RESILIENT INFRASTRUCTURE
June 1–4, 2016

EXPERIMENTAL EVALUATION OF THE ROBUSTNESS OF SINGLE PLATE SHEAR CONNECTIONS UNDER QUASI-DYNAMIC LOAD

Jacklyn E. Lesser
Graduate Research Assistant, Milwaukee School of Engineering, USA

Christopher H. Raebel
Associate Professor, Milwaukee School of Engineering, USA

Christopher M. Foley
Professor and Chair, Marquette University, Milwaukee, USA

ABSTRACT

Flexible (simple) shear connections commonly used in steel-framed buildings are very economical and are relatively easy to fabricate. These connections are used for shear resistance, but recent studies have shown that they are capable of sustaining an interaction of rotational and axial load demand necessary for steel-framed building structures to help 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 single plate shear connections under a quasi-dynamic loading scenario simulating the loss of a central column. The experimental program included eleven full-scale tests of a system consisting of two wide flange beams connected to a central wide flange column stub by means of the shear plate connections. Three, four, and five bolt configurations were tested, and two of the tests utilized galvanized bolts. 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 when subjected to rapidly applied vertical loads.

Keywords: Robustness, Progressive Collapse, Steel Structures, Connections.

1. INTRODUCTION

Typical elements within a structural steel framework, such as infill beams, girders, and connections, are intended to resist the gravity and lateral loads demanded of them based on typical design standards (e.g., AISC 2010a). These elements are designed to resist their share of the gravity and lateral loads acting on the frame, and they are usually not directly intended to contribute to the inherent robustness of the system. However, these elements can and likely do contribute to the overall robustness even without direct consideration to their contributions. Such contributions have been recognized in other design codes such as the concrete design codes (ACI 2014) where design provisions, although often prescriptive, are included with the intention of enhancing the robustness of cast-in-place and precast systems.

Most structural steel frames include gravity-load connections that are most often considered to be flexible (i.e., “simple”) and are not designed to resist bending moment or axial forces. These flexible connections have the ability to resist measurable rotational and tensile force demands that are necessary for the resistance to disproportionate collapse. The capacities of these connections and the alternate load paths developed have recently been a topic of interest by researchers; however, even considering recent advances the topic remains one that is not fully understood by the structural engineering community. Ellingwood et al. (2009) notes that the deformation capacity of elements subjected to force and moment interaction is an assumption that is worthy of further investigation.

Recent research initiatives have taken many different paths in identifying and quantifying robustness characteristics in steel framed structures. Some initiatives have focused on the entire steel framework, either as a reduced model (e.g., Main 2014; Alashker et al. 2010; Foley et al. 2008) or as an entire prototype building system (e.g., Raebel 2011). Others have narrowed their focus on the interactive behaviour of the connections within the framework by means of experimental evaluation (Raebel et al. 2012; Oosterhof and Driver 2012; Oosterhof and Driver 2011; Guravich and Dawe 2006) and by finite element or other analytical modeling (Main 2014; Main and Sadek 2012; Alashker et al. 2010; Sadek et al. 2008). The single plate connection (often also called a "shear tab") is often the connection of choice in the experimental and analytical studies due to its simplicity and relative ease in limit state identification and modeling.

The subject of the current research initiative is the shear plate beam-to-column connection as illustrated in Figure 1. A standard connection (AISC 2010b), the shear plate offers many advantages to the fabricator and engineer due to its simplicity and constructability. The conventional shear plate is assumed to resist transverse shear forces (i.e., in vertical direction parallel to the line of bolts in Figure 1) with minimal eccentricity, thus resulting in minimal moments within the connection.

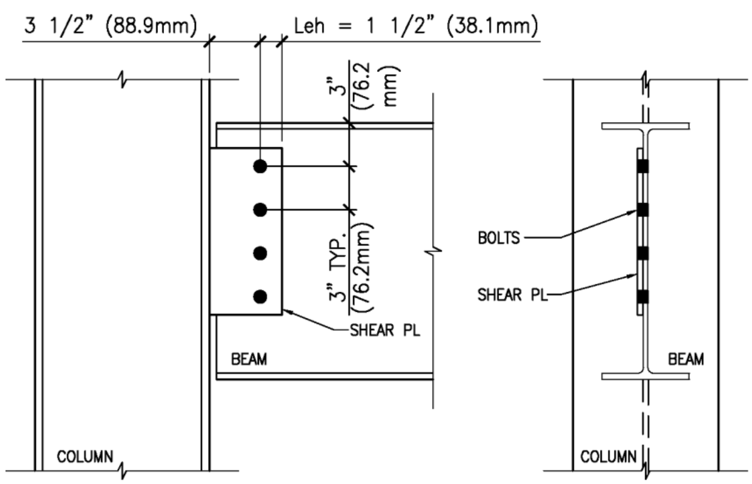


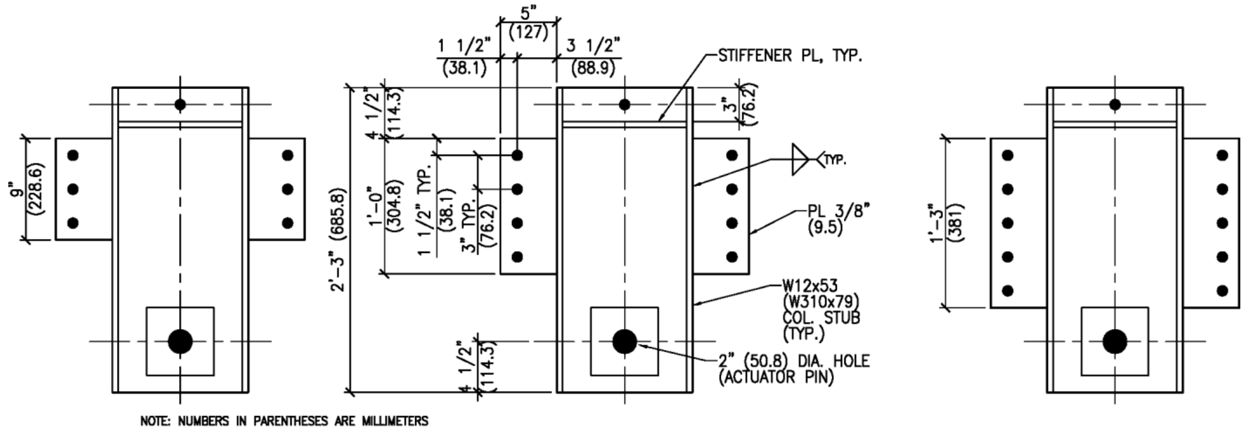
Figure 1: Shear Plate Connection Configuration.

Liu and Astaneh-Asl (2000) investigated the rotational flexibility and ductility of shear plate connections with and without the effects of slab. Geschwindner and Gustafson (2010) investigated the structural integrity of the shear plate connection by means of a limit state analysis in an effort to satisfy the integrity requirements of the International Code Council (2009) and New York Building Code (2008). However, little experimental data has been generated towards the understanding of the behaviour of simple shear plate connections subjected to a significant magnitude of rotation demand in concert with interactive axial and shear forces. Experimental studies performed by Thompson (2009) and analytical studies by both Main and Sadek (2012) and those discussed by Daneshvar and Driver (2010) have provided much needed experimental data and analytical insight into the interactive force and moment behaviour of the shear plate connection with significant rotational demand under a column removal scenario. The experiments clearly showed that the shear plate connection has the ability to resist those forces and moments and allows the framework to generate significant axial tension forces through geometric stiffness (i.e., catenary) effects as the vertical deformation progresses.

The loading rate performed by Thompson (2009) was very slow, such that the loading could be considered static, or "quasi-static." The present research initiative investigates the behaviour of the shear plate connection while undergoing interactive forces and moments, including significant rotation demands; however, the loading rate used in the present research is significantly faster than Thompson's tests. The loading rate is described as "quasi-dynamic" (also described as "dynamic" herein) because the experimental system was not able to reach speeds expected of the free-fall of a suddenly removed central column, but the speeds in the present study are significantly increased when compared to that generated during Thompson's experiments.

2. EXPERIMENTAL PROGRAM

The experimental program included shear plate connections with three bolting patterns: three rows (3ST), four rows (4ST), and five rows (5ST). Figure 2 shows the general layout and geometry of each specimen. The dimensions shown for the 4ST are typical for all specimens. The yield stress of the plate was nominally 36 ksi (250 MPa) and the modulus of elasticity was nominally 29,000 ksi (200 GPa). Material tests resulted in an average yield stress of 59 ksi (407 MPa) and an average modulus of elasticity of 30,395 ksi (209.6 GPa). The column stub is a W12×53 (W310×79).



(a) 3ST (b) 4ST (c) 5ST

Figure 2: Shear Plate Connection Configurations Included in Experimental Study.

Design of the shear plate connections followed U.S. specification requirements (AISC 2010a) and accepted design procedures (AISC 2010b). Standard size bolt holes were used (i.e., 1/16 in. (1.6mm)) larger than the bolt diameter) and the test beams and column stub were designed so that failure limit states were exclusive to the shear plate components and the bolts attaching them to the beam or column stub. Further details regarding the calculated limit states and specimen design are available (Lesser 2016; Thompson 2009).

An illustration of the experimental fixture is shown in Figure 3, and a photograph of the pre-test experimental setup is shown in Figure 4. The test specimens are centered in a two-span system that connects two reusable pin-ended test beams to a central stub column. The column stub was unrestrained above the beams in an effort to focus on the rotational demands at the shear plate connections.

Loading was applied by means of an MTS hydraulic system. An MTS 201.30T single ended hydraulic actuator with integral force and displacement instrumentation was used to apply loading through a clevis-styled heavy plate assembly connected by means of a single steel pin to a central column. The hydraulic actuator pulled down on the test specimen. Two Unimeasure Model PA-30-DS-L5M draw wire transducers (DWTs) were attached to the flanges of the column stub to measure both total and differential deflection. Averaged DWT measurements were used to define vertical deformation. Rotation at the connection was determined based on DWT measurements and assembly geometry. Force, displacement (actuator and DWT) and strain data were collected through a customized software program, and the force and actuator displacement data were also collected through the MTS controller software. Force and displacement data from the MTS system was compared to the data collected by the custom software to ensure accuracy and consistency between the two systems.

Seven Vishay “Micro-Measurements” CEA-06-062UW-350 (350 ohm) strain gages in quarter-bridge completion were applied on each beam (right and left). The gages were placed on the center face of the top flange, the center face of the bottom flange, and approximately equidistant between the flanges on the web (see Figure 3).

The W18×35 (W460×52) test beams were designed for repeated use as the test assembly was used for both shear plate (present work) and WT (Hayes 2016) testing programs. A 1/2 in. (12.7mm) doubler plate with a nominal yield stress of 36 ksi (250 MPa) was welded to each beam at the connection point to the shear plate in order to prevent damage to the beam during testing. The W18×35 (W460×52) beams were connected to the frame columns by means of a single bolted pin ended connection.

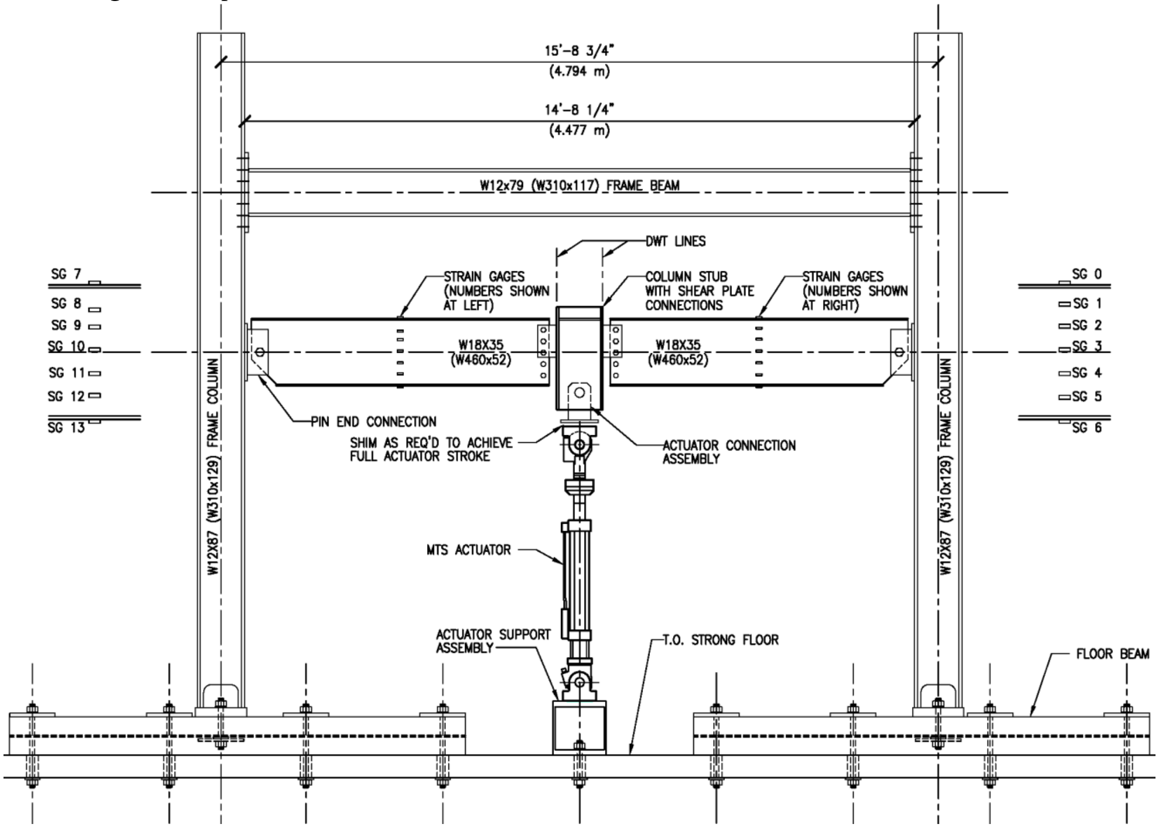


Figure 3: Fixture and Instrumentation Used in Experimental Testing.

Figure 4: Typical pre-test configuration.

3. EXPERIMENTAL RESULTS

In all, eleven specimens were tested in an effort to measure data that would result in quantifying the interacting axial force, shear force and moment at the point of the single plate connection. The actuator displacement rate was approximately 2.5 in. (63.5 mm) per second for quasi-dynamic tests. The strain gages located near mid-span of each W18 (W460) beam continuously collected data during each experiment. DWTs measured the amount of displacement at each flange of the column stub, and these measurements were used to calculate the amount of rotation at the connection. Applied force was measured through internal instrumentation in the MTS actuator, and the applied force was used to determine the shear force in the shear plate connections. The MTS actuator also measured displacement parallel to the line of action of the applied force. Figure 5 shows the post-test positions of typical tests.

(a) (b) (c) (d)

Figure 5: Post-test positions for typical (a) D3ST, (b) D4ST and (c) D5ST tests, and (d) enlarged photo at deformed bolt hole (4ST shown).

The strain data was used to determine the internal forces (axial force and moment) at the point of the strain gages. For this calculation, the strain gages applied to the top and bottom flanges were used on each beam, and the strain gages applied to the webs were used for data validation. The forces and moment calculated near mid-span was extrapolated to determine the axial force and moment at the point of the connection. Extrapolation was relatively straight-forward, knowing that axial force is constant throughout the length of the beam and moment varies linearly throughout the length of the beam, starting with zero moment at the pin ended connection. Further details regarding the computation of internal forces and moments and validation of measured data are available (Lesser 2016).

For each bolting configuration, one test was conducted as a quasi-static test, where the actuator load rate was one inch (25.4 mm) per minute. Quasi-static tests were performed in an effort to compare to previous testing (Thompson 2009) and data results showed similar trends to the previous work. These tests also served as a baseline for comparison to the quasi-dynamic tests.

3.1 Three-Bolt Shear Plate Results

One three-bolt static (S3ST1) and two three-bolt dynamic tests (D3ST2 and 3) were run. Due to a data acquisition error, D3ST3 resulted in a partial data set that was used to compare forces at bolt fractures, but could not be used for full comparison of data. A force and moment versus rotation response for both static and a typical dynamic 3ST connection is shown in Figure 6. The point of zero rotation, as shown by the vertical solid line, indicates the point where the actuator is halfway through its full stroke (i.e., the extreme negative and positive rotations relate to the actuator at its full extension and full contraction points, respectively). This point was convenient for data comparison because of its consistent spatial location for all tests. The graphs show the full extent of the test, starting at the left side of the plot and ending at the right side.

The graph clearly shows that 3ST connections begin with a modest spike in moment magnitude and then exhibit a range of sustained moment resistance as beam rotation accrues. As the system reaches its maximum moment, a transition from flexural resistance to a catenary type behaviour is seen. For the static test, the measured moment in the connection increases until it achieves a net rotation of approximately 0.04 radians. The connection then transitions from flexural to significant catenary behaviour as indicated by the rapid rate of increase in axial loading starting after it achieves a net rotation of approximately 0.05 radians. For the dynamic test, the measured moment in the connection increases until it achieves a net rotation of approximately 0.05 radians and transitions to significant catenary behaviour starting after it achieves a net rotation of approximately 0.06 radians. Geometric stiffness results in a significant increase in axial force as the bending moment in the connection plateaus and declines. This continues until a bolt fractures, at which point moment reverses and the axial force immediately begins to accrue. Once the second bolt fractures, forces and moment both drop to relatively low magnitudes.

Deformation parallel to the beam axis occurred at the bolt holes in the stem of the shear plate prior to bolt fracture (see Figure 5(d)). The bottom bolt hole showed the most significant deformation, and deformation became less pronounced on every bolt hole closer to the top of the plate. A significant level of ductility was exhibited prior to bolt fracture. Prior to initial bolt fracture the 3ST specimens were found to have compression at the upper region of the shear plate and tension in the lower half, with rotation about the center bolt. Moment reversed upon initial bolt fracture due to the redistribution of load occurring within the connection.

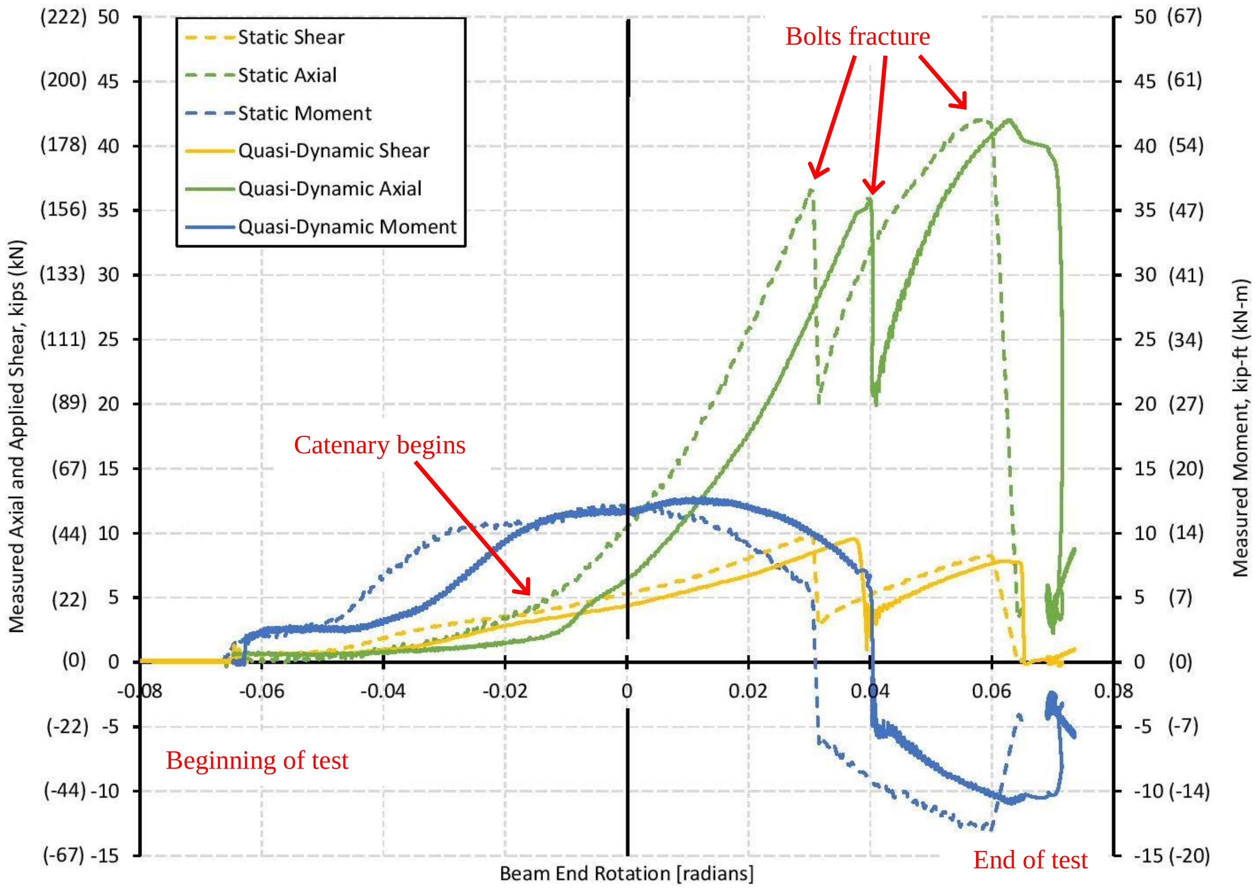


Figure 6: Response of static 3ST and typical quasi-dynamic 3ST tests.

3.2 Four-Bolt Shear Plate Results

One four-bolt static (S4ST1) and three four-bolt dynamic tests (D4ST2, 3 and 4) were run. D4ST4 used galvanized bolts, which negatively affected the strength of the connection. The pattern of results for the four-bolt tests was generally consistent with the three-bolt results. Bolt hole deformations were consistent with the three-bolt tests; however, the point of rotation was located between the two middle bolts.

A force and moment versus rotation response for both static and a typical dynamic 4ST connection is shown in Figure 7. As with the 3ST specimens, the graph shows that the connections exhibit a transition from flexural resistance to a catenary type behaviour but in the 4ST tests the axial and moment increase simultaneously. For the static test, the measured moment in the connection spikes initially and steadily increases until it achieves a net rotation of approximately 0.08 radians. Within the same range, catenary behaviour is exhibited by the rapid rate of increase in axial loading starting after it achieves a net rotation of approximately 0.02 radians. For the dynamic test, the measured moment in the connection increases until it achieves a net rotation of approximately 0.07 radians.

Transition to significant catenary behaviour starts after it rotated less than 0.02 radians, significantly less than what was observed in three-bolt tests. Geometric stiffness results in significant increase in axial force as the bending moment in the connection reaches maximum magnitude, and an earlier initiation of the geometric stiffness can be attributed to the additional bolt present in the connection. This continues until a bolt fractures, but the axial force begins to accrue again immediately following bolt fracture. The trend is repeated until one bolt remains, at which point the forces and moments drop to relatively low magnitudes.

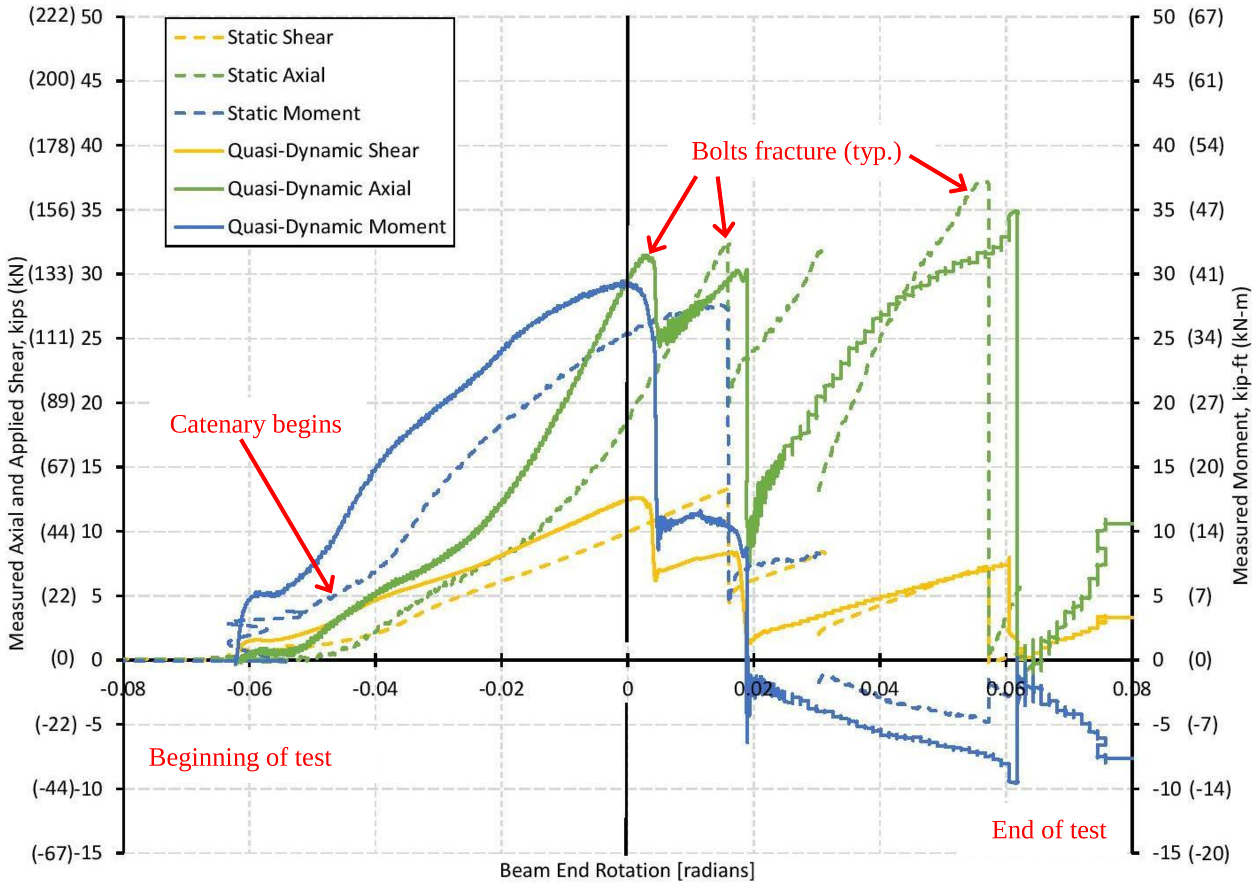


Figure 7: Response of static 4ST and typical quasi-dynamic 4ST tests.

3.3 Five-Bolt Shear Plate Results

One five-bolt static (S5ST1) and three five-bolt dynamic tests (D5ST2, 3 and 4) were run. D5ST4 used galvanized bolts, which negatively affected the capacity. The pattern of results for the five-bolt tests was generally consistent with the four- and three-bolt results. Bolt hole deformations were consistent with the other tests and the center of rotation once again gravitated toward the middle bolt.

A force and moment versus rotation response for both static and a typical dynamic 5ST connection is shown in Figure 8. As with the 3ST and 4ST specimens, the graph shows that the connections exhibit a transition from flexural resistance to a catenary type behaviour. However, for the 5ST tests the static and dynamic responses were closer together, both in terms of maximum magnitudes and for the rotation demand at maximum magnitudes.

For the static test, the measured moment in the connection spikes and increases immediately and quickly until it achieves a net rotation of approximately 0.06 radians, which coincides with the first bolt fracture. The moment curve indicates that moment magnitude is still increasing at the point of first bolt fracture. At the same time moment is accruing, axial force is also accruing and reaches maximum when the first bolt fractures. The dynamic test behaves similarly, although the moment appears to be leveling off at its maximum point, indicating that behaviour in the connection is transitioning to catenary dominant behaviour. Both moment and axial magnitudes increase between the first and second bolt fracture, but the moment magnitude quickly plateaus and slightly decreases whereas the axial force continues to increase until the second bolt fractures. Moment resistance has diminished greatly after the second bolt fracture, but axial force resistance is regained again for each subsequent bolt fracture. Moment resistance is effectively negligible following the third bolt fracture, but the system is able to develop rather significant axial forces as rotation continues. The rotation of the column stub due to unequal rotational stiffness is indicated by the differing slopes in the axial force trace after every subsequent bolt fracture.

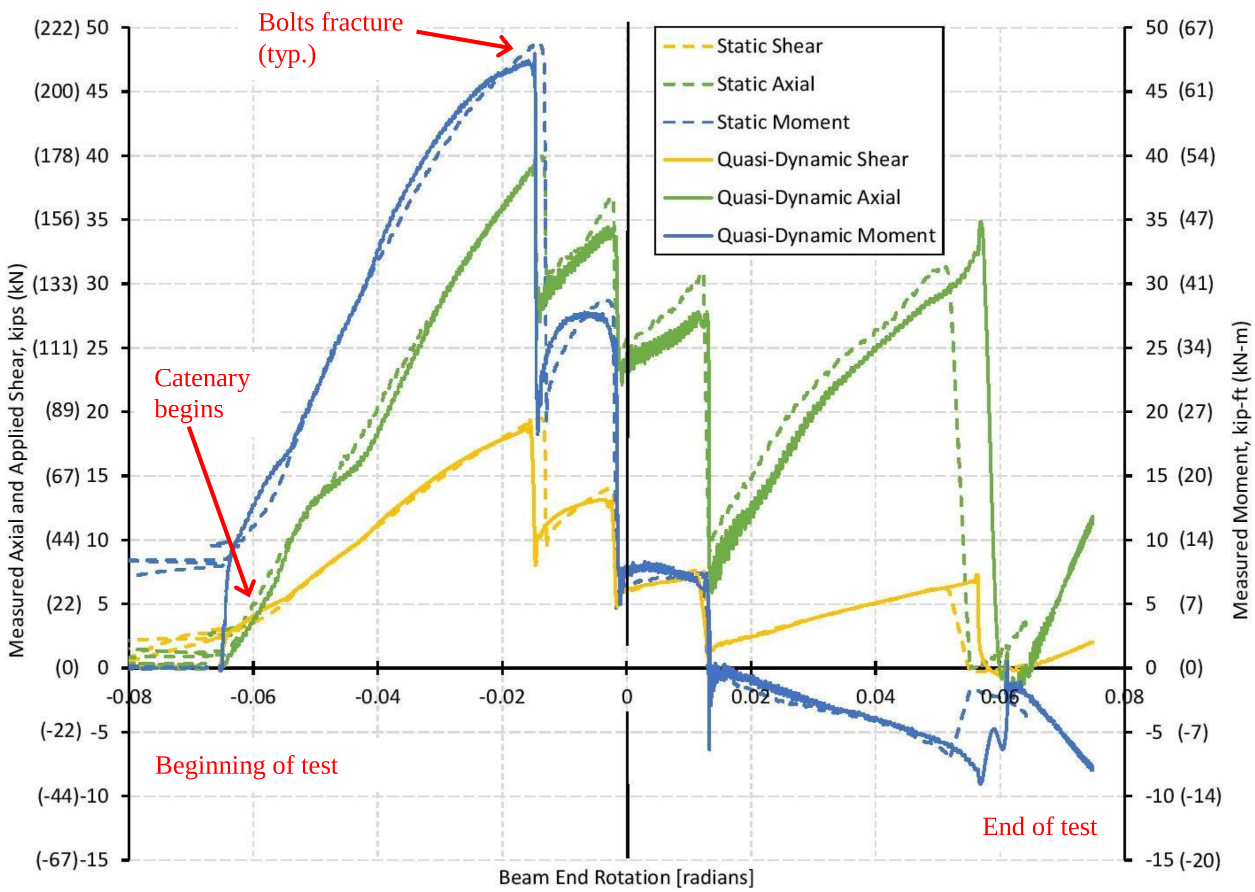


Figure 8: Response of static 5ST and typical quasi-dynamic 5ST tests.

4. DISCUSSION AND CONCLUSIONS

Eleven experimental tests (3 static and 8 quasi-dynamic) were performed and the results for six of the experiments were discussed. As was expected, a measurable magnitude of moment resistance existed in all of the shear plate connections. Also as expected, the magnitude increased as the number of bolts in the connection increased. Geometric stiffness resulted in significant axial forces in the system, and axial forces dominate the response as moment resistance plateaus and declines.

Interesting observations can be made when comparing the different configurations. The results for the 3ST configuration show an initial flexural resistance. Catenary action engaged as indicated by the sharp increase in axial force in the connection as the connection approached its point of maximum moment. Initial bolt rupture was seen in the catenary range after moment resistance had declined. Results for 4ST differ somewhat as the catenary action engaged earlier in the connection response and both flexural and axial forces increased simultaneously. Once again, the initial bolt rupture was seen in the catenary range. The 5ST configurations show a nearly simultaneous flexural and catenary engagement. Initial bolt rupture occurred at the height of flexural resistance, which is also within the catenary range.

The axial force, shear and moment for both the static and quasi-dynamic tests are very similar in magnitude at the point of each bolt fracture. However, the initial bolt fracture occurs with less net rotation in the 3ST static test as compared to the quasi-dynamic test. For the 4ST tests, the opposite trend is observed. For the 5ST, the initial bolt fractures at nearly identical net rotation magnitudes, with quasi-dynamic tests slightly preceding static tests. A parallel study of WT connections (Hayes 2016) shows similar trends for 4- and 5-bolt connections, but differs for the 3-bolt connection.

After an initial bolt fracture, the system regains axial force resistance through catenary behaviour. In the 3ST configuration, axial force resistance increases beyond the magnitude prior to initial bolt fracture. The shear force reengages, but it does not see a recovery of shear force magnitude similar to that of the axial force. In the 4ST configuration, the axial forces recover to similar magnitudes prior to each bolt fracture, but the shear force magnitudes lessen at each subsequent bolt fracture. For the 5ST, the axial forces don't quite recover to the same magnitude at each subsequent bolt fracture, and neither do the shear forces. One could point to the interaction between moment, shear and axial force in the connection as an explanation of this trend, as the 5ST has the ability to resist significantly more moment than do the other connection configurations.

Vertical displacement coupled with geometric constraint in the system induces rotational demand on the extreme upper and lower bolts in the connection. The interacting force and moment carrying capacity of the connection is limited due to the demand imposed on the extreme bolts, particularly on the lower-most bolt as it resists an additive shear force from moment and axial demand. As the system displaces, geometric stiffness (i.e., catenary) effects contribute to the load carrying capacity, but is curtailed by initial bolt fracture. Redistribution of loading was evident in each of the configurations, but the system was never able to completely recover its initial ability to resist force demands.

Robustness in a structural system is dependent on the system's inherent ductility. A ductile system has a better chance of forming alternate load paths that are necessary for structural systems to resist disproportionate collapse. A connection rotation magnitude of 0.03 radians (AISC 2010a) is commonly used in order to deem a connection as ductile. The shear plate experiments conducted exceed 0.03 radians of rotational demand, in fact effectively doubling that rotation prior to initial bolt fracture and/or exhausting moment capacity within the connection.

Sadek et al. (2008), using high-fidelity finite element analysis to study a floor system with loss of a supporting column. The connection used in the analytical study was a standard three-bolt shear tab designed using accepted procedures (AISC 2010a), so the analytical study is relevant. The results of the analytical model of the shear tab showed that the system began to lose strength at a rotation magnitude of 0.088 radians and failed at a rotation magnitude of approximately 0.14 radians. These magnitudes are of a similar order to that of the present experimental study. This comparison shows that the connections tested in this study behave similarly to those previously modeled through exhaustive analytical modeling.

ACKNOWLEDGMENTS

The authors would like to acknowledge the assistance of Dave Newman and his staff at the Marquette University Engineering Materials and Structural Testing Laboratory for their help in carrying out the experiments, and Megan Hayes for her assistance with the experiments and fruitful discussions on the data and results. The authors would also like to acknowledge the support of Germantown Iron and Steel Corp. in Jackson, Wisconsin for providing the fabricated steel used in these experiments.

REFERENCES

ACI. 2014. *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-14, American Concrete Institute, Farmington Hills, MI.

AISC. 2010a. *ANSI/AISC 360-10: Specifications for the Design of Steel Buildings (and Commentary)*, American Institute of Steel Construction, Chicago, IL.

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

Alashker, Y., El-Tawil, S., and Sadek, F. 2010. Progressive Collapse Resistance of Steel-Concrete Composite Floors. *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 136 (10), 1187-1196.

Daneshvar, H. and Driver, R.G. 2010. Application of Seismic Steel Connection Experiments to Column Removal Scenario. *2nd Specialty Conference on Disaster Mitigation*, Canadian Society for Civil Engineering, Winnipeg, Manitoba, Canada, 10 p. paper in CD-ROM proceedings.

Ellingwood, B., Marjanishvili, S., Mlakar, P., Mehrdad, S., and Williamson, E. 2009. Disproportionate Collapse Research Needs. *Structures Congress*, American Society for Civil Engineering, Austin, Texas, USA, 11 p. paper in CD-ROM proceedings.

Foley, C.M., Barnes, K. and Schneeman, C. 2008. Quantifying and Enhancing Robustness in Steel Structures: Part 2—Floor Framing Systems. *Engineering Journal*, American Institute of Steel Construction, Chicago, IL, Fourth Quarter, 267-286.

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

Geschwindner, L.F. and Gustafson, K.D. 2010. Single-Plate Shear Connection Design to Meet Structural Integrity Requirements. *Engineering Journal*, American Institute of Steel Construction, Chicago, IL, Third Quarter, 189-202.

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

Hayes, M.E. 2016. *Robustness of WT Connections under Quasi-Dynamic Loading*, MS Report, Milwaukee School of Engineering (MSOE), Milwaukee, WI.

International Code Council. 2009. *International Building Code*, International Code Council, Country Club Hills, IL.

Lesser, J.E. 2016. *Robustness of Shear Plate Connections under Quasi-Dynamic Loading*, MS Report, Milwaukee School of Engineering (MSOE), Milwaukee, WI.

Liu, J. and Astaneh-Asl, A. 2000. Cyclic Testing of Simple Connections Including Effects of Slab. *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 126 (1), 32-39.

Main, J.A. 2014. Composite Floor Systems under Column Loss: Collapse Resistance and Tie Force Requirements. *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 140 (8).

Main, J.A. and Sadek, F. 2012. *Robustness of Steel Gravity Frame Systems with Single-Plate Shear Connections*. NIST Technical Note 1749, National Institute of Standards and Technology, Washington, D.C., USA.

New York Building Code. 2008. *2008 New York City Building Code*, NYCBC, New York, NY.

Oosterhof, S.A. and Driver, R.G. 2012. Performance of Steel Shear Connections under Combined Moment, Shear, and Tension. *Structures Congress*, American Society for Civil Engineering, Chicago, Illinois, USA, 12 p. paper in CD-ROM proceedings.

Oosterhof, S.A. and Driver, R.G. 2011. An Approach to Testing the Performance of Steel Connections Subjected to Extreme Loading Scenarios. *2nd International Engineering Mechanics and Materials Specialty Conference*, Canadian Society for Civil Engineering, Ottawa, Ontario, Canada, 10 p. paper in CD-ROM proceedings.

Raebel, C.H., Foley, C.M., and Friedman, A.D. 2012. Experimental Evaluation of the Robustness of Flexible WT Steel Connections. *3rd International Structural Specialty Conference*, Canadian Society for Civil Engineering, Edmonton, Alberta, Canada, 9 p. paper in CD-ROM proceedings.

Raebel, C.H. 2011. *A Quantitative Study of Robustness Characteristics in Steel Framed Structures*, Doctoral Dissertation, Marquette University, Milwaukee, WI.

Sadek, F., El-Tawil, S. and 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), 1717-1725.

Thompson, S.L. 2009. *Axial, Shear and Moment Interaction of Single Plate "Shear Tab" Connections*, MS Report, Milwaukee School of Engineering (MSOE), Milwaukee, WI.