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Integrity of Bolted Angle Connections Subjected to Simulated Column Removal

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Abstract

Large-scale tests of steel gravity framing systems (SGFSs) have shown that the connections are critical to the system integrity, when a column suffers damage that compromises its ability to carry gravity loads. When supporting columns were removed, the SGFSs redistributed gravity loads through the development of an alternate load path in a sustained tensile configuration resulting from large vertical deflections. The ability of the system to sustain such an alternate load path depends on the capacity of the gravity connections to remain intact after undergoing large rotation and axial extension demands, for which they were not designed. This study experimentally evaluates the performance of steel bolted angle connections subjected to loading consistent with an interior column removal. The characteristic connection behaviors are described and the performance of multiple connection configurations are compared in terms of their peak resistances and deformation capacities.

Keywords

Steel; Connections; Structural Integrity; Robustness; Disproportionate Collapse; Bolted Angle; Gravity Framing

INTRODUCTION

To evaluate the integrity of a steel framed structure in the event that a column loses its capacity to support gravity loading, current disproportionate collapse guidelines suggest alternate load path analyses under notional column removal as the primary means to evaluate whether the system can redistribute the gravity loads (UFC 2009; USGSA 2003). For steel gravity framing systems (SGFSs) an alternate load path forms when large vertical deflections at the removed column mobilize tensile membrane (or catenary-type) action in

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the system (Foley et al. 2006; Sadek et al. 2008; Johnson et al. 2014). As a result, the gravity connections are subjected to large rotation and axial extension demands that are substantially different from the demands for which they were designed.

The performance of steel bolted angle connections under seismic demands (i.e., increasing magnitude cyclic loads) has been studied by a number of researchers (e.g., Shen and Astaneh-Asl (1999), Garlock et al. (2003)). These tests provide valuable data on the axial compression and tension-prying responses of the bolted angles; however, the deformation capacities of bolted angles subjected to the large monotonic rotation and axial extension demands considered in this study may be significantly different than those observed under cyclic loading.

Guravich and Dawe (2006) tested 108 full-scale connections under combined loading. Their tests were not intended to be representative of column removal, but were rather designed to measure the tension capacity of connections in combination with a nominal 0.03 radians of rotation and large shear forces. Of the total number of connections, 49 were bolted double angle connections, 36 were welded double angle connections, and 12 were welded single angle connections. Results from their tests showed that connections with angles can sustain large tension forces in combination with their design shear capacity. Although the 0.03 radian rotation used in Guravich and Dawe (2006) is significantly less than the rotations expected for connections under column removal, this study still provides valuable data for the behavior of bolted angle connections under combined loads.

Experiments involving bolted angle connections subjected to the levels of rotation and axial extension demands associated with column removal are more limited. Oosterhof and Driver (2012) tested bolted angle connection sub-assemblies under rotation and axial loads derived from a rigid hinged-beam free body diagram model to simulate a column removal. They found that bolted angle connections had considerable deformation capacities due to angle unfolding and that cracking through the column bolt line limited the connection tension capacity. Yang and Tan (2013) tested four sets of simple connections and three sets of semi-rigid connections under a center column removal scenario. Of the tests with simple connections, one used bolted web angles and another used top and seat angles. Yang and Tan (2013) found that bolted web angle connections resisted load primarily through the development of catenary action at large rotations, whereas top and seat angle connections resisted vertical load through flexural action at small rotations.

System tests of gravity frames with bolted angle connections under column loss are even more limited. The resistance of a two bay composite SGFS with top and seat angle connections was investigated by Astaneh-Asl et al. (2001a, 2001b) under an edge column pull-down scenario. Jahromi et al. (2012) tested a small-span full-scale SGFS specimen with uniformly distributed gravity loading under an interior column loss scenario. Also, Johnson et al. (2014) tested a half-scale 3-bay square SGFS specimen under four different column removal scenarios. The SGFS specimens in Jahromi et al. (2012) and Johnson et al. (2014) each used a combination of single plate shear and bolted angle connections.

This study presents test data from 17 full-scale bolted angle connection sub-assemblages subjected to demands consistent with an interior column removal scenario in a bare-steel framed system. The key attributes of the bolted angle connection responses, such as the connection force and deformation capacities, are quantified, and the influences of multiple connection parameters such as the number of bolts, bolt diameters, and angle thickness on the connection performance are evaluated and described. In addition, a novel approach to determine the deformations of fibers used to discretize the connections, first introduced in Weigand and Berman (2014), is used to calculate the connection component-level deformation capacities at connection failure, an important parameter for analysis of floor systems in practice.

BOLTED ANGLE CONNECTION SUB-ASSEMBLAGES

The steel bolted angle connection sub-assemblages tested in this study were designed to resist the shear demands resulting from a series of prototypical steel gravity framing systems, with gravity loads modeled after the SAC (i.e., the SAC Joint Venture between the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and the Consortium of Universities for Research in Earthquake Engineering (CUREE)) prototype building loads (FEMA 2000). The prototype systems encompassed a broad range of configurations typical to current industry design practice, and are described in more detail by Weigand et al. (2012). The connection configurations were selected from the prototype system designs and refined to provide a wide breadth of parameter variation. The varied connection parameters, shown in Table 1, included the number of bolts on the angle legs bolted to the column flange (n_b), bolt diameter (d_b), angle leg thickness (t_L), configuration, eccentricity with respect to the beam centerline, and gap between the beam flange and the column flange. All specimens used a simulated span of 9.1 m (30 ft), which controls the relationship between the applied rotation and axial extension as described below. The naming convention used in Table 1 has a prefix that describes the connection type (ba refers to Bolted Angle), followed by the number of bolts (e.g., 3b indicates 3 bolts), bolt diameter fraction in inches (e.g., 3/4 corresponds to 19.1 mm ($3/4$ in.)), angle thickness fraction (e.g., 1/4 corresponds to 6.35 mm ($1/4$ in.)), and additional descriptor (e.g., Offset) where applicable.

The connection sub-assemblages each had a 1219 mm (48.0 in.) long W12×72 column stub and a 1524 mm (60.0 in.) long W21×50 beam stub (Fig. 1(a)). Most connections were bolted-bolted configurations (i.e., bolted through the angle column legs to the column flange and through the angle beam legs to the beam web) with 19.1 mm ($3/4$ in.) bolts and either 6.35 mm ($1/4$ in.) or 12.7 mm ($1/2$ in.) thick angles. Fig. 1(b) shows example schematics of some of the tested connection configurations, and detailed drawings for each of the tested connection sub-assemblage can be found in Weigand (2014). The two angle thicknesses were chosen to investigate the differences in connection performance when angle leg rupture or bolt prying rupture controlled the connection tension capacity. The thinner angle thickness was also chosen so that the beam and column stubs would sustain little damage (i.e., remain essentially elastic) throughout their loading, with nearly all plastic deformations isolated to the angles. This made it possible to re-test several of the beam and column stubs with 12.7 mm ($1/2$ in.) thick angles without introducing substantial imperfections.

The bolted angles for all specimens were manufactured from ASTM A36 L4×4×1/4, L4×4×1/2, or L6×4×3/4 hot-rolled angle stock with a specified nominal yield strength of 248 MPa (36 ksi) and ultimate strength of 400 MPa (58 ksi). All bolts were grade A325 with specified nominal tensile strengths of 827 MPa (120 ksi). The beam and column stubs were A992 wide flange sections. An E71T-8-D electrode with a specified nominal ultimate strength of 483 MPa (70 ksi) was used for all structural fillet welds. The measured strengths of the angles and wide flange sections reported in Table 2, and the measured strength of the bolts are reported in Table 3.

EXPERIMENTAL SETUP

The sub-assemblages were loaded using a self-reacting load frame constructed in the University of Washington (UW) Structural Research Laboratory (SRL) (Fig. 2). The reaction frame was tied to a 762 mm (30 in.) thick prestressed concrete floor system with tensioned 38.1 mm (1 1/2 in.) diameter Williams rods. Within the reaction frame, the sub-assemblage column stubs were oriented horizontally and bolted to the foundation beams. The beam stubs spanned vertically and were fastened to the column stubs via the bolted angle connections and at the opposite end to the load beam.

Rotation and axial extension demands were applied to the connection sub-assemblages via the load beam by the three independent actuators mounted on the reaction frame. The demands were calculated by assuming that the removed column deflects perfectly vertically downward, and that all deformations occur at the connections (Fig. 3(a)). Under these conditions, the connection rotation demand, θ , is related to the vertical deflection of the missing column (termed 'simulated vertical displacement'), Δ , by:

$$\theta = \tan^{-1} \left(\frac{\Delta}{L_r} \right), \quad (1)$$

where L_r is the reduced span of the framing members, defined as the distance between the centers of gravity of connection bolt groups on the ends of the framing members in the undeformed configuration. The connection axial extension demand, δ , can be written in terms of the rotation demand as:

$$\delta = \frac{L_r}{2} \left[\frac{1}{\cos \theta} - 1 \right], \quad (2)$$

or equivalently, in terms of the deflection at the missing column as:

$$\delta = \frac{L_r}{2} \left[\sqrt{1 + \left(\frac{\Delta}{L_r} \right)^2} - 1 \right], \quad (3)$$

in which δ is always aligned with the centerline of the framing member. The relative proportions of rotation and axial extension demands vary for framing members with different spans, with longer spans resulting in increased axial extension relative to the rotation. The connection reactions due to the local displacement demands in Eqs. (1) and (3)

are essentially equivalent to those that would result from application of a concentrated load, P , at the removed column (Fig. 3(b)).

The rotation was applied to the connection sub-assemblages using a 245 kN (55 kip) actuator. The displacement of the 245 kN (55 kip) actuator, δ_{LB} , was calculated as the vertical displacement of the missing column in the simulated system scaled by the ratio of the connection offset from the load beam to the reduced span, L_r , as shown in Fig. 3(c). Axial extension was applied simultaneously by two 490 kN (110 kip) actuators, which rotated to remain parallel with centerline of the beam stub throughout testing. The UW SRL's Material Testing Systems (MTS) FlexTest 60 Controller supplied displacement signals to the horizontal and vertical actuators, relating their displacements in accordance with Eqs. (1) and (3). The connections were loaded quasi-statically to complete failure over a time span of approximately 60 minutes. Additional details on the reaction frame and loading protocol can be found in Weigand and Berman (2014).

Equipment and Instrumentation

Many instruments were used to capture the responses of the connection sub-assemblages. Photographs were taken at regular time intervals with digital single-lens reflex (DSLR) cameras stationed at multiple viewpoints with respect to the connections, to observe the global and local connection deformations. Each actuator had a built-in displacement transducer and was equipped with a 489 kN (110 kip) load cell. Wire potentiometers and electronic axis inclinometers monitored the displacements and rotations of the specimen beam stub, load beam, and actuators. Local to the connections, displacements were acquired with a dense grid of light emitting diode (LED) targets affixed to angles, bolts, and specimen beam and column stubs. The estimated uncertainty in the measured data was $\pm 1\%$, based on repeated calibrations of the instruments over the course of testing.

The data were collected by three separate units: (i) load and displacement signals from the actuators were collected by the FlexTest 60 controller and transmitted via optical cables to the Labview Hub to be recorded, (ii) signals from the potentiometers and inclinometers were recorded directly by the Labview Hub, and (iii) the positions of the LED targets were triangulated using a Northern Digital Inc. Certus HD Position Tracker (NDI OptoTrak) system with two sets of three cameras and recorded by software native to the NDI OptoTrak system.

Connection Response Quantities

Vertical and Horizontal Forces at the Column Face—To account for the rotations of the three actuators used in the experimental setup, the connection displacement and force quantities are presented in coordinates aligned with axes parallel to the column and beam longitudinal axes, prior to testing. Fig. 4 shows a free-body diagram of a connection sub-assemblage under rotation and axial extension demands. The directions of the actuator loads were assumed to intersect at the center of the load beam cross-section and to act through the actuator head swivel pins. For simplicity, only a single 490 kN (110 kip) actuator load is shown in Fig. 4; however, the contributions of the two 490 kN (110 kip) actuators were considered separately and designated as N (north) and S (south). The connection forces at

the column face were determined from summation of the actuator loads (with tension in the actuators assumed positive) as:

$$V = T_{55k} \cos \theta_{55k} - (T_{110kN} \sin \theta_{110kN} + T_{110kS} \sin \theta_{110kS}) \quad (4)$$

$$T = -T_{55k} \sin \theta_{55k} - (T_{110kN} \cos \theta_{110kN} + T_{110kS} \cos \theta_{110kS}) \quad (5)$$

$$M = T_{55k} L_{LB} \left(\cos \theta_{55k} + \sin \theta_{55k} \tan \left(\frac{\theta_{110kN} + \theta_{110kS}}{2} \right) \right), \quad (6)$$

where V was the vertical force at the column face, T was the horizontal force at the column face, and M was the connection moment. T_{55k} , T_{110kN} , and T_{110kS} were the 245 kN (55 kip), north 490 kN (110 kip), and south 490 kN (110 kip) actuator loads, respectively. Data from inclinometers placed on each actuator were used to update the directions of the loads, θ_{55k} , θ_{110kN} , and θ_{110kS} .

Fiber Displacements from Experiments—Connection displacements such as simulated vertical displacement or connection rotation are not sufficient to objectively compare the deformation capacities of connections with varying spans. Thus, a technique to compute fiber displacements from the experimental data was developed in Weigand and Berman (2014) that accounts for the combined contributions to bolt and angle deformations resulting from any combination of rotation and axial extension demands. Although the presented bolted angle connection sub-assemblages all had 9.14 m (30 ft) simulated spans and were therefore subjected to the same rotation and displacement demands, the fiber displacements are still beneficial for the calibration of fiber connection models used to simulate connections in the analysis of gravity frames in disproportionate collapse investigations. The method to compute experimental fiber displacements is summarized below.

The connection sub-assemblages were notionally discretized into fibers, where each fiber represents a characteristic width segment of connector components (i.e., bolt, welds, angles, and beam web). The locations of the fibers were determined prior to the application of load, with fiber-nodes located at the LED targets on the connection beam leg bolt-heads (Fig. 5). One node of each fiber was assumed to be rigidly attached to the fixed column stub, and the other was assumed to be rigidly attached to the beam web. The motions of the beam web fiber-nodes were computed by best-fitting a rigid-link frame structure onto the grid of OptoTrak LED targets attached to the beam web. In the undeformed configuration, the two nodes exactly coincide and the length of each fiber is zero. After deformations, the experimental fiber displacements were computed as the distance between the undeformed and the deformed locations of the fiber nodes and decomposed into axial and shear components.

RESULTS AND DISCUSSION

Characteristic Bolted Angle Connection Responses

The responses of the bolted angle connection sub-assemblages varied with the angle thickness. In nearly all of the tested bolted angle sub-assemblage specimens with 6.35 mm ($\frac{1}{4}$ in.) thick angles, the connection damage was primarily concentrated in the angles with little or no prying in the column leg bolts. However, in the bolted angle connection sub-assemblages with 12.7 mm ($\frac{1}{2}$ in.) thick angles, the increased angle strength was sufficient to induce the prying rupture of the column leg bolts as the primary deformation mechanism and ultimate failure mode. The following paragraphs describe the characteristic responses of the bolted angle connections with 6.35 mm ($\frac{1}{4}$ in.) and 12.7 mm ($\frac{1}{2}$ in.) thick angles in more detail.

Under the combined rotation and axial extension demands, the connection vertical (V) and horizontal (T) force-displacement responses underwent four primary phases of behavior, which are illustrated for specimens ba3b|34|14| and ba3b|34|12| in Fig. 6. During Phase I the connection vertical force-displacement response was characterized by a large initial stiffness as the beam leg bolts resisted moments due to their pretension. Due to the large compression stiffnesses of the angles relative to their tension stiffnesses (i.e., bearing of the angle heels against the face of the column flange in compression was significantly stiffer than prying of the angle heels away from the face of the column flange in tension), some compression was initially developed in the horizontal force-displacement responses and the centers of rotation of the angles were biased toward the compressive sides of the connections.

Slip of the beam (i.e., slip of the beam web within the web angles and slip of the beam leg bolts within the beam web and angle holes) delineated the transition to Phase II behavior. During Phase II the connections rotated, inducing flexural deformations in the angle column legs that resulted in differential separation (across the angle depth) of the angle heels from the column flange. The Phase III response began when the angle heels had completely separated from the column flange, after which the angles were subjected to increasingly large deformations. The transition between Phases II and III marks the transition between the connection flexure-dominated and tension-dominated behaviors. The largest portion of the connection resistance to vertical and horizontal load was developed during Phase III.

As the deformations increased, plastic hinge lines formed in the angles. During the tests, the plastic hinge lines were visible from flaking of the angle steel's mill scale. The hinge lines were also evidenced by the large concentrations of rotations that were plainly visible on the deformed angle cross-sections. The locations of the plastic hinges varied according to the strengths of the angle column legs relative to those of the column leg bolts (Fig. 7). The 6.35 mm ($\frac{1}{4}$ in.) thick angles developed plastic hinges along the inside edge of the column leg bolts, at the toes of the angle radius, and along the inside edge of the beam leg bolts (Fig. 7(a)). When the angles were 12.7 mm ($\frac{1}{2}$ in.) thick, plastic hinges similarly formed at the toes of the angle radius and at the inside edge of the beam leg bolts. However, the other plastic hinge in the angle column legs formed closer to the outer edge of the column bolt line such that the angle deformations pried the column leg bolts away from the column face (Fig. 7(b)).

The deformations in the angles increased until failure of the connections occurred, designating the transition to the Phase IV degradation behavior. In the connections with 6.35 mm ($1/4$ in.) thick angles, cracking at one of the plastic hinges in the angle cross-section near the angle radius led to rupture initiating from the angle tension faces that typically extended 30–50% of the angle depth. With continued loading, the initial tearing propagated until complete rupture of the angles occurred. Fig. 8 shows damage to the north web angle and column leg bolts of Specimen ba3b|34|14| after the test was completed. This pattern, with severe damage isolated to the angles and relatively little damage occurring in the column leg bolts, was typical of the connections with 6.35 mm ($1/4$ in.) thick angles. The connections with 12.7 mm ($1/2$ in.) thick angles failed when the column leg bolts ruptured due to prying. In an individual web angle, the bolt ruptures always progressed sequentially from the tension side of the connection toward the compression side; however, the ruptures were not always evenly distributed between the two web angles. Similar to the previous figure, Fig. 9 shows damage to the north web angle and column leg bolts of Specimen ba3b|34|12| after testing was completed; damage that was typical to the specimens with 12.7 mm ($1/2$ in.) thick angles.

Critical force and displacement quantities at connection failure are shown in Table 4. The quantities $\delta_{\max}$, $\theta_{\max}$, and $\delta_{\max}$ denote the simulated vertical displacement, rotation, and axial deformation, respectively, and $V_{\max}$ and $T_{\max}$ denote vertical and horizontal force at the column face, respectively. The limiting fiber displacement (i.e., the axial displacement of fiber farthest from the connection center of rotation on the tension side of the connection, denoted the ‘outermost tension fiber’) is included under the heading ‘ $d_{f,\text{lim}}$ ’. The normalized vertical resistances, $V_{\max}/V_{\text{nom}}$, were calculated as the ratios of the connection vertical resistances at failure to their nominal shear capacities, V_{nom} , using LRFD-defined limit states (AISC 2011) and the measured angle material and bolt strength properties. Normalized horizontal resistances were not included because the current design limit-state equations do not adequately capture the phenomena intrinsic to the angle prying and unfolding behaviors exhibited by the connections under column removal, and thus tended to underestimate their capacities. The material properties used in the limit state calculations were shown in Table 2. Key results from Table 4 are discussed in more detail in the following section.

Influence of Connection Parameters on the Bolted Angle Connection Responses

Binding (Gap Distance)—Binding of the beam flange on the column flange only occurred in Specimens ba3b|34|14|Gap and ba3b|34|12|Gap, which had deliberately reduced gaps between the beam and column flanges. While the beam flange was bearing on the column flange, the connection rotated about the point of contact, accelerating the progression of deformations at the bolted angle fibers. However, as the connection axial extension demands increased, the beam flange pulled away from the column flange and the connections continued resisting loads prior to failure. While Specimen ba3b|34|14|Gap ruptured at an 11% lower vertical capacity and 8.1% lower deformation capacity than Specimen ba3b|34|14|, Specimen ba3b|34|12|Gap was not adversely impacted by the binding relative to Specimen ba3b|34|12|. These results suggest that connections with relatively large deformation capacities may not be significantly influenced by binding.

Number of Bolts—Increasing the number of bolts in the connection from 3 to 5 increased the connection vertical resistance by 35%, for the connections with 6.35 mm ($1/4$ in.) thick angles, and by 54%, for the connections with 12.7 mm ($1/2$ in.) thick angles; however, those increases were less than would be expected by comparing the ratios of their nominal strengths. Fig. 10(a) compares the normalized vertical responses of Specimens ba3b|34|14| and ba5b|34|14|, which differed only by their numbers of bolts (i.e., 3 and 5, respectively). Fig. 10(b) shows a similar comparison for Specimens ba3b|34|12| and ba5b|34|12|. These results (and the connection capacities shown in Table 4) show that although the measured deformation capacities of the angle fibers ($d_{f,lim}$ values) were slightly larger for the connections with five bolts relative to those with three, their simulated vertical displacements at failure and corresponding normalized vertical and normalized horizontal resistances decreased.

The decrease in connection deformation capacity with increasing numbers of bolts can be explained by considering the deformations at the outermost tension fibers. The deformation capacities of the angle fibers remained relatively constant between tests; however the deformation demands at the outermost tension fibers of the bolted angle connections due to rotation increased (at a given simulated vertical displacement) with increasing numbers of bolts (i.e., with increasing connection depth). This effect caused the demands at the outermost fibers in Specimens ba5b|34|14| and ba5b|34|12| to exceed their deformation-controlled capacity limits at smaller simulated vertical displacements than Specimens ba3b|34|14| and ba3b|34|12|, respectively.

Angle Thickness—For each pair of connections that differed only by angle thickness (e.g., Specimens ba3b|34|14| and ba3b|34|12|, Specimens ba5b|34|14| and ba5b|34|12|, etc.), the bolted angle connections with 12.7 mm ($1/2$ in.) thick angles had larger normalized vertical capacities than the connections with 6.35 mm ($1/4$ in.) thick angles (Fig. 11). In addition, the specimens with 12.7 mm ($1/2$ in.) thick angles more often had larger simulated vertical displacements at failure than those with 6.35 mm ($1/4$ in.) thick angles, in part because increased beam web and column flange deformations added to the overall connection deformations. These tests suggest that connections with thicker web angles may generally provide better performance than those with thinner angles. However, more work would be required to validate this assertion for angle thickness other than those directly tested.

Connection Strength—Though the connection performance was evaluated in terms of the connection vertical capacities as a proportion of their strength (i.e., normalized vertical capacities), it is also interesting to examine the influence of absolute connection strength. Specimen ba3b|1|34| had the largest shear capacity of the connections in this study, and was tested to examine the influence of a large absolute connection strength on the connection performance. The connection angles were so strong that the deformations of the angle column legs were sufficient to locally warp the sub-assembly column stub flanges (Fig. 12). Though the column leg bolts sustained large prying deformations due to the angle uplift, the failure of the connection ultimately resulted from a block shear rupture in the beam web. Despite the brittle failure mode (the connection lost 95% of its capacity almost

instantaneously), Specimen ba3b|34| had the largest deformation capacity and normalized vertical resistance of the entire series of tested bolted angle connections. Much of the deformation capacity was provided by the column flange and the column leg bolts, which absorbed a significant percentage of both the connection rotation and axial extension demands.

Staggered Hole Configurations—The bolted angle connections that failed due to angle rupture failed when localized plastic strains at the toes of the angle radius (i.e., in the plastic hinge regions in Fig. 7(a)) exceeded the angle material ultimate strain. Specimen ba3b|34|14| HConfig featured staggered bolt patterns between the angle beam and column legs in order to evaluate whether distributing the angle unfolding deformations over a longer angular distance (Fig. 13) could effectively reduce the strains at the plastic hinges and enhance the connection performance.

The responses of Specimens ba3b|34|14|HConfig and ba3b|34|14| are compared in Fig. 14(a). Specimen ba3b|34|14|HConfig had a 30% improvement in strength and a 13% improvement in deformation capacity relative to the conventionally configured bolted angle connection, ba3b|34|14|. Fig. 14(b) shows the same comparison, but for the connections with 12.7 mm ($\frac{1}{2}$ in.) thick angles. Specimen ba3b|34|12|HConfig had a 5% reduction in strength and a 4% improvement in deformation capacity relative to specimen ba3b|34|12|. Unfortunately, ba3b|34|12|HConfig failed prematurely due to beam web tearout, so the extent to which the staggered configuration might have improved the performance had the intended angle prying mechanism controlled is unclear. However collectively, these results do imply that methods to reduce the concentrations of deformations at the angle plastic hinge lines could have the potential to improve connection performance under column removal.

Welded-Bolted Configurations—Two welded-bolted angle configurations were tested. Fig. 15 compares the responses of Specimens ba3b|34|14|BlegWeld and ba3b|34|14| ClegWeld (with welded angles) to Specimen ba3b|34|14|. Specimen ba3b|34|14|BlegWeld failed due to angle rupture, which was typical of the connections with 6.35 mm ($\frac{1}{4}$ in.) thick angles. However, the fixity of the beam legs (i.e., no slip condition between the angles and beam web) resulted in larger displacement demands in the angles for a given simulated vertical displacement than had occurred in the bolted-bolted configuration. The increased demands precipitated angle rupture at a reduced simulated vertical displacement, relative to Specimen ba3b|34|14|.

The angles in Specimen ba3b|34|14|ClegWeld were welded to the column face and bolted to the beam web. As the angle column legs were pried away from the column face, cracking initiated in both angles at the interface between the base metal and weld material at the weld returns. Once the cracking had extended through the returns, the prying deformations were concentrated at the column leg weld lines. This reduced the angle prying stiffness. The reductions in the angle prying stiffness and strength resulted in reduced maximum vertical and horizontal force capacities, relative to Specimen ba3b|34|14|.

Remaining Bolted Angle Connection Responses

The vertical force-displacement responses of the bolted angle connections that were not shown in the previous sections are shown in Fig. 16. For brevity, these connection responses are not discussed here; however, detailed descriptions of their responses and additional comparisons between the various connection configurations can be found in Weigand (2014).

COMPARISON OF SINGLE PLATE SHEAR AND BOLTED ANGLE CONNECTION PERFORMANCE

Fig. 17 compares the responses of a single plate shear connection and a bolted web angle connection that were designed to have similar nominal shear capacities. The single plate shear connection (i.e., Specimen sps4b|SSLT|78|38) had a 9.14 m (30.0 ft) simulated span, four 22.2 mm ($\frac{7}{8}$ in.) diameter A325 bolts, a 9.53 mm ($\frac{3}{8}$ in.) thick shear plate, short-slotted holes, and a W21×50 beam stub. The bolted angle connection had a 48% larger simulated vertical displacement at connection failure than the single plate shear connection. However the maximum vertical and horizontal resistances of the bolted angle connection were 30% and 44% lower than the single plate shear connection, respectively. The trend illustrated by Fig. 17 was not exclusive to these particular specimens, but was typical of the overarching behaviors of the two connection types. Fig. 18 shows a comparison of the simulated vertical displacements versus normalized vertical resistances at connection failure of 11 single plate shear connection specimens to those of the bolted angle connection specimens presented in Table 4. The single plate shear connections had 9.14 m (30.0 ft) simulated spans with numbers of bolts ranging from 3 to 5, shear plate thicknesses ranging from 6.35 mm ($\frac{1}{4}$ in.) to 9.53 mm ($\frac{3}{8}$ in.), and bolt diameters ranging from 19.1 mm ($\frac{3}{4}$ in.) to 22.2 mm ($\frac{7}{8}$ in.). Other connection parameters, such as bolt grade and hole type, were also varied between connections. Further details on the single plate shear connections used in the comparisons shown in Figs. 17 and 18, including specimen drawings and detailed experimental results, can be found in Weigand and Berman (2014).

Fig. 18 shows that, for gravity frames subjected to column removal, the use of bolted web angle connections would likely result in a larger vertical displacement at the missing column at connection failure than if the system used single plate shear connections. However, the increased deformation capacity of the bolted angle connections was clearly at the expense of lower maximum vertical resistances.

SUMMARY AND CONCLUSIONS

The purpose of this research was to investigate the performance of steel gravity connections subjected to column removal loading, to provide the data necessary to better understand their behavior and their contribution to the robustness of Steel Gravity Framing Systems (SGFSS). This paper has presented results from tests of steel bolted angle connections subjected to large rotation and axial extension demands consistent with an interior column removal scenario. Several key connection parameters were varied between the tests to evaluate their influences on the connection performance. The characteristic bolted angle connection

behaviors were identified from the data and were categorized into four primary phases of behavior progressing from resistance due to bolt pretension, to slip and flexural deformations, to angle and potentially bolt prying deformations in tension, and ultimately to connection failure. Key connection force and deformation capacities were reported at connection failure for each of the tested configurations, including their limiting fiber displacements, which were estimated using LED targets attached to the beam webs near the connections.

The bolted angle connections that employed 6.35 mm ($1/4$ in.) thick angles failed due to angle rupture at the toe of the angle radius. Connections that used 12.7 mm ($1/2$ in.) thick angles typically failed through sequential prying of the bolts along the angle column legs. The connections with thicker angles generally had better performance than those with thinner angles, when the vertical displacements at the missing column and the connection normalized vertical resistances were used to evaluate the connection performance.

The two bolted-welded angle configurations had the lowest vertical resistances of the tested bolted angle connections. If bolted-welded angle connections must be relied on for disproportionate collapse resistance, they may require improved detailing to reach even 5% of their nominal shear capacities.

Results from this study showed that bolted web angle connections offered increased deformations at connection failure relative to single plate shear connections. However, the opposite trend was found with respect to their normalized vertical resistances. These offsetting economies expose a trade-off between the use of these two connection types in gravity frames that may be subjected to column removal. It is presently unclear which of these two inherent benefits would prove more advantageous for supporting gravity loads when acting compositely with a concrete slab in a gravity framing system; however, estimates of the bare-steel framing-only contribution to the system capacity when subjected to column removal would favor the single plate shear connections.

The primary findings from the bolted angle connection sub-assemblages subjected to combined rotation and axial extension were as follows:

- The bolted angle connections with 6.35 mm ($1/4$ in.) thick and 12.7 mm ($1/2$ in.) thick angles provided an average of only 4.7% and 7.4% of their nominal shear capacities, respectively, as a vertical force at the column face when subjected to column removal. Clearly these capacities were not adequate to sustain column removal, and future research on SGFSs may need to consider the role of the composite slab in resisting the gravity loads.
- Larger connection depths decreased the simulated vertical displacement of the missing column at connection failure and the measured strength as a proportion of the nominal connection strength.
- The axial extension demands on the connections suppressed the likelihood of binding between the columns and the ends of the beams. When the bolted angle connections did bind, their vertical capacities had a maximum reduction of 11%.

- Offsetting the holes between the angle beam and column legs effectively reduced the concentrations of deformations at the toes of the angle radius. This implies that methods to reduce the deformations at the angle plastic hinge lines may improve connection performance.

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**Bolted Web Angle
Connection**

**Weak-Axis Bolted Web Angle
Connection**

**Bolted Top and Seat Angle
Connection**

Fig. 1.

(a) Typical bolted web angle connection sub-assembly specimen (connection details vary, see Table 1) and (b) example schematic views of tested connections

Fig. 2.
Bolted angle connection sub-assemblages mounted within self-reacting load frame

Fig. 3.
 (a) Two-span system deformation, (b) two-span system load, and (c) deformed connection sub-assembly specimen configuration

Fig. 4. Free-body diagram of connection sub-assembly experimental setup (to scale assuming that the specimen beam is rigid relative to the connection)

Fig. 5.
Method for computing experimental fiber displacements from NDI LED target data

Fig. 6.

(a, b) Phases of vertical and horizontal force-displacement behavior for Specimen ba3b|34|14| and (c, d) phases of vertical and horizontal force-displacement behavior for Specimen ba3b|34|12|

Fig. 7. Bolted web angle deformation patterns observed in the connection sub-assemblies with (a) 6.35 mm (1/4 in.) thick angles and (b) 12.7 mm (1/2 in.) thick angles. Plastic hinges are indicated by the red hatched regions.

Fig. 8. Specimen ba3b|34|14| (a, b, c) front and section views of ruptured north web angle beam leg, (d, e, f) front and section views of ruptured north web angle column leg, and (g, h, i) damage to the north web angle column leg bolts

Fig. 9. Specimen ba3b|34|12| (a, b, c) front and section views of damage to the north web angle, (d, e, f) ruptured north web angle column leg bolt heads, and (g, h, i) ruptured north web angle column leg bolt nuts

Fig. 10. Comparison between normalized vertical force-displacement responses for (a) Specimens ba3b|34|14 and ba5b|34|14 and (b) Specimens ba3b|34|12 and ba5b|34|12

Fig. 11.

Comparison between normalized vertical capacities of bolted web angle connections with 6.35 mm (1/4 in.) thick and 12.7 mm (1/2 in.) thick angles. Connections of the same configuration but different angle thicknesses are connected via a dashed line.

Fig. 12. Magnitudes of column warping in Specimen ba3b|1|34| throughout testing. (a) Mild column warping, (b) moderate column warping, and (c) severe column warping.

Fig. 13.
Lines of transverse angle unfolding in staggered bolted angle connection

Fig. 14.

Comparison of vertical force-displacement responses of (a) Specimens ba3b|34|14| and ba3b|34|14|HConfig and (b) Specimens ba3b|34|12| and ba3b|34|12|HConfig

Fig. 15. Comparison of (a) vertical and (b) horizontal force-displacement responses for Specimens ba3b|34|14, ba3b|34|14|BlegWeld, and ba3b|34|14|ClegWeld

Fig. 16. Vertical force-displacement responses of connections that were not shown in the previous figures

Fig. 17. Comparison of (a) vertical and (b) horizontal force-displacement responses for Specimens sps3b|SSLT|34|38 and ba3b|34|14

Fig. 18.
Comparison between normalized vertical capacities of single plate shear and bolted web angle connections

TABLE 1

Bolted angle connection sub-assembly configurations

Specimen Name	Connection Properties			
	n_b	C_{LEG} d_b mm (in.)	B_{LEG} d_b mm (in.)	t_L mm (in.)
ba3b 34 14	3	19.1 ($3/4$)	19.1 ($3/4$)	6.35 ($1/4$)
ba3b 34 12				12.7 ($1/2$)
ba5b 34 14	5	19.1 ($3/4$)	19.1 ($3/4$)	6.35 ($1/4$)
ba5b 34 12				12.7 ($1/2$)
ba3b 1 34	3	25.4 (1)	25.4 (1)	19.1 ($3/4$)
ba3b 34 14 Offset ¹	3	19.1 ($3/4$)	19.1 ($3/4$)	6.35 ($1/4$)
ba3b 34 12 Offset ¹				12.7 ($1/2$)
ba3b 34 14 Gap ²	3	19.1 ($3/4$)	19.1 ($3/4$)	6.35 ($1/4$)
ba3b 34 12 Gap ²				12.7 ($1/2$)
ba3b 34 14 TopSeat ³	3	19.1 ($3/4$)	19.1 ($3/4$)	6.35 ($1/4$)
ba3b 34 12 TopSeat ³				12.7 ($1/2$)
ba3b 34 14 HConfig ⁴	3	19.1 ($3/4$)	25.4 (1)	6.35 ($1/4$)
ba3b 34 12 HConfig ⁴				12.7 ($1/2$)
ba3b 34 14 BlegWeld ⁵	3	19.1 ($3/4$)	-	6.35 ($1/4$)
ba3b 34 14 ClegWeld ⁶	3	-	19.1 ($3/4$)	6.35 ($1/4$)
ba3b 34 14 Weak ⁷	3	19.1 ($3/4$)	19.1 ($3/4$)	6.35 ($1/4$)
ba3b 34 12 Weak ⁷			19.1 ($3/4$)	12.7 ($1/2$)

¹ Angles offset 76.2 mm (3.0 in.) from beam centerline.² Reduced gap of 6.35 mm ($1/4$ in.) between beam flange and column flange.³ Top-and-seat angle configuration.⁴ Angles had three 19.1 mm ($3/4$ in.) bolts on column legs and two 25.4 mm (1 in.) bolts on beam legs.⁵ Angles bolted to column face and welded to beam web.⁶ Angles welded to column face and bolted to beam web.⁷ Weak-axis configuration that frames into column web.

TABLE 2
 Material and component strengths used to calculate bolted angle connection nominal capacities

Coupon Type	Elastic Modulus MPa (ksi)	Yield		Ultimate		Percent Elongation
		ϵ_y mm/mm	σ_y MPa (ksi)	ϵ_u mm/mm	σ_u MPa (ksi)	
6.35 mm (1/4 in.) thick A36 Angles	206,330 (29,926)	0.004	382 (55.4)	0.270	669 (77.1)	31.6
W21×50 Beam Web (Longitudinal)	221,390 (32,110)	0.004	419 (60.8)	0.230	578 (83.9)	32.5
W21×50 Beam Web (Transverse)	235,680 (34,182)	0.003	437 (63.4)	0.222	594 (86.1)	28.2

Note: Values averaged from multiple coupon tests.

TABLE 3

Material and component strengths used to calculate bolted angle connection nominal capacities

Component Type	Bearing Stiffness kN/mm (kip/in.)	Bolt Deformation Capacity mm (in.)	Bolt Shear Strength (in double-shear) kN (kip)	Bolt Bearing Deformation mm (in.)
50.8 mm (2 in.) long 19.1 mm (³ / ₄ in.) diameter A325 Structural Bolts	106.6 (608.9)	5.56 (0.219)	287 (64.6) ^{<i>I</i>}	4.19 (0.165)
63.5 mm (2½ in.) long 19.1 mm (³ / ₄ in.) diameter A325 Structural Bolts	208.1 (1,188.2)	3.51 (0.138)	320 (72.0) ^{<i>I</i>}	3.47 (0.137)

Note: Values averaged from multiple component tests.

^{*I*}From results of bolt double-shear tests.

TABLE 4

Bolted angle connection sub-assembly force and deformation quantities at connection failure

Specimen Name	System, Connection Displacement Demands				Connection Vertical and Horizontal Capacities		Normalized Vert. Capacities	
	max mm (in.)	$\theta_{\max}$ rad.	$\delta_{\max}$ mm (in.)	$d_{f,lim}$ mm (in.)	$V_{\max}$ kN (kip)	$T_{\max}$ kN (kip)	$\frac{V_{\max}}{V_{nom}}$	Limit State
ba3b 34 14	1175 (46.2)	0.133	37.6 (1.481)	45.3 (1.782)	34.2 (7.70)	282 (63.3)	0.049	Angle Rupture
ba3b 34 12	1168 (46.0)	0.132	37.2 (1.465)	41.6 (1.639)	61.1 (13.74)	543 (122.0)	0.067	Bolt Rupture
ba5b 34 14	1033 (40.7)	0.117	29.3 (1.155)	46.7 (1.839)	46.4 (10.42)	373 (83.9)	0.041	Angle Rupture
ba5b 34 12	1078 (42.4)	0.118	31.9 (1.255)	47.8 (1.883)	93.9 (21.10)	780 (175.3)	0.059	Bolt Rupture
ba3b 1 34	1563 (61.5)	0.176	64.6 (2.544)	74.6 (2.937)	134.1 (30.15)	877 (197.1)	0.146	Beam Web Block Shear Rupture
ba3b 34 14 Offset	1074 (42.3)	0.122	31.7 (1.249)	41.1 (1.619)	33.0 (7.42)	258 (57.9)	0.047	Angle Rupture
ba3b 34 12 Offset	1086 (42.8)	0.116	32.3 (1.273)	39.6 (1.559)	60.5 (13.60)	533 (119.8)	0.066	Bolt Rupture
ba3b 34 14 Gap	1080 (42.5)	0.122	32.0 (1.261)	40.5 (1.595)	30.5 (6.85)	258 (58.0)	0.044	Angle Rupture
ba3b 34 12 Gap	1150 (45.3)	0.122	36.1 (1.423)	41.2 (1.622)	65.2 (14.66)	553 (124.2)	0.071	Bolt Rupture
ba3b 34 14 TopSeat	542 (21.3)	0.062	8.3 (0.325)	-	42.5 (9.55)	137 (30.8)	0.077	Angle Rupture
ba3b 34 12 TopSeat	557 (21.9)	0.063	8.7 (0.344)	-	68.9 (15.48)	46 (10.3)	0.108	Bolt Rupture
ba3b 34 14 HConfig	1328 (52.3)	0.150	47.6 (1.873)	52.9 (2.082) [/]	44.5 (10.01)	322 (72.3)	0.064	Angle Rupture
ba3b 34 12 HConfig	1216 (47.9)	0.138	40.2 (1.582)	41.7 (1.642) [/]	57.8 (13.00)	475 (106.7)	0.063	Beam Web Tearout
ba3b 34 14 BlegWeld	1067 (42.0)	0.121	31.2 (1.229)	-	27.2 (6.12)	240 (54.0)	0.039	Angle Rupture
ba3b 34 14 ClegWeld	1125 (44.3)	0.127	34.6 (1.363)	34.1 (1.342)	15.8 (3.55)	130 (29.3)	0.023	Weld Rupture
ba3b 34 14 Weak	1100 (43.3)	0.125	33.2 (1.305)	-	29.5 (6.63)	231 (52.0)	0.042	Angle Rupture
ba3b 34 12 Weak	1373 (54.1)	0.155	50.6 (1.993)	-	79.6 (17.90)	591 (132.9)	0.087	Angle Rupture and Bolt Rupture

[/]Value corresponds to fiber centered at beam leg bolt.