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Experimental Evaluation of the Robustness of Flexible WT Steel Connections

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Abstract: Flexible (simple) shear connections commonly used in steel frame buildings including shear tabs, single angles, double angles, end plates, seated, and WT connections are very economical and are relatively easy to fabricate. These connections are used for shear resistance, but it is commonly believed that they are capable of sustaining an interaction of rotational and axial load demand necessary for a steel framed building structures to resist collapse in the event of unanticipated damage scenarios. The objective of this paper is to outline and discuss an experimental effort designed to evaluate the robustness of flexible WT connections with 3, 4, and 5 bolt configurations subjected to a loading scenario commonly used to simulate loss of a central support in a steel frame building. The experimental testing provides important information regarding the ability of these connections to sustain large rotational demands in conjunction with axial tension forces generated through geometric stiffness (catenary) effects.

1. Introduction

Inherent robustness of steel framed building structures and designer-based approaches for enhancing it are not clearly defined in United States design standards (AISC 2010a). Reinforced concrete design codes include provisions intended to enhance structural integrity and inherent robustness in cast-in-place and precast concrete systems (ACI 2008). The typical steel framed structure includes many components (*e.g.* in-fill beams, girders, moment-resisting connections, flexible or simple connections) that can and likely do contribute to the inherent robustness of the overall system. Gravity-load connections present throughout a steel building framing system are most often considered to be flexible (simple) and are not designed for bending moment, rotational, and axial load demand. The ability of these flexible connections to sustain rotational and tensile force demands necessary in the formation of the alternate load paths required for system resistance to disproportionate collapse is not fully understood.

Several relatively recent research efforts have been undertaken to understand how connections assumed to be flexible at design behave with significant axial forces, transverse forces, and rotational demand present. Header angle, knife angle, single angle, and shear tab connections subjected to 0.03 radian rotational demand with constant shear force demand and tension have been studied experimentally (Guravich and Dawe 2006). Double angle and flush end plate connections subjected to axial load and rotational demands have been experimentally evaluated as well (Owens and Moore 1992). Girhammer (1980a, 1980b) conducted experimental testing to evaluate the ability of bolted heel connections and bolted end plate connections to resist loading demands likely to occur in a column loss scenario. High fidelity finite element modeling of shear tab connections has also been undertaken (Sadek *et al* 2008). There has been no experimental data generated to contribute to understanding the behavior of flexible steel WT connections subjected to combined axial tension forces in combination with significant rotation demand.

Experimental testing in the present research effort considered simple wide flange tee section (WT) shear connections shown in Figure 1. WT shapes used were WT5x22.5.

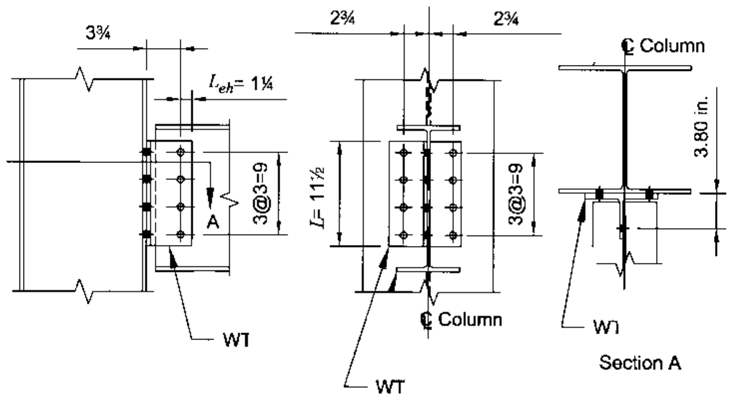

Figure 1: Flexible WT Connection Configuration Considered

A standard flexible (simple) WT connection often used as a bolted alternative to the welded-bolted shear tab connection. It is assumed to resist only transverse shear forces when designed. Their ability to resist simultaneously applied shear, rotation, and axial force demands is important to understanding the inherent robustness of steel framing systems. WT shear connections utilizing three, four, and five bolts are considered in the experimental tests discussed. The experimental testing conducted provides important information related to the ability of these connections to sustain large rotational demands in conjunction with axial tension forces generated through geometric stiffness (catenary) effects as vertical deformation is accumulated.

2. Experimental Program

The experimental program included WT connections with three bolting patterns: three rows (3WT), four rows (4WT), and five rows (5WT). Each configuration included three tests. Figure 2 shows the general layout and geometry of each WT specimen (yield stress nominally 50 ksi).

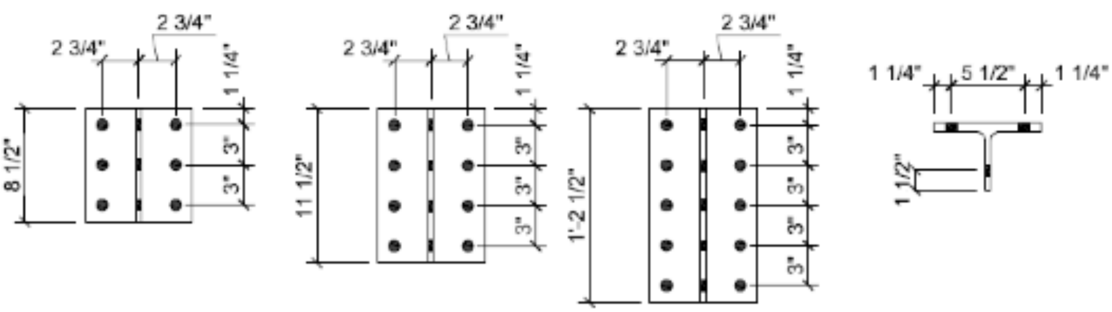

Figure 2: WT Connection Configurations Included in Experimental Study

Design of the WT connections followed U.S. specification requirements (AISC 2010a) and accepted design procedures (AISC 2010b). Standard bolt holes and 3/4-in diameter bolts were used and specimens were designed so that failure limit states were seen exclusively in the WT components. Further details are available (Friedman 2009). The experimental fixture (Figure 3) included WT test specimens centered in a two span system with pin connections on each end of central beams and a central column.

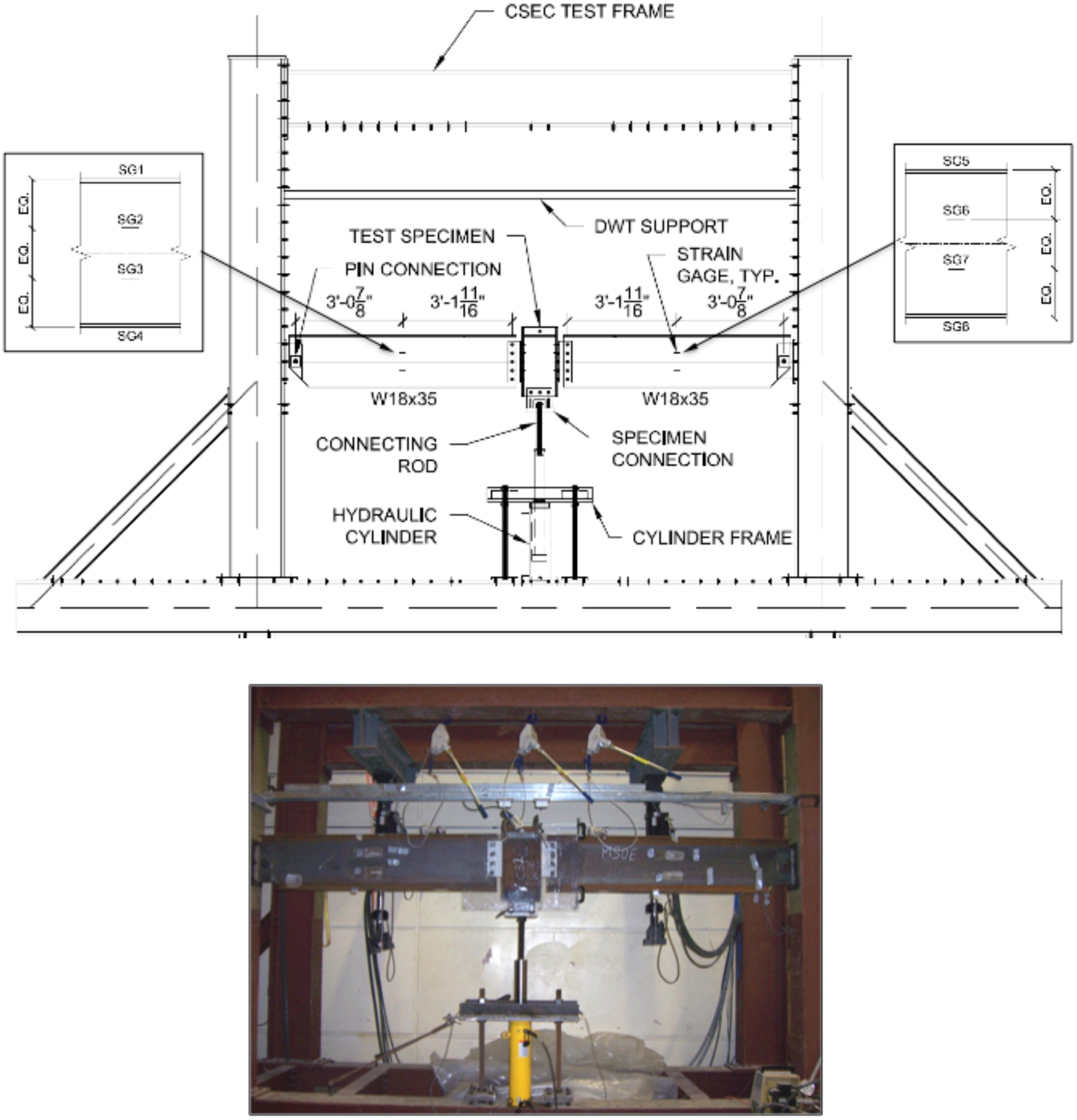

Figure 3: Fixture and Instrumentation Used in Experimental Testing.

A single acting Enerpac RR-10018 hydraulic cylinder was used to apply loading through a coupling rod connected to a re-usable central column. Two Sensotec Model 41-A530-01-03 load cells were used along with a National Instruments DAQPad Model 6020E, 12 bit data acquisition system with National Instruments LabView software. The hydraulic cylinder pulled down on the test specimen. Two Unimeasure Model PA-30-DS-L5M draw wire transducers (DWTs) were attached to the test specimen to measure deflection. Averaged DWT measurements were used to define vertical deformation and rotation at the connection was determined based on DWT measurements and assembly geometry.

Four Vishay EA-XX-125BT-120 (120 ohm) strain gages in quarter-bridge completion were applied on each beam (right and left). Gages were placed on the center face of the top flange, the center face of the bottom flange, at the top third point of the full beam depth on the web, and at the bottom third point of the full beam depth on the web.

The W18x35 test beams were used for multiple rounds of testing and the test assembly was designed to accommodate shear tab testing, WT testing, and single angle testing. A 1/2" doubler plate (yield stress of 36ksi) was welded to each beam at the connection point to the WTs in order to prevent damage to the beam during testing. The connections at the ends of the W18x35 beams were pinned connections.

3. Experimental Results

Nine specimens were tested to collect data regarding axial, shear and moment interaction of WT connections. Strain gages located near the mid-span of each W18 beam (Figure 3) continuously collected data throughout each test and the strain data was used to determine internal forces (axial load and bending moment) in the W18 beam, which were then used to extrapolate forces to the connection. Draw wire transducers (DWT) measured the amount of displacement at the top plane of the test specimen and these displacements were used to determine the amount of rotation at the connection. Data collected also included the load applied to the coupling rod and central column stub.

Strain gage readings were used to compute the axial force and bending moment in the beams and connections. Linear extrapolation of moment from strain gage location to connection was performed. Computations for the left beam used strain gage 2 and strain gage 3, which were located 2.95 inches above and below the beam neutral axis, respectively. Computations for the right beam used strain readings from strain gage 5 and strain gage 8, which were located 8.85 inches above and below the beam neutral axis, respectively. Further details regarding the computation of axial forces, deformations, rotations, shear loading and bending moments are available (Friedman 2009).

3.1 Three-Bolt 3WT Results

All of the three-bolt WT specimens (3WT1, 3WT2, and 3WT3) failed due to a bolt shear rupture at the bottom bolt of the connection. Figure 4 includes illustration of the final position of the test assembly and the WT connection at the end of testing.

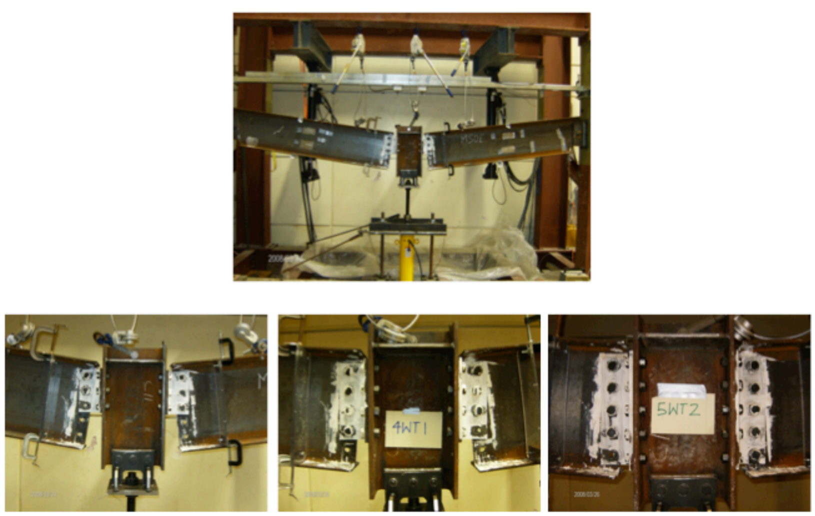

Figure 4: Connection Configuration Typically Found and Termination of Testing

There was evidence of significant bolt bearing deformations at the hole of the WT stem at the bottom bolt and a minor amount of deformation along the edge of the WT stem. There was a significant level of ductility exhibited before bolt shear rupture occurred. The upper half of the 3WT connection was found to be in compression while the lower half was in tension with rotation about the center bolt.

The force-rotation response typical of the 3WT connections shown in Figure 5 indicates that the 3WT connections exhibit a transition from flexural resistance to a catenary type behavior. The graphs show that the measured moment in the connection increases until it reaches approximately 0.12 radians of rotation. The connection transitions from flexural resistance to significant catenary behavior indicated by the rapid rate of increase in axial load at approximately 0.07 radians. Geometric stiffness results in significant increase in axial loading as the bending moment in the connection lessens. This continues until bolt fracture.

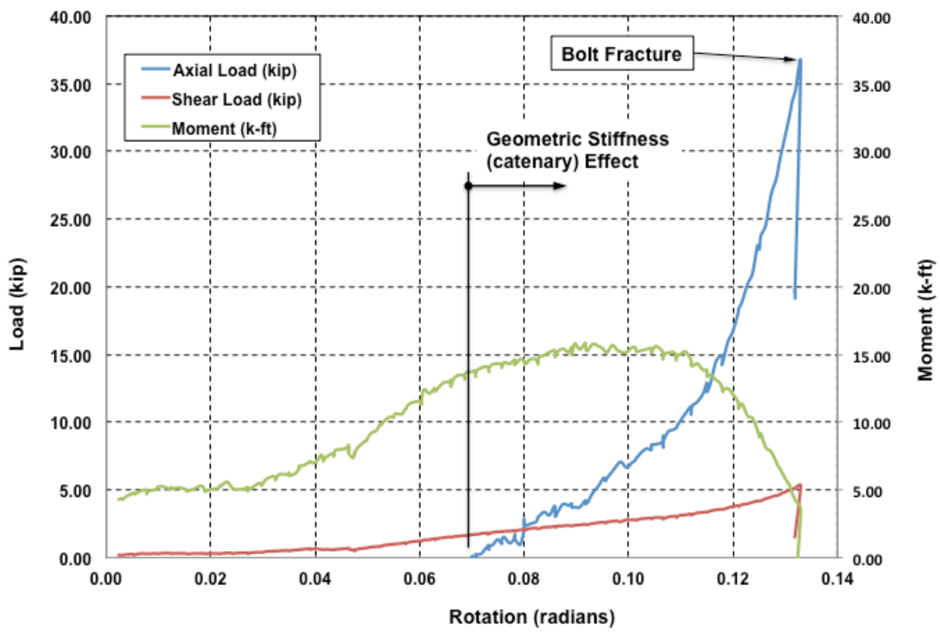

Figure 5: Response Typical of 3WT Connections (Right Beam)

Bolt shear rupture in the bottom bolt occurred in the catenary behavior region for all specimens. At the point of shear rupture (approximately 0.13 radians), the main load transfer mechanism was catenary action (geometric stiffness effect) with a minimal amount of bending moment. Maximum bending moment magnitudes reached in the 3WT specimens ranged from13.8 to16.8 k-ft, which represents approximately 5-6% of the W18x35 beam plastic moment capacity. The maximum axial force measured in the beams (and connections) ranged from 35 to 41.4 kips at failure. Rotations were significant and ranged from 0.092 to 0.102 radians at the maximum moment. Rotation demand at failure ranged from 0.125 to 0.133 radians.

3.2 Four-Bolt 4WT Results

Bolt shear rupture was the defining limit state for the 4WT connections. Figure 4 illustrates the final position of the test assembly at the termination of the test. The response of 4WT1 and 4WT2 connections is shown in Figure 6 and the response of the 4WT3 is shown in Figure 7. The axial load accumulation after the connection plastic moment is attained occurs at the same rate in the 4-bolt connections as that in the 3-bolt connections. The rotational capacity of the 4WT connections was much less than that seen in the 3WT connections and this follows behavioral assumptions outlined in seismic design procedures (FEMA 2000). Bending moment capacity of the 4WT connections is greater than the 3WT connections as expected.

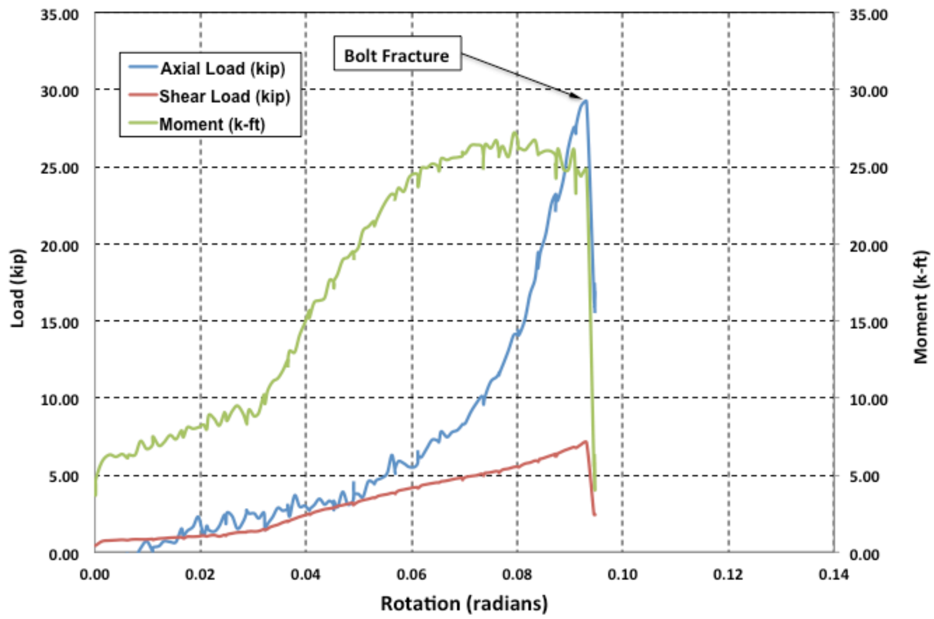

Figure 6: Response Typical of 4WT1 and 4WT2 Connections (Right Beam)

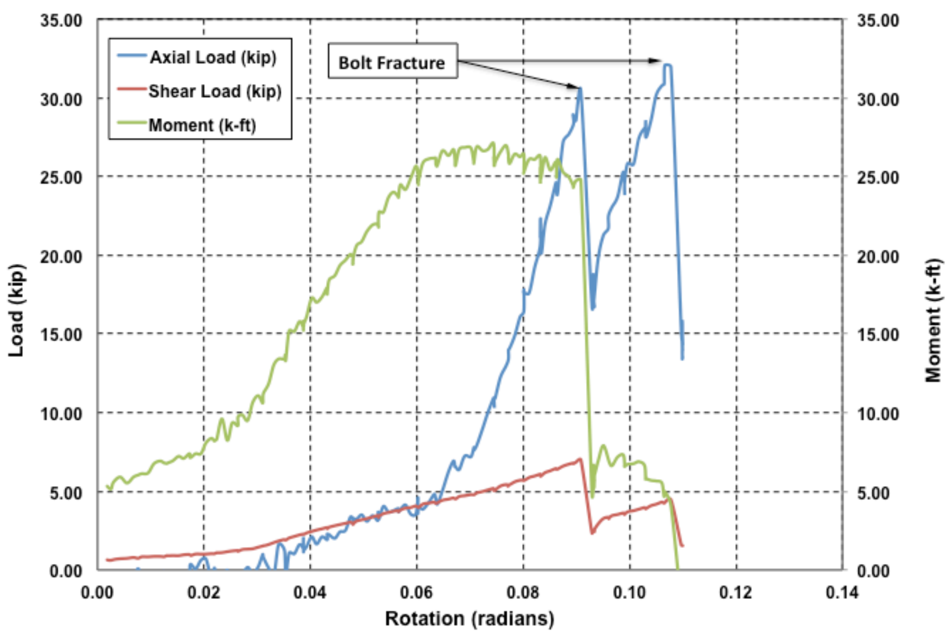

Figure 7: Response for 4WT3 Connection (Right Beam)

Specimen 4WT3 included loading re-distribution following initial bottom-bolt fracture. The system was able to carry further loading with significantly reduced bending moment and axial loading, but the magnitude of the shear did not fully recover following initial bolt rupture. Peak rotational capacity in all 4WT connections was approximately 0.09 radians.

3.3 Five-Bolt 5WT Results

All of the 5WT tests terminated with bolt shear rupture at the bottom bolt of the connection. Figures 8 and 9 illustrate the response of the connections during testing. The flexural resistance of the 5WT connections exceeded 4WT and 3WT connections as expected. However, it is interesting to note that the axial load occurring at the strength limit state for the system is relatively consistent among all connection configurations tested.

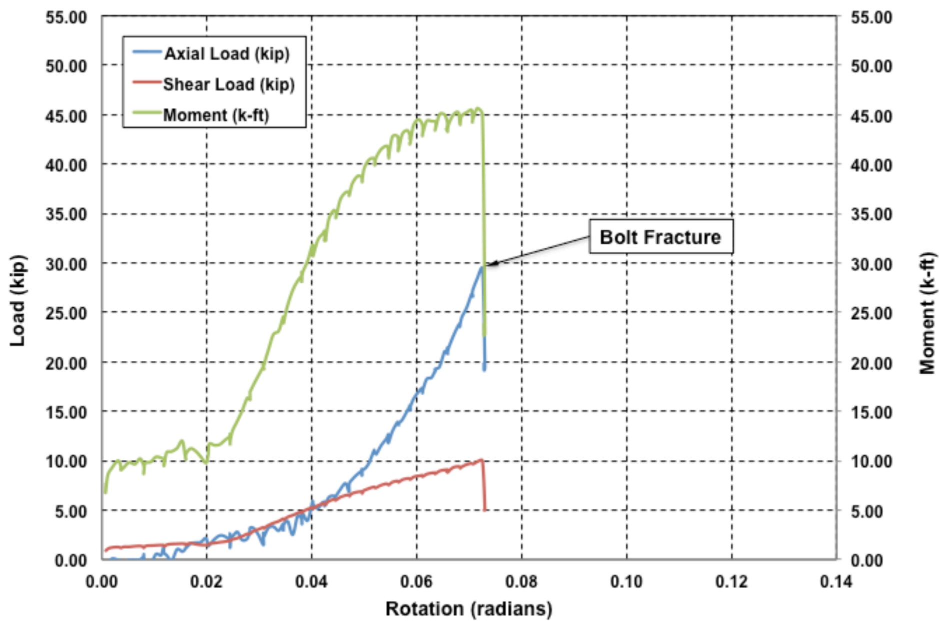

Figure 8: Response Typical of 5WT1 and 5WT2 Connections (Right Beam)

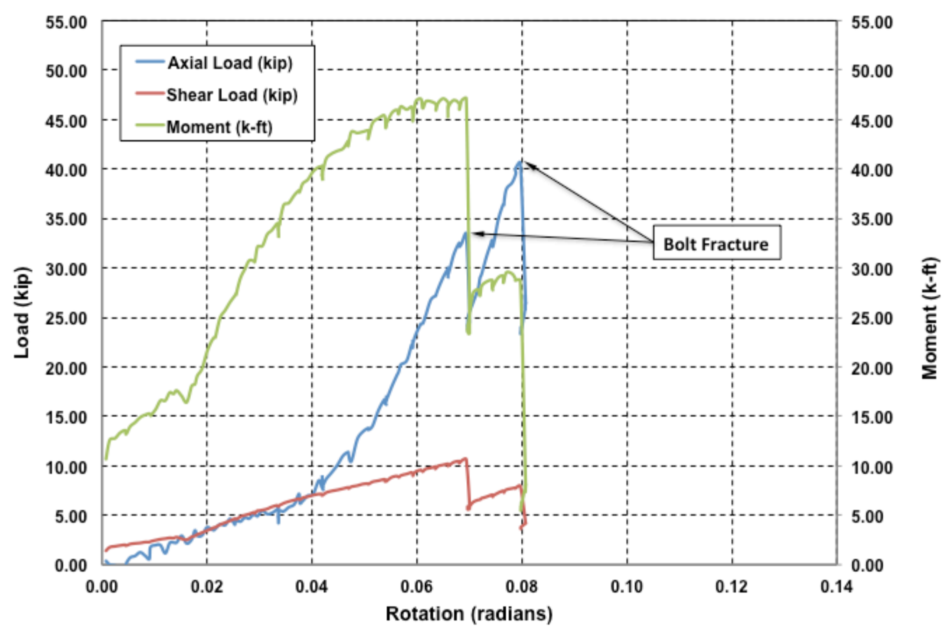

Figure 9: Response Typical for 5WT3 Connection (Right Beam)

The rotational capacity of the 5WT connections tested was less than that seen in the 4WT and 3WT connections. This follows expected behavior based upon strain demands at the extreme bolt locations as additional bolt rows are inserted. The rotational capacity for all 5WT connections was approximately 0.07 radians. Connection 5WT3 included force re-distribution after initial bolt fracture similar to that seen in connection 4WT3, but the load carrying capacity of the system as exhibited by shear force carried by the beam never fully recovered.

4. Summary and Conclusions

Nine experimental tests of WT shear connections were undertaken and the results were discussed. The WT connections tested exhibited measurable bending moment capacity as expected. An increase in the number of bolt rows in the WT connections results in an increase in flexural resistance of the connections. This flexural resistance then is the majority contributor to the load carrying capacity in the system.

The contribution of catenary action to the load carrying capacity of the system is more significant in the 3WT connection configuration than in the 4WT and 5WT configurations because bolt fracture prematurely limits system capacity. As a result, the ability of geometric stiffness (*i.e.* catenary) effects to contribute to the load carrying capacity is likely to be precluded by premature bolt fracture. The axial load occurring at the strength limit state for the system considered was relatively consistent among all connection configurations tested resulting from extreme bolt fracture.

The testing of Astaneh and Nader (1989) included imposition of targeted rotational demand to measure the amount of flexibility and ductility of the connections and this former testing showed minor yielding. The present experimental work included greater imposed rotational demand. A comparison of results of the present experimental tests and those of Astaneh and Nader (1989) indicates that if axial loads are not present, a WT connection may have more rotational capacity than the present experimental results exhibited.

The experimental testing of Guravich and Dawe (2006) included rotational demand imparted with constant applied transverse shear force. The sustained rotational demand in this former work was much less than that seen in the present testing. The present experimental results suggest that limited geometric stiffness effects (*i.e.* catenary action) would occur at the rotational demand limits seen in this former work (Guravich and Dawe 2006). Furthermore, the magnitude of the transverse shear at the location of the connection seen in the present experimental results was very low at bolt fracture.

Sadek *et al* (2008) used high-fidelity finite element (FE) analysis to study a floor system with loss of a supporting column. The simulation model in this former effort was similar to the experimental fixture used in the present research. The connection used in the FE model was a standard three-bolt shear tab designed using accepted procedures (AISC 2010a). The 3WT connection considered in the present study is expected to behave in a similar manner to the 3-bolt shear tab connection in this former work. The results of these former FE simulations (including limit states seen at failure) and the present experimental results for the 3WT connections exhibited similar behavior.

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References

AISC 2010a. *Specifications for the Design of Steel Buildings*, American Institute of Steel Construction, Chicago, IL.

AISC 2010b. *Manual of Steel Construction 14th Edition*, American Institute of Steel Construction, Chicago, IL.

ACI 2008. *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-08, American Concrete Institute, Farmington Hills, MI.

Astaneh, A. and Nader, M. 1989. Design of Tee Framing Shear Connections, *Engineering Journal*, American Institute of Steel Construction, Chicago, IL, First Quarter, pp. 9-20.

FEMA 2000. *State of the Art Report on Connection Performance*, FEMA-355D, Federal Emergency Management Agency, Washington, DC.

Friedman, A.D. 2009. *Axial, Shear and Moment Interaction of WT Connections*, MS Report, Milwaukee School of Engineering (MSOE), Milwaukee, WI.

Girhammar, U. A. 1980a. *Behaviour of Bolted Beam-Column Connections under Catenary Action in Damaged Steel Structures*. Lulea: Swedish Council for Building Research.

Girhammar, U. A. 1980b. *Dynamic Fall-Safe Behaviour of Steel Skeleton Structures Having Bolted Connections*. Lulea: Swedish Council for Building Research.

Guravich, S. J. and J.L. Dawe. 2006. Simple Beam Connections in Combined Shear and Tension, *Canadian Journal of Civil Engineering*. Vol. 33, pp. 357-372.

Owens, G. W. and D.B. Moore. 1992. “Steelwork Connections – The Robustness of Simple Connections.” The Structural Engineer, Vol. 70 (3), pp. 37-45.

Sadek, F., El-Tawil, S. Lew, H.S. 2008. Robustness of Composite Floor Systems with Shear Connections: Modeling, Simulation, and Evaluation, *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 134 (11), pp. 1717-1725.