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EXPERIMENTAL EVALUATION OF CONCENTRICALLY-BRACED FRAME BEAM-COLUMN CONNECTION FLEXURAL RESPONSE

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ABSTRACT

This study investigates the flexural behavior and performance of beam-column connections in concentrically braced frames (CBFs) as part of evaluating the reserve lateral-load-resisting capacity in CBFs. Seven beam-column connections with gusset plates, employing double angle and end plate details, were studied using full-scale experiments to determine their flexural strength, stiffness and ductility. In this paper, the effects of end plate and angle thickness are evaluated, along with the influence of bolt distribution and weld type (end plate connections) and the impact of a supplementary seat angle (double angle connections). The connection behavior and performance are quantified using normalized moment vs. story drift data. In comparison to a baseline double angle detail, all connection variations increased the strength and stiffness. The end plate variations resulted in larger increases in strength, but were limited by bolt fracture. The double angle variations increased the strength by smaller margins, but strength loss occurred more gradually and larger drifts were sustained.

Research Motivation

Currently it is common for steel seismic lateral-load-resisting systems, particularly concentrically braced frames (CBFs), in moderate seismic regions to be designed using a response modification coefficient, *R*, of three, which allows for seismic detailing to be ignored. However, this procedure has no clear basis and there is no assurance that the amount of ductility inherent in these systems justifies the selection of $R = 3$. This issue was studied by Hines et al. (2009) using nonlinear time-history analysis and it was determined that there may be an unacceptable probability of structural collapse under maximum considered earthquake (MCE) seismic hazard for buildings designed with $R = 3$. However, little experimental data is available for $R = 3$ structures to quantify their reserve lateral-load-resisting capacity, which has the potential to significantly affect collapse probability. As a result, the importance of studying flexural response of beam-column connections in CBFs has become clear since these connections may contribute appreciable reserve capacity after braces have fractured.

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In addition, an alternate design philosophy is emerging for moderate seismic regions where wind is typically more critical than seismic (Hines and Fahnestock 2010). In this design approach, CBF braces and connections are designed for wind forces, but the gusset plate-brace connections are permitted to fracture under seismic demands. The resulting period elongation typically reduces seismic demands, but a flexible semi-ductile reserve system is necessary to provide collapse prevention performance under these demands. One possibility for providing the necessary reserve capacity is to employ the flexural strength and stiffness of the beam-column connections in the CBF after the braces are no longer active.

The present research investigates behavior and performance of beam-column connections in CBFs within the context of reserve capacity. The connections represent current-practice details and relatively simple variations on current-practice details, which are intended to improve reserve capacity. Seven beam-column connections with gusset plates, employing double angle and end plate details, were studied using full-scale experiments to determine their flexural strength, stiffness and ductility. In this paper, the effects of end plate and angle thickness are evaluated, along with the influence of bolt distribution and weld type (end plate connections) and the impact of a supplementary seat angle (double angle connections). The behavior and performance of the connections is assessed considering strength, stiffness and ductility.

A similar study by Kishiki et al. (2008) experimentally evaluated the flexural behavior of gusset plate connections for use in buckling-restrained braced frames (BRBFs). This study was initiated as a result of connection-related limit states observed in prior BRBF testing programs. Although the application is significantly different than the present research, the experimental results provide insight into a largely unexplored aspect of braced frame behavior, namely flexural behavior of CBF beam-column connections with gusset plates. In this study, the gusset plate increased the flexural stiffness, strength and ductility of the connection. However, local buckling of the beam flanges occurred at a story drift of 0.02 rad, which led to softening of the connections and, ultimately, to ductile fracture of the beam flanges near the critical section of the beam at the toe of the gusset plate.

Prototype Connection Design

To evaluate reserve capacity in CBFs for moderate seismic regions, a set of beam-column connections with gusset plates was developed for experimental evaluation. A standard bracing connection with double angles, which is typical of current practice, was established as a baseline and subsequent connection details were modest changes to improve the flexural strength, stiffness and ductility of the connection while minimizing the cost increase. The beam and column sizes for the prototype connections, W10x49 and W14x90, respectively, were taken from a CBF designed by LeMessurier Consultants (Hines 2007). The portion of the prototype braced frame used to develop the test specimens had a 9'-0” story height and a 19'-0” bay width. The design forces for the baseline double angle connection were also taken from this building. The beams and columns were ASTM A992 steel, the connection plates and angles were ASTM A36 steel, the ¾” diameter bolts were ASTM A325 steel and E70XX weld material was used.

The baseline connection, denoted CN1, was designed according to the uniform force method, as described in the American Institute of Steel Construction (AISC) *Steel Construction*

Manual (2005a). After designing CN1, modifications were made to the connection elements to study potential methods for enhancing the flexural capacity of the connection. These connections are denoted CN2 – CN8. The primary connection variations that were explored are shown in Fig. 1 and a detailed summary of the connection parameters is given in Table 1.

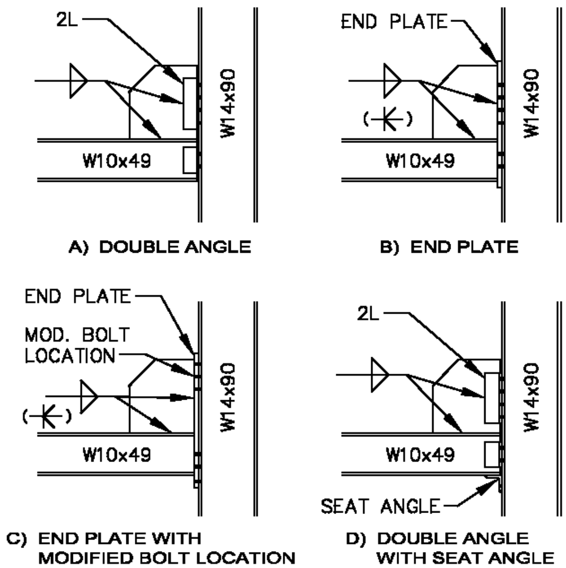


Figure 1. Connection details.

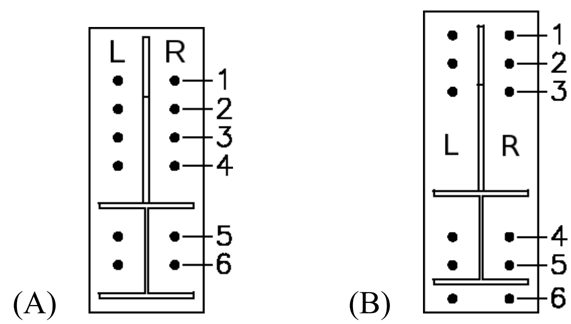


Figure 2. Bolt locations: (A) standard; (B) modified.

Table 1. Summary of connection parameters.

Specimen Number	Connecting Element	Gusset Plate-Beam Weld	Bolt Layout	Beam	Column
CN1	3/8" double angles	5/16" fillet	Standard	W10x49	W14x90
CN2	1" end plate	CJP	Standard	W10x49	W14x90
CN3	1" end plate	5/16" fillet	Standard	W10x49	W14x99
CN4	1" end plate	CJP	Modified	W10x49	W14x90
CN5	1" end plate	5/16" fillet	Modified	W10x49	W14x90
CN6	5/8" double angles with seat	5/16" fillet	Standard	W10x49	W14x99
CN7	5/8" double angles	5/16" fillet	Standard	W10x49	W14x90
CN8*	5/8" double angles with seat	1/2" fillet	Standard	W10x49	W14x99

*At the time this paper was written, CN8 had not been tested.

For connections CN2 – CN5, the CN1 3/8" double angles were replaced with a 1" end plate. Fig. 1B illustrates an end plate connection with a bolt configuration that matches the baseline connection, whereas Fig. 1C illustrates a similar connection with modified bolt locations where the bolts have been shifted to the top and bottom of the end plate to develop greater flexural stiffness and strength. Fig. 2 shows the geometry for the standard and modified bolt locations, as well as the numbering scheme used to reference the bolts in the discussion of test results. Figs. 1B and 1C also show that within these configurations, the gusset plate-beam and gusset plate-end plate welds were varied between fillet and complete-joint-penetration (CJP).

For connections CN6 – CN8, the CN1 3/8” double angles were replaced with 5/8” double angles. In addition to increasing the angle thickness, a 5/8” seat angle was added to connections

CN6 and CN8 to aid in transferring the beam bottom flange force to the column. A fillet weld was used to join the beam and gusset plate in CN6 and CN8, but the fillet weld size was 5/16" for CN6 and 1/2" for CN8. The bolt configurations for CN6, CN7 and CN8 conformed to the baseline connection, with additional bolts required for the seat angles in CN6 and CN8.

Large-Scale Test Setup

The T-shaped test setup for the large-scale connection subassembly is illustrated in Fig. 3. The beam-column subassembly was extracted from the prototype frame by assuming inflection points at story mid-height and beam mid-span. This assumption is based on the scenario that arises after both braces in the story have fractured. Seven large-scale connections have been tested in the Newmark Structural Engineering Laboratory at the University of Illinois at Urbana-Champaign, and an additional test will be conducted in the near future. Symmetric cyclic loading, based on the SAC protocol (FEMA 2000) and described in Table 2, was used to evaluate connection behavior and performance in this research.

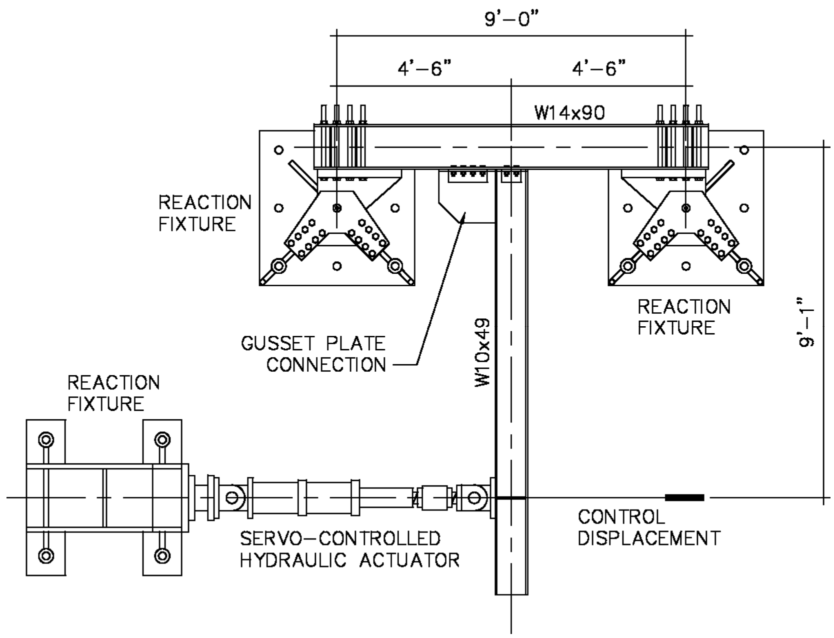


Figure 3. Beam-column subassembly test setup.

Table 2. Cyclic loading protocol.

Story Drift Angle (rad)	Load Point Displacement (in)	Number of Cycles
0.00375	0.41	6
0.005	0.55	6
0.0075	0.82	6
0.01	1.09	4
0.015	1.64	2
0.02	2.18	2
0.03	3.27	2
0.04	4.36	2
0.05	5.45	2
0.06	6.54	2

Moment-Rotation Response of Connections

Normalized moment vs. story drift is used to quantify the global behavior and performance of the prototype connections. The normalized moment is the ratio of the applied moment, M, to the expected plastic moment $M_{p,exp}$. The applied moment, computed at the toe of the gusset plate, is equal to the actuator load multiplied by the moment arm of 90 inches. $M_{p,exp}$ is defined based on the AISC *Seismic Provisions for Structural Steel Buildings* as:

$$M_{p,exp} = 1.1 R_y F_y Z_x \quad (1)$$

where R_y is the ratio of expected yield stress to the specified minimum yield stress, equal to 1.1 for ASTM A992 steel; F_y is the specified minimum yield stress; and Z_x is the plastic section modulus (AISC 2005c). Story drift is computed as the beam tip displacement divided by the distance to the centerline of the column, equal to 109 inches. The envelope of each connection response is plotted in Fig. 4. In addition, Table 3 summarizes the normalized moment and story drift for each connection at its maximum moment and maximum story drift. Because the connections are not symmetric with respect to the axis of bending, results are given for both positive moment and negative moment. Positive moment is defined as moment that induces tension in the gusset plate, which corresponds to points in the first quadrant of Fig. 4.

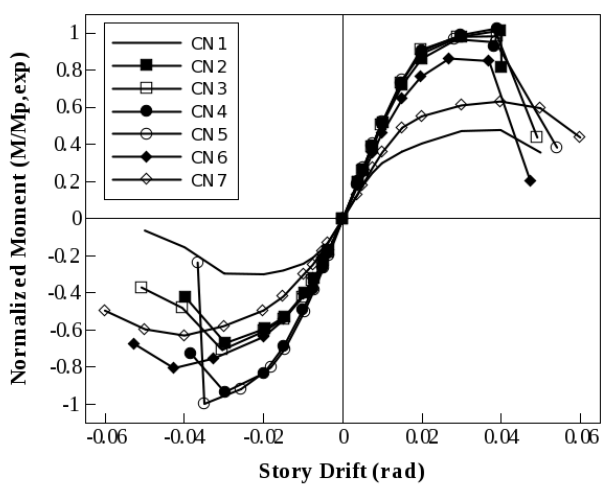


Figure 4. Normalized moment vs. story drift envelopes.

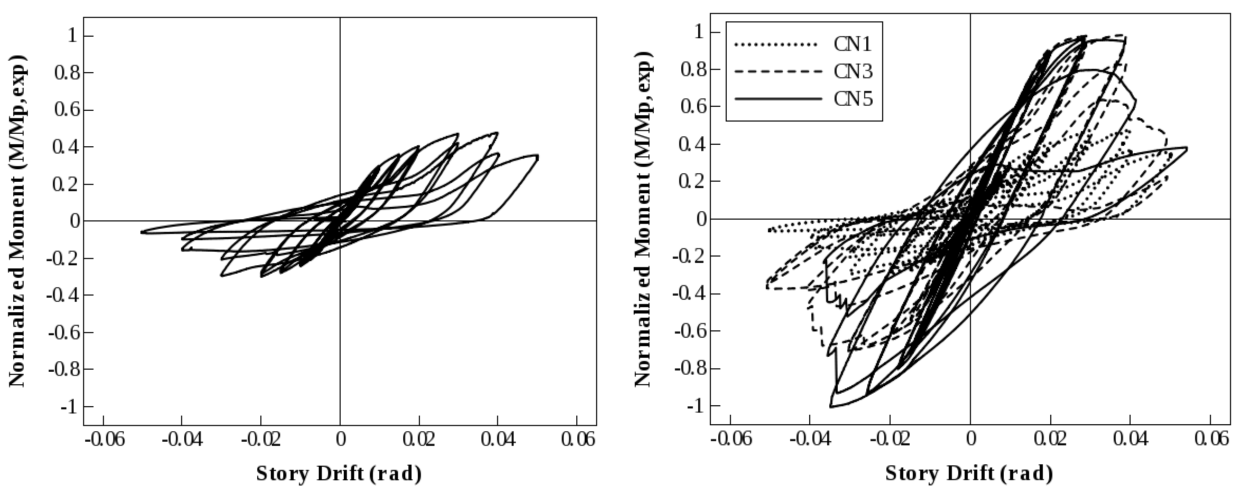
Table 3. Summary of connection behavior.

Specimen Number	Secant Stiffness $K_S \cdot (L/EI)_{beam}$	At Maximum Positive Moment		At Maximum Positive Drift		At Maximum Negative Moment		At Maximum Negative Drift	
		$\frac{M}{M_{p,\exp}}$	Drift (rad)	$\frac{M}{M_{p,\exp}}$	Drift (rad)	$\frac{M}{M_{p,\exp}}$	Drift (rad)	$\frac{M}{M_{p,\exp}}$	Drift (rad)
CN1	8.0	0.48	0.04	0.36	0.05	-0.30	-0.02	-0.06	-0.05
CN2	23.0	1.01	0.04	0.82	0.04	-0.59	-0.02	-0.42	-0.04
CN3	24.4	0.98	0.04	0.44	0.05	-0.71	-0.03	-0.37	-0.05
CN4	25.6	1.02	0.04	0.93	0.04	-0.93	-0.03	-0.73	-0.04
CN5	26.0	0.97	0.03	0.38	0.05	-1.00	-0.035	-0.24	-0.036
CN6	16.8	0.86	0.03	0.20	0.05	-0.81	-0.05	-0.67	-0.05
CN7	8.2	0.63	0.04	0.44	0.06	-0.63	-0.04	-0.49	-0.06

The results from CN1 are discussed first and compared to assumptions related to connection stiffness. After establishing the baseline behavior, the results from the end plate connections, CN2 – CN5, are presented and their behavior compared to the behavior of CN1. Finally, the results from the modified angle connections, CN6 and CN7, are presented along with comparisons to the baseline and end plate connections.

Behavior of Baseline Connection (CN1)

Current practice assumes that brace connections composed of double angles have negligible flexural strength and stiffness. However, the results from CN1 suggest that double angle brace connections have flexural strength and stiffness that could aid the collapse resistance of structures designed using R = 3. The cyclic normalized moment vs. story drift history for CN1 is illustrated in Fig. 5A, which shows that the peak positive and negative moments for CN1 are 48% and 30% of $M_{p,exp}$, respectively, and that the connection was able to sustain story drift between 0.03 and 0.04 rad without significant strength degradation. The normalized secant stiffness, $K_S \cdot (L/EI)_{beam}$, computed from data for connection moment vs. connection rotation at a rotation of 0.005 rad, is 8.0. The commentary to Chapter B of the *AISC Specification* defines the boundary between simple and partially restrained connection as $K_S \cdot (L/EI)_{beam} = 2$. The commentary also states, "Connections that transmit less than 20 percent of the plastic moment of the beam at a rotation of 0.02 rad may be considered to have no flexural strength in design." Since CN1 transmitted 40% of $M_{p,exp}$ in positive bending and 30% of $M_{p,exp}$ in negative bending at 0.02 rad, the stiffness and strength of CN1 warrant consideration in the design of a CBF.



(A) (B)

Figure 5. Normalized moment vs. story drift: (A) CN1; (B) CN1, CN3 and CN5.

Effect of End Plate and Bolt Location (CN3 and CN5)

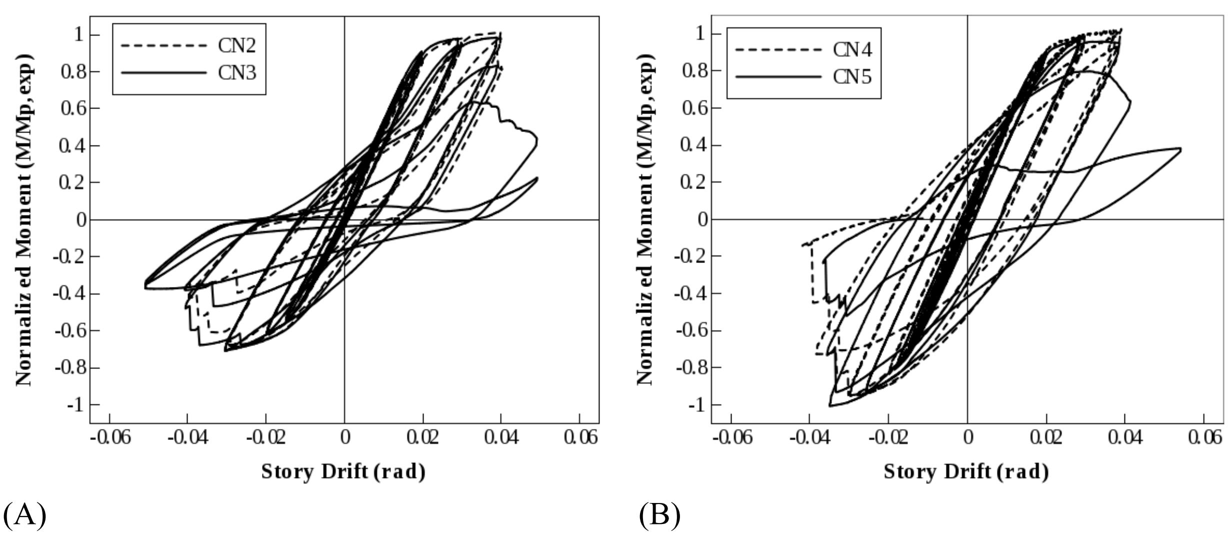
The first variation from the baseline connection was to use a 1" end plate instead of double angles. As shown in Fig. 2, two bolt configurations were investigated: (1) in CN3 the bolts were located in the same configuration as the double angle connection bolts and (2) in CN 5 the bolts were moved to the top and bottom of the end plate. The welds joining the gusset plate, beam and end plate were all fillet welds with the same size as the gusset plate-beam weld from CN1. The cyclic normalized moment vs. story drift histories for specimens CN3 and CN5 are given in Fig. 5B, along with the response from CN1.

As stated previously, replacing the 3/8" double angles with a 1" end plate significantly increased the flexural strength of the brace connections. In positive bending, CN3 and CN5 achieved $M_{p,exp}$ with CN5 also reaching $M_{p,exp}$ in negative bending. CN3 reached 71% of $M_{p,exp}$ in negative bending. As noted in Table 3, the end plate connections also had secant stiffneses that were significantly larger than the 3/8" double angle connection. However, the larger capacity of the end plate connections led to increased tensile demand on the bolts, which resulted in bolt fractures during negative bending of CN3 and CN5. The strength loss due to the fractures is shown in Fig. 5B. The failure of bolts R6, L6 and R5 in specimen CN3 occurred during the first cycle at 0.04 rad story drift, followed by failure of bolt L5 during the second cycle. For CN5, bolts R6 and L6 fractured simultaneously during the second cycle at 0.04 rad story drift, and bolts L5, R5 and R4 failed during the first cycle at 0.05 rad story drift.

Both connections experienced strength degradation in positive bending due to fracture of the gusset plate-beam fillet weld. The fracture initiated at the toe of the gusset plate, at approximately 0.03 rad story drift in both tests, and propagated along the length of the weld. Significant yielding of the gusset plate, beam web and beam flanges was also observed in CN3 and CN5.

Effect of Weld Type (CN2 and CN4)

For CN2 and CN4, complete joint penetration welds were used to join the beam, end plate and gusset plate. The normalized moment vs. story drift histories for CN2 and CN4 are plotted in Fig. 6 with the results from CN3 and CN5.



(A) (B)

Figure 6. Normalized moment vs. story drift: (A) CN2 and CN3; (B) CN4 and CN5.

The peak strengths in positive and negative bending, and the story drifts at which they occur, are almost identical between CN2 and CN3 and between CN4 and CN5. CN2 and CN4 also exhibited bolt fractures beginning at 0.04 rad story drift in negative bending, similar to CN3 and CN5. In addition, the strength of the CJP weld increased the demand on bolts R1 and L1 at higher drift levels. In CN2, bolt R1 fractured during the first excursion to 0.05 rad drift and similar concerns about the bolts in CN4 led to the decision to end the test after two cycles of 0.04 rad story drift. Both CN3 and CN5 sustained at least one loading cycle of 0.05 rad drift due to lower bolt demand, which was a direct result of the gusset plate-beam fillet weld fracture.

Effect of Angle Thickness and Seat Angle (CN6 and CN7)

The final connection variations aimed to increase the strength of the brace connection through thicker angles and a supplemental seat angle. CN7, tested before CN6, was a replica of CN1 with the 3/8" double angle thickness increased to 5/8". For CN6, the 5/8" double angle thickness was maintained and a supplemental 5/8" seat angle was welded to the bottom flange of the beam and bolted to the column. The length of the seat angle was determined by the workable gage of the column and the minimum bolt edge distance. The normalized moment vs. story drift histories for CN6 and CN7 are given in Fig. 7.

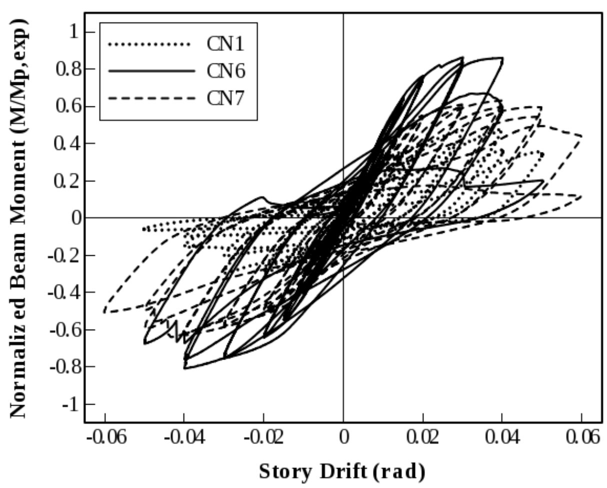


Figure 7. Normalized moment vs. story drift for CN1, CN6 and CN7.

Figure 7 illustrates that increasing the thickness of the double angles, from CN1 to CN7, significantly increased the flexural strength of the connection and the addition of the seat angle, CN6, increased the flexural strength beyond the capacity reached in CN7. It is also apparent from Figs. 4 and 7 that the increase in angle thickness had a larger impact on the negative moment strength and addition of a seat angle had a larger impact on positive moment strength. In addition, the seat angle had a much larger influence on the secant stiffness of the connection. CN6 was two times stiffer than CN1 and CN7. Both CN6 and CN7 sustained at least one cycle of loading at 0.05 rad story drift, with CN7 sustaining two cycles at 0.06 rad story drift. However, neither connection reached load levels achieved by the end plate connections.

In addition, bolts R6 and L6 of CN6 fractured in negative bending during the first excursion to 0.05 rad story drift. The drift level at which the failures occurred was comparable to the end plate tests, but the load level was lower due to prying forces induced by the flexibility of the double angles. Both connections (CN6 and CN7) also experienced gusset plate-beam fillet weld fractures similar to those in CN3 and CN5.

Summary and Conclusions

Large-scale testing was used to study the behavior and performance of beam-column connections with gusset plates under cyclic loading. These connections are capable of providing reserve lateral-load-resisting capacity in concentrically-braced frames for moderate seismic regions. The following conclusions are drawn from the global response of the test specimens.

- The baseline brace connection, CN1, which was a typical double angle detail, had significantly more stiffness and strength than has traditionally been considered in design of concentrically-braced frames. CN1 exceeded the strength and stiffness thresholds commonly used to classify simple connections, as defined in the Commentary to the *AISC Specification*.

- The end plate connections demonstrated greater strength than the angle connections. In addition, the end plate connections with a modified bolt location demonstrated greater negative moment strength than the end plate connections with the standard bolt location. All end plate connections achieved or nearly achieved positive moment of $M_{p,exp}$. The modified end plate connections achieved or nearly achieved negative moment of $M_{p,exp}$ and the standard end plate connections had a negative moment strength of 60 to 70% of $M_{p,exp}$.

- The modified angle connections, CN6 and CN7, exhibited more deformation capacity than the end plate connections. However, neither CN6 nor CN7 was able to achieve $M_{p,exp}$ in positive or negative bending. Positive bending strength was increased more by adding a seat angle and negative moment strength was increased more by increasing the angle thickness. Bolt fractures occurred in CN7 due to prying forces induced by flexibility of the double angles.

Ongoing and Future Research

There are several short- and long-term goals of this braced frame connection research program. In the short-term, connection CN8, which is a 5/8" double angle connection with a seat angle and a 1/2" fillet weld between the gusset plate and beam, will be fabricated and tested. The primary goal of CN8 is to prevent gusset plate-beam fillet weld fracture without resorting to a complete joint penetration weld. Three-dimensional finite element models of the connections examined in the large-scale tests will be created and validated using the test results. The validated numerical models will be used in a parametric study that will examine a broader range of variables including: the effects of beam depth, concrete floor slab and gusset plate damage sustained prior to brace fracture.

The parametric numerical study will also lead into a second experimental phase that will vary beam depth and connection parameters identified in the numerical study. The second phase of testing will seek to identify the most efficient approaches for increasing the stiffness, strength and ductility of brace connections, which will then contribute reserve capacity for lateral-load-resistance. Numerical system studies are also planned to quantify the strength and ductility demands on connections within a reserve lateral-load-resisting system and the ability of the connections to prevent structural collapse.

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