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EXPERIMENTAL EVALUATION OF THE ROBUSTNESS OF WT CONNECTIONS UNDER QUASI-DYNAMIC LOAD

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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 flexible WT connections. The experimental program included twelve full-scale tests of a system consisting of two wide flange beams connected to a central wide flange column stub by means of the WT connections. Three, four, and five bolt configurations were tested. The system was subjected to a quasi-dynamic loading scenario simulating the loss of a central support column. 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

Design approaches for integrating robustness into steel framed building structures are not clearly defined in many United States codes and design standards (e.g., AISC 2010a). In contrast, reinforced concrete design codes include provisions, although often prescriptive, intended to enhance inherent robustness in cast-in-place and precast concrete systems (ACI 2014). Many elements within the structural steel framework, such as infill beams, girders and flexible connections, are not intended to contribute to the inherent robustness of the overall system. However, these elements can and likely do contribute to overall robustness without direct consideration to their contributions.

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. 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 has been a topic of research interest recently, but even with recent advances the subject is not yet fully understood. Ellingwood et al. (2009) notes that the deformation capacity of elements subjected to force and moment interaction is an assumption that deserves further investigation.

Many recent research 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). Many of the research programs mentioned focus on shear tab connections due to their simplicity and relative ease in limit state identification and modeling. It is also easy to identify limit states for the flexible WT connection, but some of the limit states are more complicated than the shear tab.

The subject of the current research initiative is the flexible WT beam-to-column connection as illustrated in Figure 1. A standard connection (AISC 2010b), the flexible WT is often used as a bolted alternative to the traditionally used welded-bolted shear tab connection when the fabricator or engineer opts to minimize the need for welding. The standard WT is assumed to resist transverse shear forces (i.e., in vertical direction parallel to the line of bolts in Figure 1).

Astaneh and Nader (1989) investigated the flexibility and ductility of WT connections under rotation demand of 0.07 radians. However, little experimental data has been generated towards the understanding of the behaviour of flexible WT connections subjected to a significant magnitude of rotation demand in concert with interactive axial and shear forces. Experimental studies performed by Friedman (2009) and reported by Raebel *et al* (2012) have provided much needed experimental data and insight into the interactive force and moment behaviour of the flexible WT connection with significant rotational demand under a column removal scenario. The experiments clearly showed that the WT 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.
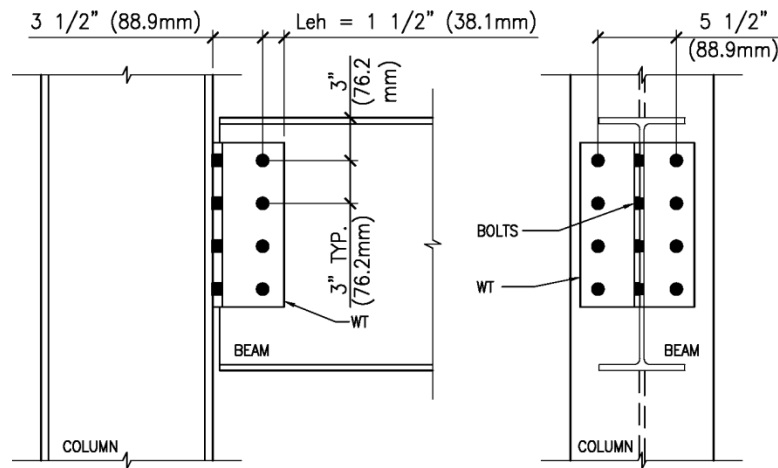

Figure 1: Flexible WT Connection Configuration.

The loading rate performed by Friedman (2009) was very slow, such that the loading could be considered static, or "quasi-static." The present research initiative investigates the behaviour of the flexible WT connection while undergoing interactive forces and moments, including significant rotation demands; however, the loading rate used in the present research is significantly faster than Friedman'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 Friedman's experiments.

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 four tests. Figure 2 shows the general layout and geometry of each WT specimen. The dimensions shown for the 4WT are typical for all specimens. The yield stress of the WT was nominally 50 ksi (345 MPa) and the modulus of elasticity was nominally 29,000 ksi (200 GPa).
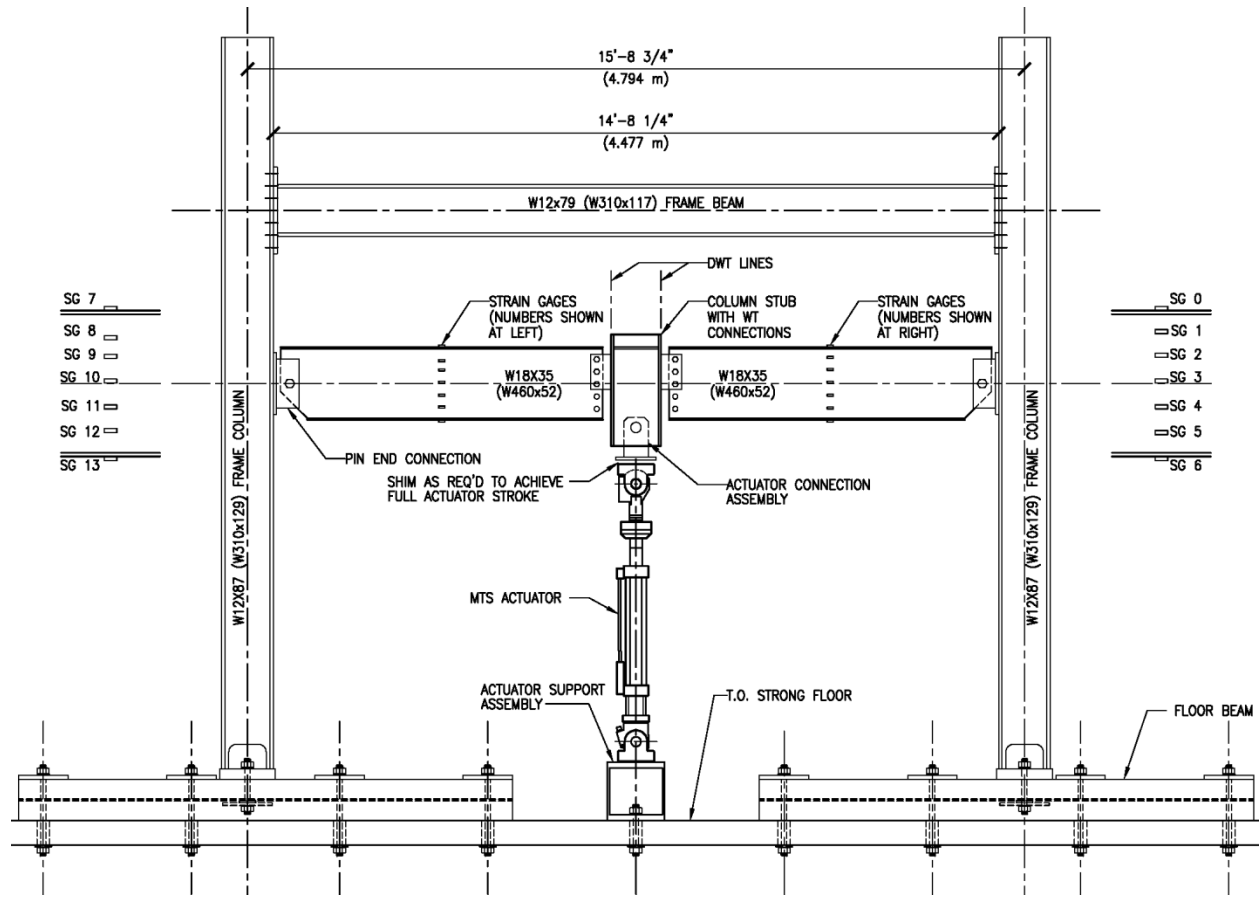
Material tests resulted in an average yield stress of 64 ksi (442 MPa) and an average modulus of elasticity of 29,237 ksi (201.6 GPa). The WT used were WT5×22.5 (WT125×33.5).

<center>(a) 3WT   (b) 4WT   (c) 5WT</center>

Figure 2: WT5×22.5 (WT125×33.5) 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 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 WT components and the bolts attaching them to the beam or column stub. Further details regarding the calculated limit states and specimen design are available (Hayes 2016; Friedman 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 WT test specimens are centered in a two-span system that connects two re-usable 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 WT 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 was collected through a customized software program, and the force and actuator displacement data was 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 WT (present work) and shear tab (Lesser 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 WTs 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, twelve 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 WT 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 WT 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) |
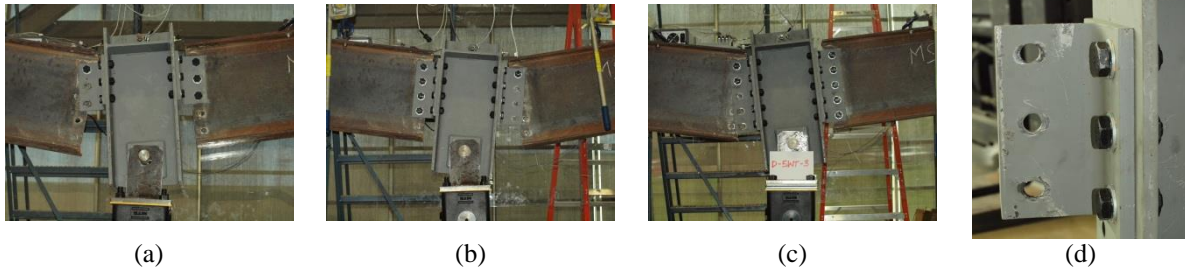
Figure 5: Post-test positions for typical (a) D3WT, (b) D4WT and (c) D5WT tests,
and (d) enlarged photo at deformed bolt hole (3WT shown).

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 (Hayes 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 (Friedman 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 WT Results

One three-bolt static (S3WT1) and three three-bolt dynamic tests (D3WT2, 3 and 4) were run. A force and moment versus rotation response for both static and a typical dynamic 3WT 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 3WT connections exhibit a transition from flexural resistance to a catenary type behaviour. For the static test, the measured moment in the connection increases until it undergoes an approximate net rotation of 0.07 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 rotates approximately 0.05 radians. For the dynamic test, the measured moment in the connection increases until it undergoes an approximate net rotation of 0.08 radians and transitions to significant catenary behaviour starting after it undergoes an approximate net rotation of 0.06 radians. Geometric stiffness results in significant increase in axial force as the bending moment in the connection declines. This continues until a bolt fractures, but the axial force begins to accrue immediately following the first bolt fracture (illustrated in dynamic test trace in Figure 6). 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 WT 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 WT. A significant level of ductility was exhibited prior to bolt fracture. The 3WT specimens were found to have compression at the upper region of the WT and tension in the lower half, with rotation about the center bolt.
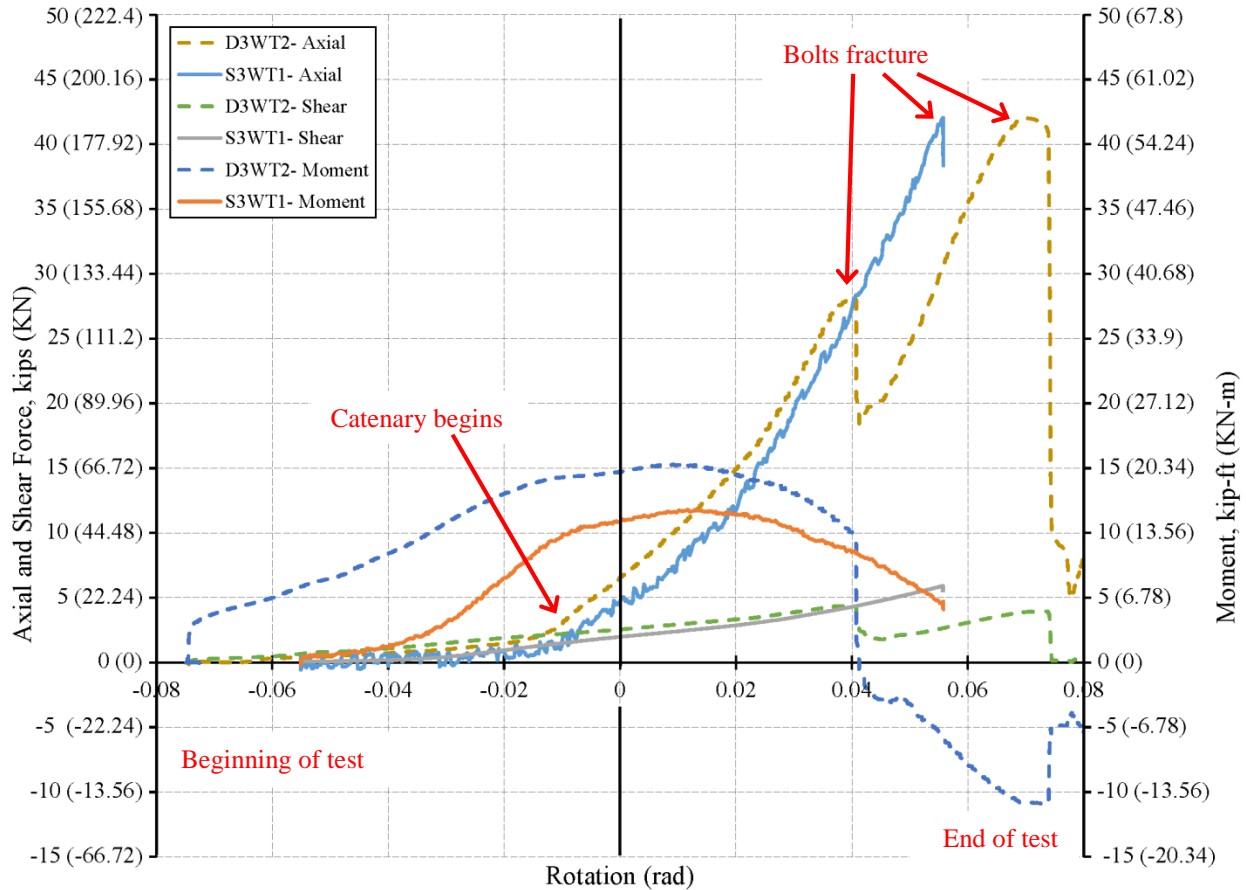

Figure 6: Response of static 3WT and typical dynamic 3WT tests.

3.2 Four-Bolt WT Results

One four-bolt static (S4WT1) and three four-bolt dynamic tests (D4WT2, 3 and 4) were run. 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 4WT connection is shown in Figure 7. As with the 3WT specimens, the graph shows that the connections exhibit a transition from flexural resistance to a catenary type behaviour. For the static test, the measured moment in the connection increases until it undergoes an approximate net rotation of 0.08 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 undergoes an approximate net rotation of 0.05 radians. For the dynamic test, the measured moment in the connection increases until it undergoes an approximate net rotation of 0.07 radians, which is slightly less than was observed in the three-bolt dynamic tests. Transition to significant catenary behaviour starts after it undergoes an approximate net rotation of 0.03 radians, half of what was observed in three-bolt tests. Geometric stiffness results in significant increase in axial force as the bending moment in the connection declines, 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. Where multiple bolts fracture, the trend is repeated until one bolt remains, at which point the forces and moments drop to relatively low magnitudes.
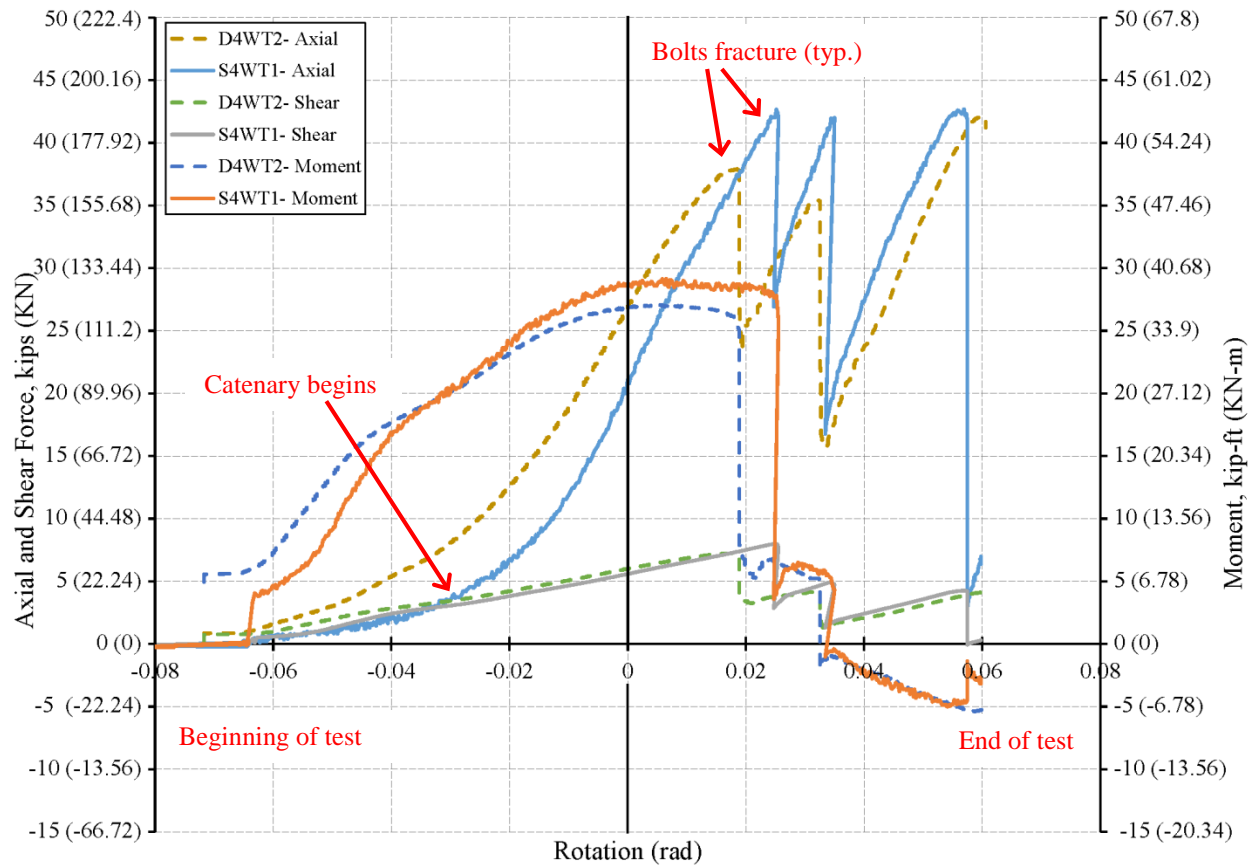

Figure 7: Response of static 4WT and typical dynamic 4WT tests.

3.3 Five-Bolt WT Results

One five-bolt static (S5WT1) and three five-bolt dynamic tests (D5WT2, 3 and 4) were run. 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 5WT connection is shown in Figure 8. As with the 3WT and 4WT specimens, the graph shows that the connections exhibit a transition from flexural resistance to a catenary type behaviour. However, for the 5WT 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 increases until it undergoes an approximate net rotation of 0.07 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 levels 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 whereas the axial force continues to increase until the second bolt fractures. Moment resistance is lost after the second bolt fracture, but axial force resistance is regained again for each subsequent bolt fracture. 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.
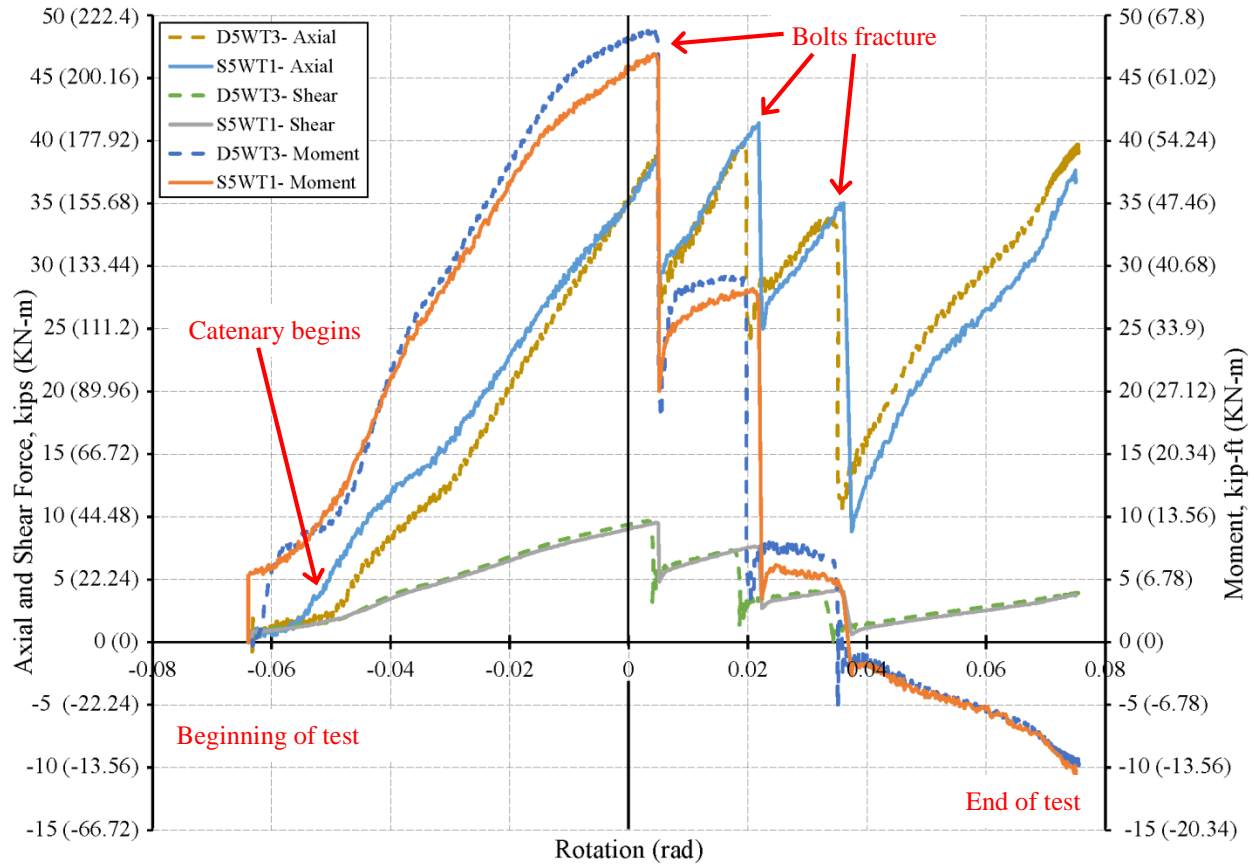
Figure 8: Response of static 5WT and typical dynamic 5WT tests.

4. DISCUSSION AND CONCLUSIONS

Twelve experimental tests (3 static, 9 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 WT connections, and the magnitude increased as the number of bolts in the connection increased. For a period of time, flexural resistance dominated the load carrying capacity of the system. Geometric stiffness resulted in the realization of significant axial forces in the system, and axial forces dominate the response as moment resistance plateaus and declines.

The results for the 3WT configuration show an initial flexural resistance. As the connection neared its flexural resistance limit, catenary action engaged as indicated by the sharp increase in axial force in the connection. Initial bolt rupture was seen in the catenary range after moment resistance had declined. Results for 4WT show a similar trend; however, the catenary action initiated earlier in the connection response when the flexural resistance was still accruing. Once again, the initial bolt rupture was seen in the catenary range. The 5WT configurations show a nearly simultaneous flexural and catenary engagement. Initial bolt rupture occurs at the height of flexural resistance, which is also within the catenary range.

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. 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 in no case was the system able to completely recover its ability to resist force demands.

Experimental results show that the initial bolt fracture occurs with less rotation demand when the connection undergoes a quasi-dynamic load as compared to a quasi-static load. This is clearly evident with the 3WT and 4WT results. However, as more bolts are added to the connection, the difference in rotational demand closes as is evident in the 5WT results. This is largely due to the ability for the 5WT connections to better resist moment demand.

An interesting observation can be made for all configurations. After an initial bolt fracture, the system regains axial force resistance through catenary behaviour. In the 3WT configuration, axial force resistance increases beyond the magnitude prior to initial bolt fracture. Although the shear force reengages, it does not see an increase similar to that of the axial force. In the 4WT configuration, the axial forces recover to similar magnitudes prior to each bolt fracture. However, the shear force magnitude lessens at each bolt fracture. For the 5WT, 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.

A key factor in providing inherent robustness in a structural system is ductility. A ductile system has a better chance of forming alternate load paths that are necessary for structural systems to resist disproportionate collapse. AISC (2010a) typically uses a rotation magnitude of 0.03 radians as a rotation capacity for a connection identified as ductile. The WT experiments conducted herein well exceed that rotational demand and effectively double the stated rotation capacity prior to initial bolt fracture and/or exhausting moment capacity within the connection.

A relevant analytical study was conducted by Sadek *et al* (2008), using high-fidelity finite element (FE) analysis to study a floor system with loss of a supporting column. Although the connection used in the analytical study was a standard three-bolt shear tab designed using accepted procedures (AISC 2010a), the 3WT connection considered in the present study behaves in a similar manner to the three-bolt shear tab because the stem of the WT is of similar thickness and dimensions. 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.

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