15]. Specimen 2 was a brace replacement retrofit of a similar connection but with bolted gusset-to-angle connections [19]. The new brace was an HSS5×5×3/8. Both the brace local slenderness and brace-to-gusset weld deficiencies were addressed, but the gusset-to-beam welds were still deficient based on the BDP. Specimen 3 was nominally identical to Specimen 2 but had an additional retrofit: demand critical weld filler metal was placed over the existing non-demand critical weld to comply with the BDP criteria including a 0.75 resistance factor (i.e., satisfying the approximate design limit DCR shown in Table 1) [19].

The angles were relatively thin and prying of the bolts was expected in the specimens. With respect to the angle thickness required to eliminate prying action, these angles had DCRs between 2 and 3. Although prying increases demands on the bolts, the associated angle deformation is viewed as a yielding mechanism that may enhance drift capacity.



(a) Plan view

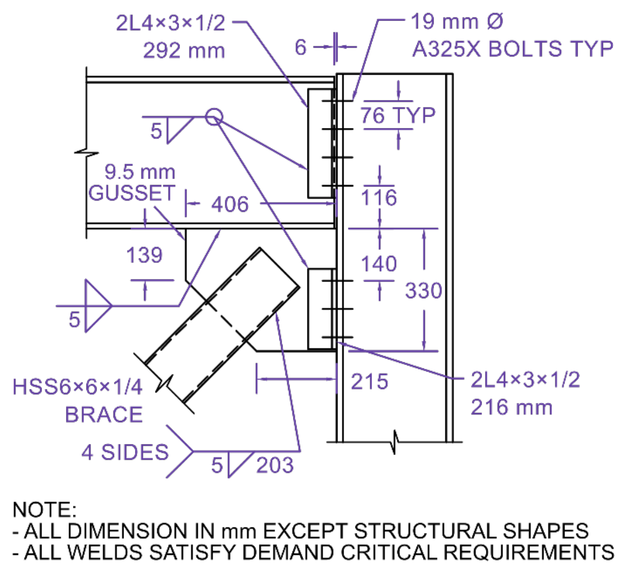
(b) Specimen photograph

Fig. 3 – Experimental setup

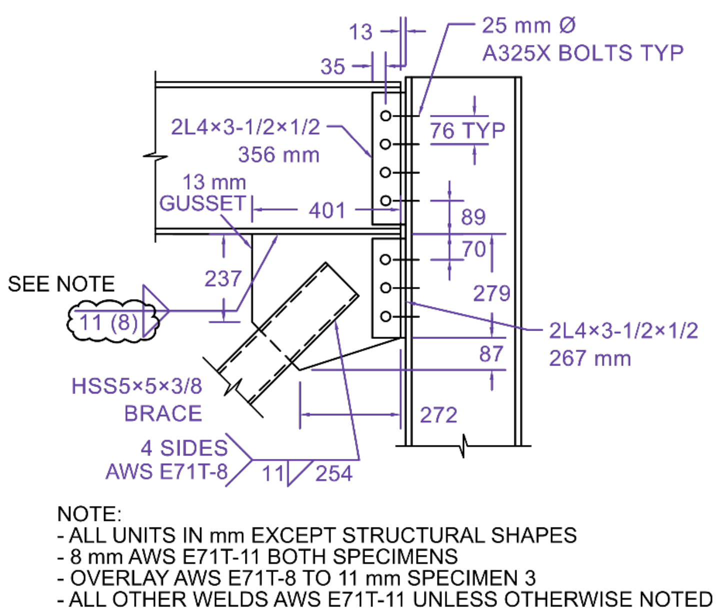
4. Experimental Results

Figures 4c, 4d, and 4e show the hysteretic response of the three double-angle-connection specimens. In these plots, story shear is normalized by the lateral component of the brace yield force. After the brace fractures or disconnects from the frame due to connection fracture, frame action provides significant lateral resistance; this is not shown in Fig. 4. The following discussion uses drift range, the total drift achieved in brace compression and tension, as the primary metric for lateral deformation because loading in both directions contributes to brace fracture [20].

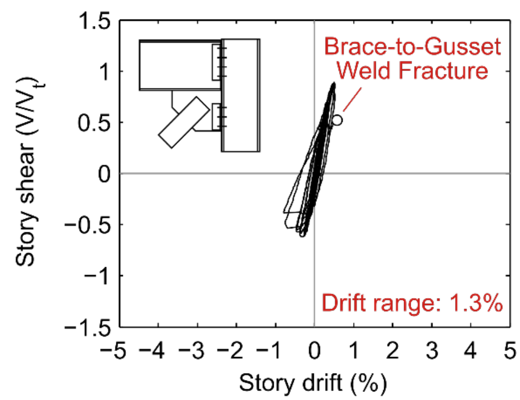
Specimen 1 exhibited nonductile behavior that is expected for an NCBF [13], sustaining brace-to-gusset weld fracture at only a 1.3% drift range (see Fig. 5b). This failure mode is well predicted by its DCR of 1.7. Although the brace was locally slender, the deficient weld dominated the response. Brace yielding was precluded by brace-to-gusset weld fracture, but the brace buckled out-of-plane and began to form a plastic hinge at its midspan prior to weld fracture, as shown in Fig. 5a. This accelerated plastic hinging is characteristic of locally slender HSS braces [21] and is a significant concern if the connections have less severe deficiencies [13].



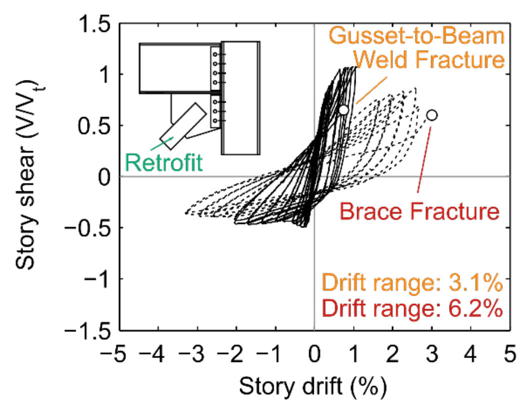
(a) Drawing of Specimen 1



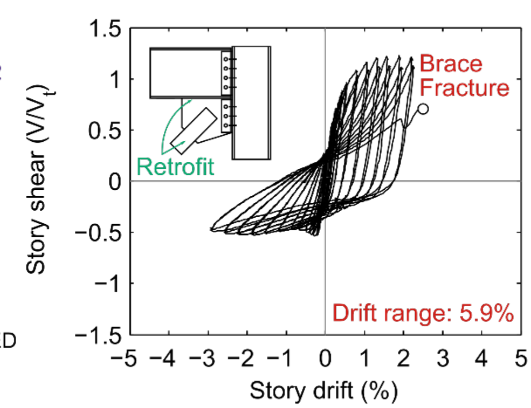
(b) Drawing of Specimens 2 and 3



(c) Specimen 1 hysteresis



(d) Specimen 2 hysteresis



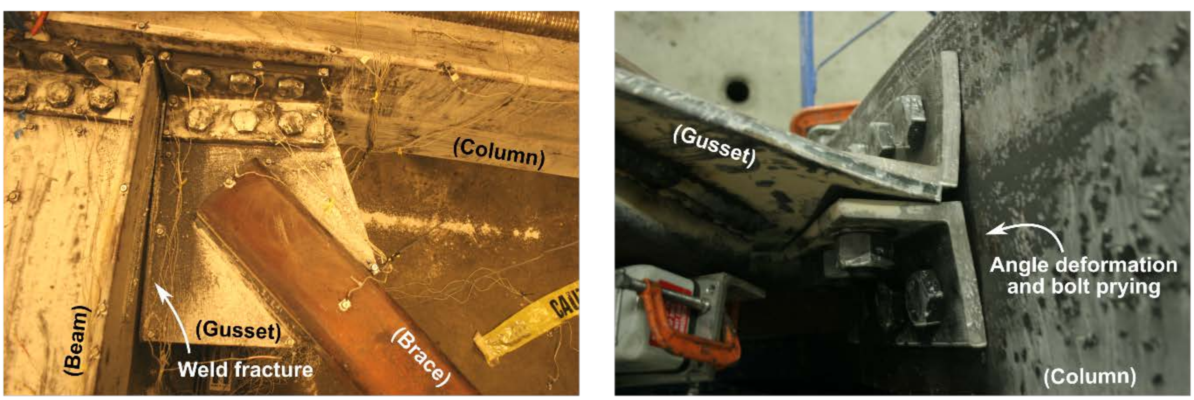
(e) Specimen 3 hysteresis

Fig. 4 – Specimen drawings and hysteretic response

(a) (b)

Fig. 5 – Specimen 1 (a) brace midspan and (b) brace end at 1.3% drift range

Specimen 2 was a brace-replacement retrofit and therefore both brace local slenderness and brace-to-gusset weld fracture deficiencies were mitigated. The frame was able to achieve a drift range of 3.1% prior to fracture of the gusset-to-beam weld (see Fig. 6a). The weld had a DCR of 1.0 based on the BDP but had sufficient resistance based on the UFM, even considering design factors. This event is viewed as a local failure mode because the gusset plate maintained a connection to the frame through the bolted double-angle connection, as shown in Fig. 6a. The brace force was therefore limited to the capacity of the angles, which developed significant prying action as shown in Fig. 6b. However, the angles had significant deformation capacity. Although the brace deformation in tension was limited, it deflected further out-of-plane in compression and eventually formed a plastic hinge at its midspan. The brace fractured at a drift range of 6.2% due to deterioration at this region.



(a) (b)

Fig. 6 – Specimen 2 (a) gusset-plate connection at 3.1% drift range and (b) angle deformation at 6.2% drift range

Specimen 3 eliminated the gusset-to-beam weld deficiency using weld overlay and achieved brace fracture at a 5.9% drift range. The gusset-to-beam weld was designed for the BDP (DCR of 0.7) and only developed minor cracks at the weld toes opposite the column faces which did not propagate. The deformation capacity of the frame was higher than BDP-designed SCBFs, which achieve brace fracture at drift ranges around 5% [8]. This result can be attributed to significant prying deformations in the angles, which was similar to that observed in Specimen 2 (see Fig. 6b).

5. Yield and Failure Hierarchy

The double-angle test specimens exhibited different hysteretic behaviors due to their respective yield and failure hierarchies. Ideally, the yield and failure hierarchy can be directly established by comparing the DCRs. Yield mechanisms with DCRs at or slightly above 1.0 are beneficial because they improve drift capacity. Failure modes with DCRs at or above 1.0 are critical because they are expected to occur before brace yielding, resulting in lateral strength degradation. Geometric limits delay failure modes, so geometric limits with DCRs above 1.0 are also critical to the failure hierarchy [13].

Specimen 1 had a Whitmore yielding DCR of 1.0 but the brace-to-gusset weld fractured before the brace yielded, so this had no effect on response. Specimens 2 and 3 had beam-to-angle bolt bearing DCRs of 1.0 but the observed bearing deformations in the beam were minimal. This may be attributed to large deformations exhibited by the angles and conservatism of the bolt bearing resistance expressions. The prying action DCR was not computed based on the brace capacity, but it suggests that the associated angle-deformation yielding mechanism is possible. Allowing prying of the angle bolts enhanced the deformation capacity of Specimen 3, which performed better than may be expected of an SCBF [8].

The results show that in retrofit, mitigation of failure modes is important but some failure modes are more important than others. Specimen 3 had no failure mode DCRs above 1.0 and thus achieved delayed brace fracture as intended. Specimens 1 and 2 had failure mode DCRs at or above 1.0, and connection fractures occurred before brace fracture. In Specimen 1, brace-to-gusset weld fracture controlled the response and was especially critical because the brace was disconnected from the frame. Gusset-to-beam weld fracture was the first failure mode in Specimen 2, but this is considered a local failure mode because the gusset-to-angle bolts maintained a load path between the brace and frame. In retrofit, failure modes with the potential to disconnect the brace should be prioritized. It is also noted that the UFM DCR for gusset-to-beam weld fracture did not suggest this vulnerability in Specimen 2 (DCR of 0.5), and therefore the BDP is recommended as a more robust method for evaluating welded gusset-to-frame connections.

6. Conclusions

An extensive research program is underway which studies the seismic vulnerability and retrofit of NCBFs. Based on an infrastructure review of NCBFs in the US, these systems have widespread deficiencies because unwanted failure modes are not suppressed using capacity-based design. Further, NCBFs do not meet geometric limits that ensure ductility, and their welds do not meet minimum toughness requirements for demand critical welds. Experimental work has been conducted to understand the consequences of brace and connection deficiencies at the subassemblage level and how they may be mitigated in retrofit.

This paper presents the results of three (3) double-angle-connection specimens as a case study. Behavior of the existing system (Specimen 1) was poor and limited to a drift range capacity of 1.3% when the primary failure mode occurred. The retrofits improved this deformation capacity to about a 6% drift range, but Specimen 2 sustained an intermediate, local failure mode prior to brace fracture. The tests support the use of the BDP for evaluating gusset-to-frame welds and designing their retrofits and demonstrate the benefit of harnessing a secondary yield mechanism (deformation associated with prying) in retrofit. However, the influence of these different local response types on system seismic performance remains uncertain. An ongoing analytical study links experimentally calibrated local behavior to system performance, because understanding these effects will inform new retrofit guidelines for NCBFs that will aid practicing engineers.

7. Acknowledgements

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