Review of Research on the Fire Behavior of Simple Shear Connections

Erica C. Fischer, Oregon State University, Corvallis, OR, USA, erica.fischer@oregonstate.edu
Rachel Chicchi, University of Cincinnati, Cincinnati, OH, USA, rachel.chicchi@uc.edu
Lisa Choe, National Institute of Standards and Technology, Gaithersburg, MD, USA, lisa.choe@nist.gov

Abstract:
Previous building fires and research has shown that simple (shear) connections develop large axial force and flexural demands during a fire leading to potential failure of the connections and progressive collapse of a building. These findings have motivated decades of international research on the fire behavior of simple (shear) connections. While this research all confirmed that simple connections experience large axial force and flexural demands, it occurred without consistency of testing methodology, testing setup, and reporting of results. Due to this inconsistency, the decades of research have yet to be synthesized into comprehensive code changes for the design of simple connections in fires. This paper will summarize international experimental and numerical research on simple (shear) connections and highlight the inconsistencies between this research in an effort to motivate consistent testing and data reporting for future research developments.

1. Introduction and state-of-the-practice

Steel simple (shear) connections resist forces and moments during a fire that they are not designed for at ambient temperature, leading to damage or failure of the connections. This concept has been explored during investigations of building fires [1] and full-scale fire tests [2] and motivated decades of international research on the fire behavior of simple connections with the aim to better understand these demands. This research has led to international consensus that simple connections will experience force and flexural demands during a fire they are not designed for at ambient temperature. However, the research on the fire behavior of simple connections that led to this consensus is not consistent with respect to testing methodologies, setups, and reporting of data collected during the experiment. Therefore, code [3-6] adopted methodologies developed to protect steel-frame buildings in fires have not been validated against experimental tests and trends in the testing data are not consistent. This paper will summarize code adopted methodologies in the US and Europe for the design of simple connections in fire scenarios, identify the inconsistencies in testing methods employed by researchers around the world for the fire behavior of connections, and identify the inconsistencies in reporting of data and measurements by researchers around the world for fire behavior of connections.

Codes and standards within the US and Europe [3-6] address connection capacities during a fire in two ways. First, is the temperature-dependent shear and tensile capacities. In the US, the shear and tensile capacity of the connections are calculated using the ambient connection design methodologies of the AISC *Specification* Chapter J [3] with temperature-dependent material properties of the connection materials as calculated through AISC *Specification* Appendix 4. In Europe, specific design equations and strength reduction factors are provided in Annex D of Eurocode 3, Part 1-2 [6]. Annex D also recognizes that during a fire there will be a thermal distribution through the connection and the bottom flange of the connection region will be 88% of the temperature of the bottom flange at the beam midspan. However, this thermal gradient has not been benchmarked against experimental data. The second way with which codes and standards address connections during a fire is through integrity provisions. All of the codes and standards [3-6] stipulate that integrity provisions should be met at ambient temperature. Structural integrity provisions are minimum strength criteria to ensure connectivity of members within a building. Conformance of these provisions provides structural integrity for normal service conditions and minor unanticipated events throughout the lifetime of the structure.

The aforementioned provisions vary greatly by code. ASCE 7 [4] stipulates that a mechanical connection between a girder and a column must resist a force acting parallel to the girder. This force is equal to 5% of the unfactored dead load plus live load imposed by the girder onto the column. The AISC *Specification* [3] Chapter B stipulates that connections between members must resist an axial force equal to two-thirds of the required vertical shear strength for design. When the end connections are used for bracing a column, the end connection shall have a nominal tensile strength equal to 1% of two-thirds of the required column axial strength (P_u). The New York City (NYC) Building Code [5] modified the AISC *Specification* provisions to be stricter. It specifies that end connections must be designed to resist a minimum available tensile strength equal to the larger of the available shear strength of the connection or 45 kN (10 kips). The US codes and standards [3-5] provide quantifiable minimum design strengths for end connections. In contrast, Eurocode 3 [6] stipulates that the fire resistance of a bolted or welded joint is assumed to be sufficient if the demand-to-capacity ratio of the joint is less than or equal to the demand-to-capacity ratio of any of the connected members. All four of these codes and standards also stipulate that the connections should be designed for the demands imposed on them throughout a fire scenario. However, these documents do not provide guidance on how to quantify those demands without a detailed finite element model of the building.

2. Paper objectives

The fire behavior of simple connections has been researched internationally, particularly on US connection types such as shear tab, single-angle, double-angle, and end-plate connections. This research includes both numerical modeling [7-13] and experimental testing [10, 14-23]. This research has concluded that: (1) large axial force demands (both compression and tension) develop in connections throughout a fire causing damage or failure of the connections, and (2) simple connections with ductility perform better during a fire and will therefore contribute to the overall performance of steel-frame buildings in fires. These simple connections are only designed for shear at ambient temperature, yet during a fire will experience axial force demands (compressive and/or tensile) and potentially imposed moments. Ductility of the connection can enable catenary action of the steel or composite beam; therefore, improving the overall performance of steel-frame buildings during a fire.

Regardless of the common conclusions, researchers have tested and modeled connections in a variety of different ways. These different testing methods differ with respect to test setup, exposed fire scenario, testing in steady-state versus transient fire exposures, and inclusion of a concrete deck. In addition to different testing methods, researchers measure and report different parameters to characterize the connection behavior (e.g. connection rotation, temperature distribution, axial force demand in the connection). These inconsistencies make quantifying the fire behavior of connections using previous research very difficult. A comprehensive literature review was performed by da Silva et al. [24] to summarize the experimental work conducted prior to 2005. Temperature distributions within the joint were also presented, as well as typical failure modes.

The objectives of this paper are to: (1) summarize previous research on simple connections and characterize the research with regards to testing methodology, testing setup, and measured and reported results; (2) examine the variability of characteristics with regards to simple connection research; and (3) demonstrate that due to variability in the characteristics of simple connection research, the building design industry is unable to benchmark code provisions or develop comprehensive design solutions that prevent or limit connection failure during a fire. This paper expands upon the previous study [24] and suggests a need for conformance of testing and reporting test results of steel connection behavior in a fire scenario.

Section 3 of the paper summarizes a comprehensive literature review of experimental and numerical studies of simple shear connections. The research studies in the literature review are characterized with respect to research characteristics such as testing setup, testing methodology, and measured and reported parameters. Section 4 synthesizes the literature review to actively compare research characteristics discussed within the literature review. This section will group the research together to highlight the variability of research characteristics. Finally, the authors will make a call for action for testing of simple connections in a uniform manner such that design solutions can be developed for resilient steel-frame building design.

3. Summary of previous testing of connections

3.1 Component-level testing

Yu et al. [18] and Hu and Engelhardt [10] used stub beam and column assemblies to test connections at steady-state heating. These tests isolated the connection from the rest of the frame, testing only a singular component of a steel frame building. These tests resulted in axial load-deformation capacities and limiting failure modes of varying connection configurations at varying temperatures. Yu et al. (2009) performed a series of experiments to obtain the moment-rotation behavior of simple shear connections. Stub beams (UB305x165x40) were connected to stub columns (UC254x89) with fin plate (shear tab) connections. All of the specimens were tested in a furnace with an inclined tensile force applied to the beam. The angle to which the load was applied represented a predetermined ratio of shear to tying force demand on the connection. Fourteen specimens were tested with varying bolt diameters (20 mm and 24 mm), grade of bolt (Grade 8.8 and 10.9) and number of bolts in the connection (single column of three bolts or two columns of three bolts each). A composite slab was not included in the test setup and the specimens were tested at steady state temperatures of 20ºC, 450ºC, 550ºC, and 650ºC, therefore, there was no thermal gradient through the connection. The test results showed that fin plate (shear tab) connections can have a moment capacity throughout a fire that can contribute to increasing the survival time of a building. All of the tested fin plate connections failed in bolt shear fracture at elevated temperature. Yu et al. [18] concluded that reduction in connection strength with increasing temperature was highly dependent upon the temperature-dependent retention factors (reduction factors) of bolt shear strength regardless of the ambient temperature design. Tests with increased bolt strength resulted in the ambient controlling limit state as block shear whereas the elevated temperature limit state remained bolt shear fracture.

Hu and Engelhardt [10] conducted numerical modeling and experimental testing of single-plate (shear tab) connections in tension to explore the behavior and failure modes during heating and cooling phases, as well as forces and deformations developed in connections. The experimental testing was similar to that conducted by Yu et al. [18], but mechanical loads were only applied in the direction longitudinal to the beam (not at an angle). Stub beams with three 19 mm (¾”) ASTM F3125 (grade A325) bolts and 9.5 mm (⅜ inch) thick shear tabs were used. Each specimen used a W12x26 beam, 254 mm (10 in.) in length. A composite slab was not included in the test setup, and connections were tested at steady-state steel temperatures of 20ºC, 400ºC, 500ºC, 550ºC and 700ºC. Bearing failure of the beam web was the controlling failure mode at ambient temperature. At 400ºC, tear out of the web controlled but bolt shear deformation was also observed. At temperatures above 400ºC, the failure mode changed to bolt shear fracture with little deformation of the bolt holes. Peak test loads and failure modes were reported as well as the axial load-displacement curves of the tested connections, which were found to be comparable with the results from finite-element models.

Spyrou et al. [25 26] evaluated the tensile and compressive zones of extended end-plate connections for a beam in a beam-to-column connection. This research also included development of mathematical models to predict failure modes such as prying leading to plastic hinges or bolt fracture (under tension) and crushing or buckling (under compression). Component tests were also performed with an electric furnace to study if component tests can predict failure modes and minimize the need for more complex assembly testing which includes moment-rotation-temperature-thrust relationships. A total of 29 T-stub tests were conducted using different column sections to test the capacity of the tension zone (UC152x30, UC203x46, UC203x71, and UC203x86). To test the capacity of the compression zone of end-plate connections, Spyrou et al. [25] tested 45 specimens with varying thicknesses of plates to mimic the presence of a column section and varying the diameter of the bolts (12 mm and 20 mm). These tests occurred at steady state temperatures ranging between 500-700ºC. In all of the compression and tension tests, the dimensions of the T-stub remained the same throughout the testing series.

In all of these component-level tests [10, 18, 25, 26], the reported results are axial load-displacement curves and limit states at varying temperatures. However, these tests were performed at a constant rotation and temperature, thereby removing variables such as fire scenario, temperature distribution through the connection, and rotation of the connection throughout a fire scenario.

3.2 Cruciform Assembly Tests

Lawson [14] experimentally tested eight specimens in a cruciform assembly, including two double-angle connections, two extended (“rigid”) end plate connections, and four flush (“semi-rigid”) endplate connections. Half of the specimens were protected with 60 min. equivalent fire protection and the other specimens were unprotected. For the double angle tests, one specimen had a concrete flat slab and the other had a composite slab on metal deck and both beams were UB305x165x40. Stub beams with a length of 1280 mm (50.4 in.) were connected to a column in a cruciform shape. All beam-to-column connections were subjected to the ISO 834 [27] time-temperature curve. Loads were applied to the beam ends to simulate realistic flexure and shear demands in the beams. Both of the tests using double-angle connections exceeded their design fire resistance and withstood rotations in excess of 0.1 rad. Seven of the eight tests exceeded their target fire resistance and no bolt or weld failures occurred. Time versus rotation curves were reported as well as temperature versus time with temperatures recorded at the bottom flange, bottom bolt, top bolt, and top flange. Temperatures in the bolts were much lower than the bottom flange beam temperature, minimizing bolt deformations but leading to noticeable deformations in the angles or endplates.

Al-Jabri et al. [16] performed five series of experimental tests in a cruciform arrangement. Twenty end-plate connections with flush (partially restrained) and flexible (simple) conditions were tested in a furnace. These tests followed the test program conducted by Leston-Jones et al. [15] which tested smaller sections (UB254x102x22 beams to UC152x152x23 columns) using full-depth end-plate connections. Specimens with and without slabs were tested. Al-Jabri et al. [16] expanded the study to consider other member sizes and connection types. The cantilevered beams were 1.9 m long and the column was 2.6 m tall. Specimens were loaded at a constant load level and the furnace temperature was increased until failure. Test series 1 and 2 consisted of flush end-plate connections, no fire protection on the steel and UB254x102x22 and UB356x171x51 beam sections, respectively. Test series 3 consisted of flexible end-plate connections, no fire protection on the steel, and UB356x171x51 beam sections. Test series 4 and 5 consisted of flexible end-plate connections with a composite slab, and UB356x171x51 and UB610×229x101 beam sections, respectively. The results from the tests showed that specimens in test series 1 experienced localized deformation at the top of the end-plate, column web buckling, and column flange deformation. Specimens in test series 2 experienced end-plate similar end-plate deformations without column or beam damage. Cracks were observed along the weld at the end-plate. Significant end-plate deformations were observed in the tested specimens of test series 3-5. Large cracks in the concrete originating at the face of column flange resulted in a loss of stiffness of the connection. The presence of the slab acted as insulation and a heat sink to reduce the beam top flange temperature by 20-30%. Moment-rotation curves were generated for each test up to 600°C. The bare steel specimens exhibited minimal degradation of the moment-rotation capacities at temperatures below 400°C, and pronounced degradation at temperatures greater than 400°C. For specimens within testing series 4 (pinned connection with composite slab), degradation was most notable beyond 500°C, though it was less than the bare steel conditions. For moments up to 30% of the moment capacity of the connection, the rotation under elevated temperatures was approximately the same as ambient conditions; this was due to the concrete slab primarily carrying the loads at low load levels. Less degradation was observed in test series 5 because of the large member sizes and the presence of the concrete slab.

The results from all of these cruciform tests included temperature distributions through the connection throughout the test and temperature and connection rotation histories. These tests did not, however, include reported data on the axial forces in the connection throughout the fire exposure. The test specimens and setups for these experiments performed by Lawson [14] and Al-Jabri et al. [16] included a concrete deck. However, these tests were not performed on a frame thereby the testing setup was required to mimic the restraint a frame might impose on the thermal elongation of the beam.

3.3 Frame Tests

Many researchers have investigated the fire performance of simple shear connections as a part of a beam-column frame [8, 9, 11, 13, 17, 20-23]. These studies include both experimental and numerical investigations.

Liu et al. [17] experimentally investigated the behavior of a one-bay gravity frame subjected to varying levels of axial restraint. Axial restraint was applied to the adjacent bays of the gravity frame with braces of varying axial stiffness. Both end-plate and double-angle connections were tested. No slab was used in testing and the beams were UB178x102x19. Two actuators applied point loads to the beam at approximately the third points of the span such that the beam was in four-point bending. These loads were applied at different load ratios of 0.3, 0.5, 0.7 based on the flexural capacity of the beam at ambient temperature. The furnace time-temperature curve closely modeled the ISO 834 [27] curve. Results included bottom flange temperature versus mid-span deflection and hogging (negative) moment versus bottom flange temperature. The double-angle connections resisted moments less than 15% of the moment capacity of the beam until the gap (8 mm) between bottom flange and the column flange closed. At this time the end moment increased to about 18 kN-m (13.3 kip-ft). Local buckling of the beam near connections was not observed as the test ended shortly after the bottom flange of the beam came into contact with the column flange, and bolt hole deformation was observed in the beam web. Axial restraint in the frame was found to reduce run-away deflections. The ability of the beam to support itself through axial tension during large deflections was observed during tests using end-plate connections. The run-away deflections slowed as the axial force in the beam switched from compression to tension. However, this behavior was not observed in the specimens using a double-angle connection and Liu et al. [17] contributed this behavior due to slipping of the bolts in the bolt holes.

Garlock and Selamet [8] conducted a numerical study on failure modes of the beam-to-girder shear tab connection used in the Cardington test [28] on a 6 m (19.7 ft) by 9 m (29.5 ft) composite floor assembly. The floor assembly was simulated using a commercially available finite element software where solid elements were used for steel members and connections. One-dimensional linear spring elements were implemented to model restraints provided by the surrounding structure and the floor slab. No weld failure was assumed in the model. Modeling results were validated against experimental test data. A parametric study was performed to evaluate the effects of fire characteristics on the connection behavior. Four different fires were used in the study including both the experimentally measured and design fires (with an opening factor of 0.04) used in the Cardington test as well as the Eurocode parametric fires simulating fast and slow fires with opening factors of 0.1 and 0.01, respectively. Results showed that, regardless of fire scenarios, the shear-tab connection reached consistent failure modes. Web and flange local buckling both occurred at beam ends during the early heating phase with steel temperatures reaching approximately 140–370°C. The maximum compressive force (at the onset of flange local buckling) was about 700 kN (157 kip). Excessive bolt hole elongation resulted due to tensile forces of 400 kN (90 kip) during the controlled cooling phase when the average beam temperature was approximately 110–200°C. The connection behavior (forces, deformations, and temperatures at failure) were affected by high-temperature flexural stiffness and ductility of a composite floor slab; however, the concrete deck section and slab reinforcement were not modeled explicitly in this study.

Selamet and Garlock [9] further examined the factors influencing the high-temperature behavior and strength of shear-tab connections using the same model developed in Garlock and Selamet [8]. A total of nine models representing a beam-connection assembly were developed by varying connection details, such as bolt grade, bolt pretension, the geometry and location of bolt holes, the thickness of connecting elements, and the gap distance (beam setback). The results demonstrated that the ductility of connections can be enhanced by increasing the gap distance between the beam bottom flange and the girder web. Increasing the gap distance by a factor of two improved the connection ductility by approximately 40%. Oversize and short-slot holes did not appear to have a significant effect on the connection performance relative to standard holes. Pretensioning bolts and adding doubler plates to the beam web were both found to increase the tensile capacity of fire-exposed connections. In particular, when the combined thickness of the beam web and doubler plate is equal to the shear tab thickness, the tensile capacity was approximately 1.75 times greater than the capacity of the beam web alone. However, the use of lower grade bolts or increasing the distance of a bolt hole group relative to the beam end could lead to other failure modes such as bolt shear. The study suggested a simple equation to estimate the bolt hole bearing capacity as a function of summation of unequal tensile forces in bolts due to local buckling. However, the methods and observations made in this study are limited to the connection performance governed by bolt shear or tearout failure, ignoring potential weld shear at elevated temperatures.

Wang et al. [19] conducted an experimental study on 2 m (6.6 ft) long UB178x102x19 beams with various beam-to-column connections, including fin-plate (shear-tab), end plate, and web cleat (all-bolted double-angle) connections. The bare steel frames were exposed to ISO 834 [27] fires using a furnace. A concrete slab was not included within the specimens, but the top flange was protected in order to simulate the heat sink provided by the slab. The top flange of the beam was also stiffened to mimic lateral restraints from the floor slab. The beam assembly was loaded to 40% of the ambient plastic capacity using a four-point bending scheme. This study also evaluated the effect of axial restraints with two different sizes of support columns (UC152x152x23 versus UC254x254x73). The experimental results showed that for all ten specimens, rapid displacement rates were developed at 700°C or higher.

The midspan vertical displacement of the beam reached in the range of span/8 to span/6 without collapse for all tests except the specimen with flexible end plates connected to a larger column. Complete fracture of the beam web occurred for this end plate specimen. Shear tab and double-angle connections resulted in a maximum compressive force of approximately 20 kN (4.5 kip), which was about half of the vertical shear force of the connections. Plastic hinging (local buckling failure) formed in columns at the connection location. The compressive axial force was four to six times greater when attached to a rigid column; however, no compression failure in connections was observed. The specimen with shear tab connections failed by weld fracture in tension (45 kN, 10 kip at very large displacements) when the beam temperature reached over 750°C. The double-angle connection exhibited superior ductility and robustness (strength) at elevated temperatures relative to the other connection types. This study omitted the influence of the flexural stiffness of the concrete slab on the connection rotations. The results did not include reported temperature distributions through the cross section, and the configuration of connections (e.g., bolt patterns and aspect ratio of connection plates) are not commonly used in construction practice. Lastly, the connection performance during cooling was not discussed. More importantly, experimental errors associated with mechanical and fire loading were present but not discussed how these errors influenced the behavior and failure modes of connections.

Pakala et al. [20] conducted experimental testing of beams and connections within a one-bay frame. Two W12x30 filler (secondary) beams with a length of 3505 mm (138 in.) were constructed for each test. Two tests were conducted: one included a corrugated steel deck (without concrete) and the other was a composite slab on metal deck. The connections were double-angle connections with three 22 mm (⅞ inch) diameter bolts. Each test was subjected to the ASTM E119 [29] fire time-temperature curves for 75 and 90 min, respectively, before cooling at rates of 8oC/min and 15°C/min. Two point loads were applied at each beam, separated at the centre with a distance of 860 mm (34 in.). Loads were applied at 40% and 50% of the calculated ambient ultimate moment capacity of the beams. Beam end rotation versus time and strain versus time were reported. Strain was measured at the top flange, web, and bottom flange. Axial forces in the beam and composite floor slab were also reported as a function of fire exposure time. Local buckling of the top flange and web of the beam without a concrete slab was reported. These deformations resulted in large deflections, leading to, large connection rotations and lateral torsional buckling of the beam. Significant deformations of the double-angle connections were observed; however, no failure of the connections occurred. Deformations in the connections were less than the first test without concrete slab. The tested double-angle connections demonstrated an ability to transfer fire-induced moments and withstand large rotations of approximately 0.1 rad and axial forces of approximately 400 kN (90 kip) in compression and approximately 200 kN (45 kip) in tension without a prominent failure occurring.

Pakala and Kodur [11] performed a numerical investigation into the fire behavior of double-angle connections within a one-bay frame (UB 178x102x19 beam, 2 m in length with UC254x254x73 columns). The analysis excluded the influence of a composite concrete deck. The study used varying loading ratios (30-70%) on the beam and the fuel load (510, 750, 725 MJ/m^2) also varied. The investigation concluded that the maximum compressive axial force in the connection does not vary with increasing load ratio and remains about 80 kN (18 kip). However, the tensile axial force in the connection increases with increasing load demands on the beam varying from 40 kN (9 kip) for a beam with a 30% load ratio to about 96 kN (21.6 kip) for a beam with a 100% load ratio. In addition, increasing load demands on the beam increases connection rotations during the fire exposure with a maximum rotation of about 0.03 rad for a beam with a 100% load ratio. The results of the simulation showed that the difference in fuel loads does not change the magnitude of the axial in the connection, rather the time the maximum axial force occurs during the fire exposure. However, an increase in heating duration (with increasing fuel loads) will increase the connection rotation. Lastly, Pakala and Kodur [11] showed that modeling the connection within a frame is the only way to estimate the restraint on the connections from the columns within a reasonably range that has been reported in research.

Fischer et al. [21] tested seven composite beams (3.8 m in length) with simple connections and a flat slab to investigate the behavior of the connections during a heating and cooling protocol. The beam sizes tested were W10x17, W10x22, and W12x22 beams. The connection type (shear tab, all-bolted single-angle, all-bolted double-angle, and all-welded double-angle) and applied loads were varied in the tests. Data collected during the experiments included temperature distributions through the cross section of the composite beams and along the length of the composite beam, displacement measurements of the beams, and rotation of the end connections. This data was used to benchmark numerical analyses [30], where supplemental data could be measured (e.g. axial forces in the connection). The tests showed that large axial force demands (compression and tension) develop in the connections throughout a fire scenario with a cooling phase. The maximum axial compression force in the connection can be upwards of 3.7 times the shear demand in the connection and the maximum axial tensile force can be upwards of 1.8 times the shear demand. The large tensile demands causing localized damage in the connections. Bolt shear deformation, bolt hole elongation, and shear tab fracture were all observed during the tests. Connections with ductility (angled connections) resulted in less axial force demands in the connection. End connection rotations of up to 0.08 rad were reported from the tests. Due to limitations of the laboratory, the beam length was shorter than what would be in a real building and the columns of the testing frame were larger than typical gravity columns.

Fischer and Varma [13] performed a parametric study on 7.6 m long W12x22 composite beams connected to W12x68 columns in one-bay and three-bay frames to investigate the influence of ductile connection detailing per AISC *Specification* [3] on the axial force demands in connections during a fire. Each of the models incorporated a composite deck either through a flat slab or slab on metal deck. The parameters considered were angled connections (single-angle and double-angles), short slotted holes per AISC *Specification* [3], continuous rebar around the column, hourly rating for fire resistance, and fire exposure (design-basis fire and ASTM E119 [29]). The results of the simulations showed that increasing the thickness of fire protection eliminates the amount of damage observed in the connection after the fire. All of the composite beams with simple shear connections that were unprotected (no fire protection) or had an equivalent thickness of fire protection to a 1 hr fire resistance rating (FRR) had bolt shear fracture and/or shear tab fracture during the cooling phase of the design-basis fire with cooling. In addition, coped bottom flanges and slotted holes did not prevent damage to the connection but prevented failure of the connection during a design-basis fire with cooling. Lastly, more ductile connection detailing (e.g. slotted holes, coped bottom flange, angled connections) decreased the axial compression and tension demands in the connection during the fire.

Choe et al. [22, 31] conducted a series of compartment fire tests on 12.8 m (42 ft) long composite floor beam assemblies with varying end support conditions. Test variables included two types of simple shear connections (shear tab and welded-bolted double-angle connections) and the presence or absence of slab continuity over girders. All four specimens were hydraulically loaded with a total gravity load equivalent to the ASCE 7 [4] load combination (1.2×dead load + 0.5×live load) for extraordinary events and exposed to natural gas-fueled compartment fires with the heat release rate of 4000 kW. Each specimen was constructed as a partially composite beam, consisting of a lightweight concrete slab cast on 76 mm (3 in) deep trapezoidal steel decking and a W18×35 steel beam insulated for a 2-hour fire resistance rating. Either two angles or a single plate were connected to the beam web using three 19 mm (¾ in) diameter structural bolts and attached to the column flange using 8 mm (5/16 in) fillet welds. The specimens with double-angle connections had a coped bottom flange. Two specimens included the slab continuity achieved by #4 reinforcing bars and welded wire fabric fixed at the girder locations. The measured lateral stiffness of support columns at the location of connections was about 180 kN/mm (1028 kip/in). The test results indicated that all four specimens exhibited local buckling in the beam near the connection region due to a large compressive force of 700–1000 kN (156–225 kip) when the beam temperature reached 400–500°C and the midspan displacement was about span/110. The local buckling capacity of the specimen with double-angle connections was about 25% smaller than that with shear-tab connections due to the coped bottom flange. The slab continuity increased the local buckling capacity by 7% for the specimens with shear-tab connections but did not influence the local buckling capacity of the specimens with double-angle connections. Double-angle connections achieved large ductility (allowing the midspan beam deflection of span/20) during heating but failed by weld fracture in tension during cooling. Shear-tab connections failed in combined tension and moment during heating at the beam temperature of 700–800°C and midspan beam deflection of span/22. The slab continuity for these specimens influenced failure modes (weld shear for the specimen without slab continuity and bolt shear for the specimen with slab continuity). The measured tension capacity was 85–107 kN (19–24 kip) similar to the applied vertical shear of 98 kN (22 kip). However, the end moment induced in the connection was not measured.

Kordosky et al. [23] tested two composite floor specimens that are typical of North American construction with wide-flanged beams (W12x26) supporting a slab on metal deck. One specimen used spray applied fire protection material thickness equivalent to a 2hr fire resistance rating [32], while the other had exposed steel framing. Both specimens were tested in a gas-fire furnace and exposed to the ASTM E119 standard fire [29]. The beam span was 3.34 m (10'-11.5") and the beam-to-column connections were bolted shear tab connections where the shear tab was bolted to the steel beam using three 19 mm (¾ inch) diameter bolts and welded to the column. Each specimen was loaded in four-point bending where a 158 kN (35.5 kip) point load was applied at the ⅓ points to mimic a moment diagram similar to a distributed loading scenario. This applied load along with the self weight of the beam resulted in a 60% utility factor at ambient temperature. Thermocouples were used to measure the temperature distribution through the cross section of the specimens at multiple locations along the length, and through the depth of the shear tab connections. String potentiometers were used to measure the deflection of the specimens throughout the standard fire exposure. Observations after the tests showed that all bolts within the connections experienced permanent shear deformation and the bolt holes in the beam web showed signs of warping; however, did not fail and maintained load carrying capacity. The protected beam showed limited local buckling, whereas the unprotected beam showed more significant out of plane deformation of the beam. Both beams experienced midspan deflections well in excess of the limiting deflections and maximum bottom flange temperature per ASTM E119 [29]. While the axial force in the beams and connections were not measured, they were calculated through benchmarked finite element models [33]. The modeling results indicated that only compression forces were in the beams and connections throughout the fire exposure. The maximum axial force in the connections were 250 kN for the unprotected steel beam and 150 kN for the protected steel beam.

The results and testing methodologies of these frame tests vary widely. The reported results all included temperature histories of the beam bottom flange, however, not all of the testing results included temperature distributions through the connection. In addition, some researchers reported axial force and connection rotation histories, but not all of the research studies measured this throughout the tests. The testing setups varied in that some of the tests scaled the length of the beam and some of the experiments included a concrete deck, while others only used a blanket on the top flange of the beam to simulate the effects of a concrete deck. The testing methodologies varied widely between the experiments. Researchers used different fire exposures or heating rates, some with cooling, and others without. In addition, researchers loaded the beams to different loading ratios, which will impact when the connections themselves fail. These inconsistencies make it difficult to quantitatively compare the results from the tests to one another to draw any conclusions or to validate the code provisions.

3.4 Floor system tests

The British Research Establishment and British Steel sponsored a series of seven compartment fire tests on an eight-story building called the Cardington Tests. The building was a full-scale steel-frame building with concrete on metal deck. The total height of the building was 33 m (108 ft) with a floor plan of 5 bays by 3 bays, each 9 m (30 ft) in length and width [2, 34, 35]. The building was designed as a wind-controlled building and throughout all of the compartment fire tests an imposed load of 2.4 kPa was applied through the use of sandbags. All of the girder-to-column connections were end plate connections and the girder-to-beam connections were fin plate (shear tab) connections. These connections were of standard dimensions and therefore, the beams and girders throughout the building were UB254, UB305, UB356, and UB610 and the columns were UC305x198x118 and UC254x89. The last of these tests was an office fire demonstration. The compartment was built out with real office furniture where the fuel load was equivalent to 46 kg/m^2 of wooden cribs. The compartment was 18 m x 10.5 m on the first floor of the building. The columns and girder-to-column connections were heavily protected to prevent collapse of the building. However, all beams and beam-to-girder connections were left exposed. After flashover, the fire was ventilation-controlled with a maximum heat release rate of 58MW. Maximum gas and steel temperatures were 1150°C and 1100°C respectively [36]. High-temperature strain gages were used to measure the horizontal forces in the exposed steel beams at locations near the fin plate connections throughout the fire and thermocouples were used to measure the thermal distribution through the cross section of the beams and columns. The strain measurements were utilized to calculate the stress in the beams close to the fin plate connections and in the columns. These stress values were reported as a function of temperature throughout the fire [36]. The results of this test showed that very high tensile stresses developed at the connections due to thermal contraction of the deflected beams.

Choe et al. [37] conducted the first of four compartment fire experiments on a full-scale two-story steel framed building designed and constructed following current U.S. construction practice [3, 4]. The 9.1 m × 6.1 m composite floor assembly, situated in the middle edge bay on the first floor of the prototype building, was tested to failure under a natural gas fueled compartment fire (simulating ASTM E119 temperature-time curve) and simultaneously applied mechanical loads (2.7 kPa). The ends of 9.1 m long W16×31 beams were connected to the column flange (W12x106) and at midspan of the 6.1 m long W18×35 girders using standard shear tabs; Extended shear tabs were used to connect the ends of W18×35 girders to the column web (W12x106). The test setup included a concrete on metal deck, with the use of lightweight concrete (a minimum specified compressive strength of 28 MPa) and a 76 mm deep formed steel decking. The thickness of the topping concrete was 83 mm required for the 2-hour fire resistance rating with exposed steel deck. The cold-formed welded wire reinforcement of 59 mm^2/m was embedded 41 mm below the top surface of the concrete slab. The floor slab was acting partially composite with steel floor beam assemblies through headed stud anchors (shaft diameter 19 mm). The corresponding degree of composite action was about 65% of the ambient yield strength of the steel beams. The load ratio (i.e., total load normalized by the ambient design capacity) of the test assembly was approximately 0.3 for both the secondary composite beam (W16x31) and standard shear tabs of the same beam. This test demonstrated that all fire-protected floor beams and girders met the ASTM E119 limiting temperatures. Also, these steel members never reached runaway at large vertical displacements (up to the ratio of L/16). However, the heated slab formed through-depth center cracks next to the secondary beam at 70 min, failing to contain fire in the compartment of origin. The test fire and mechanical loading was extinguished at 107 min. The test floor assembly did not collapse during cooling. The extended shear tabs welded to the interior columns exhibited noticeable out-of-plane deflection but no bolt failures. The extended shear tabs welded to the exterior columns deflected little, whereas there were partial shear ruptures in the lower bolts. The connection temperature data for this experiment are presented in Dai et al. [38]. However, it is not certain when these bolts began to rupture and there were no explicit measurements made to quantify the fire-induced forces in the connection regions.

4. Variation in research approaches

As previously summarized, a number of researchers have performed experimental and numerical studies to evaluate the behavior of simple shear connections in fire. While all of these researchers reported the failure mode of the connections (when applicable) and damage observed (e.g. bolt hole elongation), due to a wide variety of parameters used in these studies, it is difficult to synthesize research findings and provide prescriptive design recommendations. Uniformity in testing methods will help establish a consistent design approach for simple (shear) connections with integrity under fire conditions.

4.1 Variation in Test Setup

Test setups varied among test programs and some of these differences are summarized in Table 1. Test specimens varied from individual components to members within a structural bay [8, 9, 11, 13, 17, 19-22]. Some of the tests consisted of stub beam-to-column connections on one or both sides of the column [10, 18]. Other tests included beams in a single frame or as part of a bay of a structure [17, 19-23]. These subassemblage and frame tests vary in dimensions, size, testing methodology, and reported data to the large-scale floor tests [34, 37], therefore, it is difficult to compare the results across the varying scales of testing.

4.2 Variation in Passive Fire Protection

There were also differences in the level of passive fire protection provided in each test. Fire protection on the connecting member (i.e., beam or girder) will affect the temperature of that member and, thus, the thermal elongation, axial forces, and rotations generated within the connection. Similarly, increased fire protection in the connection region will decrease the temperature of the connection elements (i.e. bolts, welds) and, therefore, increase the strength and stiffness capacity of the connection to resist axial forces and rotations. Thus, the amount of fire protection on the connection as well as the beam will affect the performance of the connection. Current guidance on fire protection in IBC [32] and

ASTM E119 [29] is limited to members and/or floor systems, but does not include specific guidance on connection regions Some studies [13, 22, 23, 31, 33, 37] considered varying levels of fire protection. The remainder of the studies either did not incorporate any passive fire protection on the connection region [7, 9-12, 15-19, 21, 25, 26] or did not vary the thickness of fire proofing [8, 14, 20].

Table 1: Summary of variation in test setup
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4.3 Variation in Specimen Type

Tests that consist of realistic representation of the thermal restraint provided in a realistic floor assembly would provide more realistic representation of expected behavior in building structure. Further, some tests were performed with only steel members (beams, columns), while others included a concrete slab or a composite slab on metal deck. The composite slab has been shown to contribute to the structural integrity of the system by providing continuity and assisting with catenary action. In addition, the thermal gradient through the beam will be different with and without the presence of a composite slab. Due to the high expense, many experimental tests were not full-scale with spans and sizes less than typical beam sizes; very few tests [22, 31, 34, 37] were conducted at full-scale.

4.4 Variation in Heating Protocol

There is also significant variability in the heating protocol used in tests. Many experiments tested using steady state heating to reach a specified temperature without attempting to simulate a realistic fire condition [10, 18]. Other tests used standard fire curves such as ISO 834 [27] and focused on the temperature effect on simple connection behavior. A few tests considered both the growth and decay of a fire using natural fire or parametric fire curves [20, 21, 31, 34]. These fires were typically produced using a furnace or radiant heaters, or were numerically simulated in finite-element models.

4.5 Variation in Load Ratio

Load ratios varied from 0.2 to 0.7 and some of the tests were based on a displacement-controlled protocol [10, 18] and not achieving a specific load ratio. Lawson [14] varies the load ratios on the beam from 0.2-0.7 (in increments of 0.1) of the ambient flexural capacity of the beam. Liu et al. [17] tested beams with load ratios of 0.3, 0.5, and 0.7 of the ambient flexural capacity of the beam to include load ratio as a parameter of the testing series. Wang et al. [19] varied the flexural capacity of the connection rather than the load ratio on the beam itself. Lastly, Fischer et al. [21] tested beams with load ratios of 0.37, 0.41, and 0.6 of the ambient flexural capacity of the beam. Kodosky et al. [23] tested composite beams with a load ratio of 0.35 with regards to service loading. The load ratios indicated by Fischer et al. [21] and Kodosky et al. [23] correspond to the ratio of the tested specimen to the ambient flexural capacity of the composite beam. Liu et al. [17] did not use a composite slab on the beam, therefore the load ratios tested are for the steel beam itself, rather than for a composite beam.

4.6 Variation in Reported Data

Test data from previous experimental tests varied in terms of the results that were reported. Table 2 outlines the different data that was measured and reported in the literature. Most tests presented the peak failure load, failure mode, corresponding failure temperature, and failure time. A variety of measured parameters such as axial forces, deformations, or rotations as a function of temperature or time recorded after ignition of a fire were inconsistently measured and reported. Measured maximum tension and maximum compression in the connection were not always reported, making it difficult to draw conclusions relative to the structural integrity provisions. In addition, often researchers only reported beam bottom flange connection histories or maximum beam bottom flange temperature, without any reporting of the connection temperature(s).

None of the aforementioned component-level tests specifically evaluated the effects of thermal gradients on failure modes of connections. A few frame tests [22, 23, 28, 38] reported temperature distributions through the connection regions and connecting members (composite beams or girders) when subjected to compartment or furnace fires. The connections used in these tests allowed large end rotations of the composite members developing thermal gradient under fire exposure. However, temperatures of the connections were highly influenced by thermal shading from the adjoining structures and fuel distribution within a test compartment [28, 38]. Furthermore, among these tests, there is significant variability in the fire protection of connections, size of test specimens, and stiffness of the surrounding reaction frame. Therefore, the shear-tab connections used in these tests exhibited a dissimilar failure mode, and the influence of the thermal gradient on the behavior of the connection could not be conclusive.

Table 2: Summary of variation in data measured and reported
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5. Conclusions and recommendations for future research

Both European and US design provisions [3-6] provide guidance for engineers to design connections for structural integrity. This guidance is now being applied to performance-based design of structures for fire conditions. However, this code-prescribed guidance is not aligned with the vast history of connection research on the behavior of connections in fire. The authors synthesized previous research on the fire behavior of simple shear connections around the world. This connection research highlighted varying methods used for experimental tests and numerical modeling lack of consistency (due to a wide range of parameters) has provided scatter in the data.

To date, much of the experimental and numerical research has made simplifications or tested connections in unrealistic conditions. The data from these tests have greatly contributed to the understanding of gravity connection behavior in steel-frame buildings in fires. However, without full-scale experiments or numerical models (that do not include idealizations), the research community is unable to validate previous simplifications and apply much of the knowledge gained to performance-based fire design for steel-frame buildings. In addition, the lack of full-scale experiments under realistic fire conditions creates a void of data to benchmark numerical models and perform parametric studies.

Standard methods of fire protection design [29, 32] do not have standardized testing methods for simple connections, thereby leaving a large gap in the fire protection design process. Recognizing the impact of examined test parameters on the performance of simple connections under fire, the following recommendations for simulating and measuring realistic connection behavior in fire are suggested to develop consistent design methodologies around the world.

1) ***Orient beam specimens in a frame configuration to mimic realistic restraint and connection rotations under realistic loading conditions***

When researchers are investigating the behavior of simple connections under fire loading conditions, the boundary conditions of the beams must be realistically represented. These boundary conditions will influence the axial forces measured in the connections and subsequently the beams. This recommendation will allow the verification and validation of developed constitutive models for connections.

2) ***Include a continuous composite slab***

In steel-frame buildings, gravity beams are typically composite with a concrete deck. This concrete deck contributes to the flexural capacity of the steel beam and develops the thermal gradient through the composite section when exposed to a fire.. Therefore, a continuous composite slab across a beam will represent the beam capacity realistically throughout the experiment. Also, the presence of a composite slab continuous over the girders will help redistribute thermally restrained forces and displacements into the surrounding bays which would remain cooler during a fire event. Without the presence of the slab, the thermally restrained forces at the connection might be overestimated.

The concrete slab will also act as a heat sink for the steel beam and create a thermal gradient through the cross section of the steel beam. This thermal gradient will influence the vertical deflection of the steel beam throughout the fire and subsequently the connection rotations.

Lastly, the concrete slab braces the top flange of the steel beam in a steel-frame building. This bracing prevents lateral-torsional buckling of the beam under ambient conditions. When the concrete slab is not included within the experiment and the beam is not braced, local buckling may occur and cause the beam to fail prematurely. The presence of the concrete slab prevents this premature failure and resulting underestimation of the steel beam flexural capacity.

3) ***Apply elevated temperatures using a design-basis fire that provides a heating and cooling phase in order to understand connection behavior under both phenomena***

The previous research summarized within this paper has demonstrated that forces in connections will change during the heating and cooling phases of a design-basis fire. Therefore, research that aims to provide code recommendations for steel-frame building design should consider both phases of a fire to quantify the demands in the connections. Much of the previous research summarized in Tables 1 and 2 only included the heating portion of the fire. However, there is a lack of data on the behavior of these connections during the cooling phase of the fire, which is when many researchers observed bolt shear fracture and weld fracture.

4) ***Measure the maximum tension and compression in the connection, as well as the maximum connection rotation either during the test or through benchmarked finite element simulations***

Simple connections will be subjected to both compression and tension throughout a fire while they undergo large rotations. Researchers should ideally report the axial force and rotation histories of the connection throughout the fire; however, when this data is not available, researchers should report the maximum tension and compression forces and maximum connection rotations. These findings will help to determine code-prescribed performance objectives. One such performance objective is a simplified relationship between tension and shear that can be applied in building codes in order to increase structural integrity of these connections.

5.1 Recommendations for future research

There are large gaps in knowledge on connection behavior in fires. Double angle and shear tab connections have been studied somewhat extensively. The following list are gaps in knowledge the authors have identified:

- Studies on single angle connections are needed since they are widely used in construction practice, but their fire performance has not been well characterized yet.
- Duration of heating, maximum fire temperature, and varying fire protection thicknesses will influence the temperature of steel members. Many studies have occurred using the standard fire [27, 29] and with unprotected members. However, data on how different fire intensities influence connection behavior and how varying thickness of fire protection can assist in mitigating connection failure has not been thoroughly investigated.
- The influence of bolt group geometry on connection behavior,
- Temperature-dependent material properties of welds and the influence of weld size on connection behavior,
- The impact of thermal gradients through connections on the behavior of the connection in combined shear and axial force (compression and tension).

Further evaluation on a larger set of test results, potentially including the above-mentioned gaps, can highlight rational and simplified ways to design simple (shear) connections for integrity at elevated temperatures. This is critical for the progression of structural fire engineering around the world.

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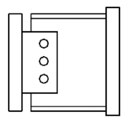
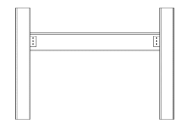
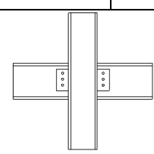
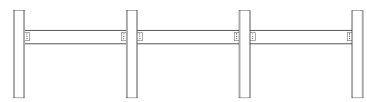
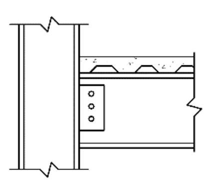
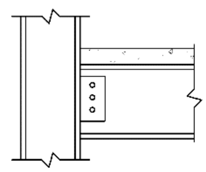
Table 2: Summary of variation of test setup

Experimental vs. Analytical	
Experimental Al-Jabri et al. [16] Choe et al. [22, 31, 37] Fischer et al. [21] Hu & Engelhardt [10] Kordosky et al. [23] Lawson [14] Leston-Jones et al. [15] Liu et al. [17] Pakala et al. [20] Spyrou et al. [25, 26] Wald et al. [28] Wang et al. [19] Yu et al. [18]	*Analytical* Drury et al. [33] Fischer & Varma [13] Garlock & Selamet [8] Hantouche et al. [12] Hu & Engelhardt [10] Pakala & Kodur [11] Sarraj et al. [7] Selamet & Garlock [9]

Test Set up	
Component Method Hu & Engelhardt [10] Spyrou et al. [25, 26] Yu et al. [18] 	*Single Bay* Choe et al. [22, 31] Drury et al. [33] Fischer et al. [21] Garlock & Selamet [8] Hantouche et al. [12] Kordosky et al. [23] Liu et al. [17] Pakala et al. [20] Pakala & Kodur [11] Wang et al. [19] 
Cruciform Lawson [14] Al-Jabri et al. [16] Leston-Jones et al. [15] 	*Multi-bay or System Level* Choe et al. [37] Fischer & Varma [13] Wald et al. [36] 

With / Without Slab		
With Composite Slab on Metal Deck Al-Jabri et al. [16] Choe et al. [22, 31, 37] Drury et al. [33] Fischer et al. [21] Fischer & Varma [13] Garlock & Selamet [9] Kordosky et al. [23] Lawson [14] Pakala et al. [20] Selamet & Garlock [9] Wald et al. [36] 	*With Concrete Flat Slab* Fischer & Varma [13] Lawson [14] 	*Without Slab* Al-Jabri et al. [16] Hantouche et al. [12] Hu & Engelhardt [10] Leston-Jones et al. [15] Liu et al. [17] Pakala et al. [20] Pakala & Kodur [11] Spyrou et al. [25, 26] Wang et al. [19] Yu et al. [18]

Connection Type			
Shear Tab Choe et al. [22, 31,37] Drury et al. [33] Fischer et al. [21] Fischer & Varma [13] Garlock & Selamet [8] Hu & Engelhardt [10] Kordosky et al. [23] Yu et al. [18] Wald et al. [36] Wang et al. [19]	*End Plate* Al-Jabri et al. [16] Lawson [14] Leston-Jones et al. [15] Liu et al. [17] Wang et al. [19]	*Double Angle* Choe et al. [22, 31, 37] Fischer et al. [21] Fischer & Varma [13] Hantouche et al. [12] Lawson [14] Liu et al. [17] Pakala & Kodur [11] Pakala et al. [20] Wang et al. [19]	*Single Angle* Fischer et al. [21] Fischer & Varma [13]

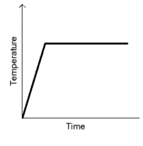
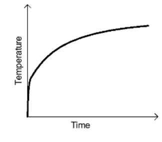
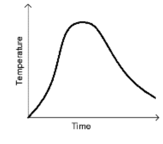
Steady State vs. Transient (experimental only)		
Steady State Al-Jabri et al. [16] Hu & Engelhardt [10] Leston-Jones et al. [15] Spyrou et al. [25, 26] Yu et al. [18] 	*Transient – Standard Furnace* Fischer et al. [21][1] Kordosky et al. [23][1] Lawson [14] Liu et al. [17] Pakala et al. [20][1] Wang et al. [19]  [1]with controlled cooling	*Transient –Compartment Fire* Choe et al. [31, 37] Wald et al. [36] 

Table 3: Summary of variation in data measured and reported

Tensile force in the connection Choe et al. [31] Fischer et al. [21] Hantouche et al. [12] Hu & Engelhardt [10] Lawson [14] Spyrou et al. [25, 26] Wald et al. [36] Wang et al. [19] Yu et al. [18]	**Compressive force in the connection** Choe et al. [31] Drury et al. [33] Fischer et al. [21] Hantouche et al. [12] Spyrou et al. [25, 26] Wald et al. [36] Wang et al. [19]
Temperature history of the connection Choe et al. [22, 31] Dai et al. [38] Drury et al. [33] Kordosky et al. [23] Lawson [14] Wald et al. [36]	**Temperature history of the bottom flange** Choe et al. [22, 31, 37][1] Dai et al. [38][2] Drury et al. [33] Kordosky et al. [23][1] Lawson [14][1,2] Wald et al. [36] Wang et al. [19] [1] also top flange and web [2] also top/bottom bolts and column
Maximum temperature of connection during fire exposure Choe et al. [22, 31] Dai et al. [38] Drury et al. [33] Fischer et al. [21] Kordosky et al. [23] Lawson [14] Wald et al. [36]	**Maximum bottom flange temperature during fire exposure** Choe et al. [22, 31, 37] Dai et al. [38] Drury et al. [33] Fischer et al. [21] Kordosky et al. [23] Lawson [14] Wald et al. [36] Wang et al. [19]
Maximum connection rotation Choe et al. [22, 31, 37] Fischer et al. [21] Hantouche et al. [12] Lawson [14] Yu et al. [18]	**Load ratio** Choe et al. [31, 37] Fischer et al. [21] Hantouche et al. [12] Kordosky et al. [23] Lawson [14] Liu et al. [17] Wang et al. [19]
Maximum beam displacement Choe et al. [31, 37] Drury et al. [33] Fischer et al. [21] Hu & Engelhardt [10] Kordosky et al. [23] Wald et al. [36]	**Moment-rotation curves** Al-Jabri et al. [16] Drury et al. [33] Leston-Jones et al. [15]