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Full-scale Tests of Stabilized and Unstabilized Extended Single-plate Connections

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ABSTRACT

Little guidance is available in the literature that is suitable for use in the design of extended single-plate connections. In particular, these connections have not been studied sufficiently for scenarios that involve axial load, which is relatively common in industrial applications. To address this shortcoming, an investigation into the behaviour of extended single-plate connections was completed by testing 23 full-scale specimens. Connections with and without stabilizer plates were tested that varied in plate thickness, plate depth, and the number of horizontal bolt lines. Horizontal loads varied from 500 kN in compression to 200 kN in tension. The influences of the test variables on connection behaviour, capacity, and failure mode are discussed. Among the findings of this research are that despite their slenderness these connections tend to be quite ductile, and the capacities of the unstabilized connections without axial load were much larger than those predicted by available design provisions.

1. Introduction

Extended single-plate connections have the same configuration as conventional single-plate connections, but normally frame into the supporting member's web and extend beyond its flanges, as shown in Fig. 1. This creates a larger geometric eccentricity, e, the distance between the face of the support and the bolt group centroid, that must be accounted for explicitly in design. The recommended limit on eccentricity for conventional single-plate connections is 89 mm (3.5 in.) according to the *Steel Construction Manual* [1], whereas the extended type can have eccentricities up to 360 mm (14 in.) in many practical applications. The effect of this large load eccentricity on the stability and performance of the plate is investigated herein. Since axial loads are common in steel shear connections, and an understanding of their influence on connection behaviour is important to design, this parameter was included in the research program. Finally, extended single-plate connections can be stabilized by welding the plate either directly to the flanges of a girder or to perpendicular stabilizer plates located between the flanges of a column; thus, both stabilized and unstabilized plate behaviour was studied.

2. Literature review

2.1. Shear connection rotation

Single-plate connections are typically classified as simple, or shear, connections. Based on an analysis completed by Astaneh [4], the rotational demand on a shear connection when the plastic moment is reached in a "typical" beam is approximately 0.03 rad. This rotation can be considered a reasonable upper limit for conventionally-loaded shear connections, and is commonly targeted as the maximum rotation during tests of shear connections.

2.2. Conventional single-plate connections

Research into the behaviour of single-plate connections has a long history, and design procedures began to develop in the mid-1900s. Although design procedures had been proposed by Lipson [10] and Richard et al. [14] that were verified and refined by other researchers, the procedure proposed by Astaneh et al. [3] laid the foundation for the design methods used today. Astaneh et al. [2] completed an extensive research program to investigate the behaviour of conventional single-plate connections. Three-, five-, seven-, and nine-bolt connections were tested and the ductility was found to increase with connection depth. The behaviour of single-plate connections was unaffected when short slotted holes were used. Beam-to-girder connection tests were also completed and the flexibility of the support allowed larger

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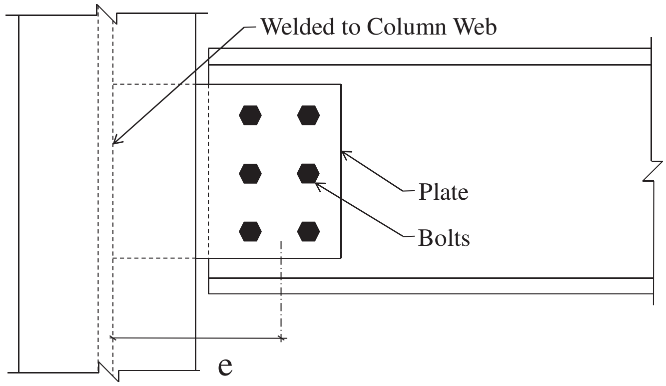

Fig. 1. Extended Single-plate Connection to Column Web.

rotations to occur due to deformation of the girder web. A change in failure mode was evident when results from tests with rigid and flexible support conditions were compared. Bolt fracture was more common for rigidly supported single-plate connections, whereas weld fracture was often observed when the plate was connected to a girder web.

Additional research was completed by Creech [7], who varied several parameters including the support type and number of bolts. It was found that consideration of the eccentricity of the shear force was only required when calculating the bolt group capacity for two- or three-bolt connections and, therefore, connections with more than three bolts could be designed by calculating the direct shear capacity of the bolt group. However, flexible supports were found to reduce the bolt group capacity because the additional rotation from the girder web shed the effect of the eccentricity to the bolt group.

Guravich and Dawe [9] conducted physical tests to examine the behaviour of common shear connections, including single-plate connections, when subjected to combined shear and tension, and in particular they studied the effect of shear force on the connection's ability to resist tension. The tensile capacities of single-plate connections were found after 0%, 50%, and 100% of the factored bolt shear capacity was applied in shear. The tests showed that as long as the shear stress in the plate is less than approximately 50% of the yield stress, the full tensile capacity of the connection is maintained.

A series of eight specimens with more than one vertical bolt line was tested by Marosi et al. [11]. Although a larger number of bolts was used, the connections were able to reach rotations comparable with those of single-line connections.

2.3. Extended single-plate connections

Moore and Owens [13] found that for extended single-plate configurations, large deformations were caused by a combination of vertical plate deformation and rotation. The rotation was significantly larger when the connection was made to the column web instead of the flange. At the ultimate load, extended single-plate connections were more likely to fail at the weld line, whereas shorter plates tended to fail at the bolt line. The extended configuration also had a tendency to twist about the longitudinal axis.

Sherman and Ghorbanpoor [15] conducted an extensive testing program on extended single-plate connections. Thirty-one tests were completed, including six beam-to-column, two beam-to-girder, 14 stabilized beam-to-column, and nine stabilized beam-to-girder connections. Plate twisting was observed during many of the extended single-plate tests when stabilizer plates were not present, as was web yielding in the columns and girders with thinner webs. Buckling of the plate was also observed for two connections in beam-to-girder tests when it was stabilized by welding the plate directly to the top and bottom girder flanges.

A research program on stabilized beam-to-column extended single-plate connections was completed by Goodrich [8]. In both physical tests and finite element simulations the plates were stiffened at the top and bottom to determine how the stabilizer plates influence the connection behaviour and if a reduced eccentricity can be used in design. The capacities of the three geometries tested were more than twice those predicted by the design procedure the researchers had proposed, and exceeded the AISC allowable stress design safety factor of 1.67.

The use of multiple vertical bolt lines was investigated by Metzger [12]. Four beam-to-column connection tests were completed with the plates welded to the column flange instead of the column web. The connections either failed due to weld fracture or the beam failed prior to connection failure. The welds that failed were smaller than those recommended by AISC [1] for single-plate connection design.

Research investigating the behaviour of extended single-plate connections is extremely limited, especially for the case of multiple vertical bolt lines in the connection to the beam web. While design provisions exist for extended single-plate connections [1], they have not been substantiated sufficiently with full-scale connection tests.

3. Experimental program

In order to gain a better understanding of the behaviour of extended single-plate connections, 23 full-scale tests were carried out. A key aspect of this test program is the presence of axial load because, although the stability of this type of connection has always been a design concern, no prior tests had been completed considering the effect of an axial compressive or tensile force. In addition to varying the magnitude of tension or compression, the program also investigates the influence on connection behaviour of plate thickness, number of horizontal bolt lines, and the presence or absence of stabilizer plates. The research was limited to an investigation of local connection behaviour, rather than that of the beam as a whole.

3.1. Test specimens

The specimens were fabricated from ASTM A572 Grade 50/CSA-G40.21 Grade 350W steel and, as shown in Fig. 2, differ in the number of horizontal bolt lines, the plate thickness, and the use of stabilizer plates. These three variables are used to identify specimen groups with the same geometry using an alphanumeric I.D. that begins with 2B, 3B, or 5B, depending on the number of horizontal bolt lines. The plate thickness to the nearest millimeter, either 10 or 13, follows to represent 10 mm (3/8 in.) and 13 mm (1/2 in.) plates, respectively. Finally, if the specimen is unstabilized (unstiffened), a "U" follows the plate thickness or, if stabilizer plates (stiffeners) are present, an "S" is used to complete the group I.D. For example, the stabilized specimens with two horizontal bolt lines and a 13 mm (1/2 in.) thick plate are identified as specimen group 2B-13-S.

The magnitude of the horizontal load applied to the specimen, in kilonewtons, is added to the end of the specimen group I.D. to differentiate among test results of specimens with the same geometry. This force is followed by a "C" if it was tested under compression, a "T" if under tension, or nothing if no horizontal load was applied. One test was repeated, except the beam was not rotated to investigate the influence of beam rotation on the connection capacity. Because there was no horizontal force, this test is identified by adding an additional zero to the end of the I.D. (2B-10-U-00) to distinguish it from the test conducted with the beam rotated to 0.03 rad.

To determine how the variables described above influence the behaviour of extended single-plate connections, typical sizes and dimensions were chosen for the remaining parameters based on the extensive experience of a major local steel fabricator, including the column size, connection plate length (parallel to the beam), bolt diameter, number of vertical bolt lines, fillet weld size, and stabilizer plate configuration, as shown in Fig. 2. Because it is common in the design of extended single-plate connections, the specimens in this program have two vertical bolt lines. The

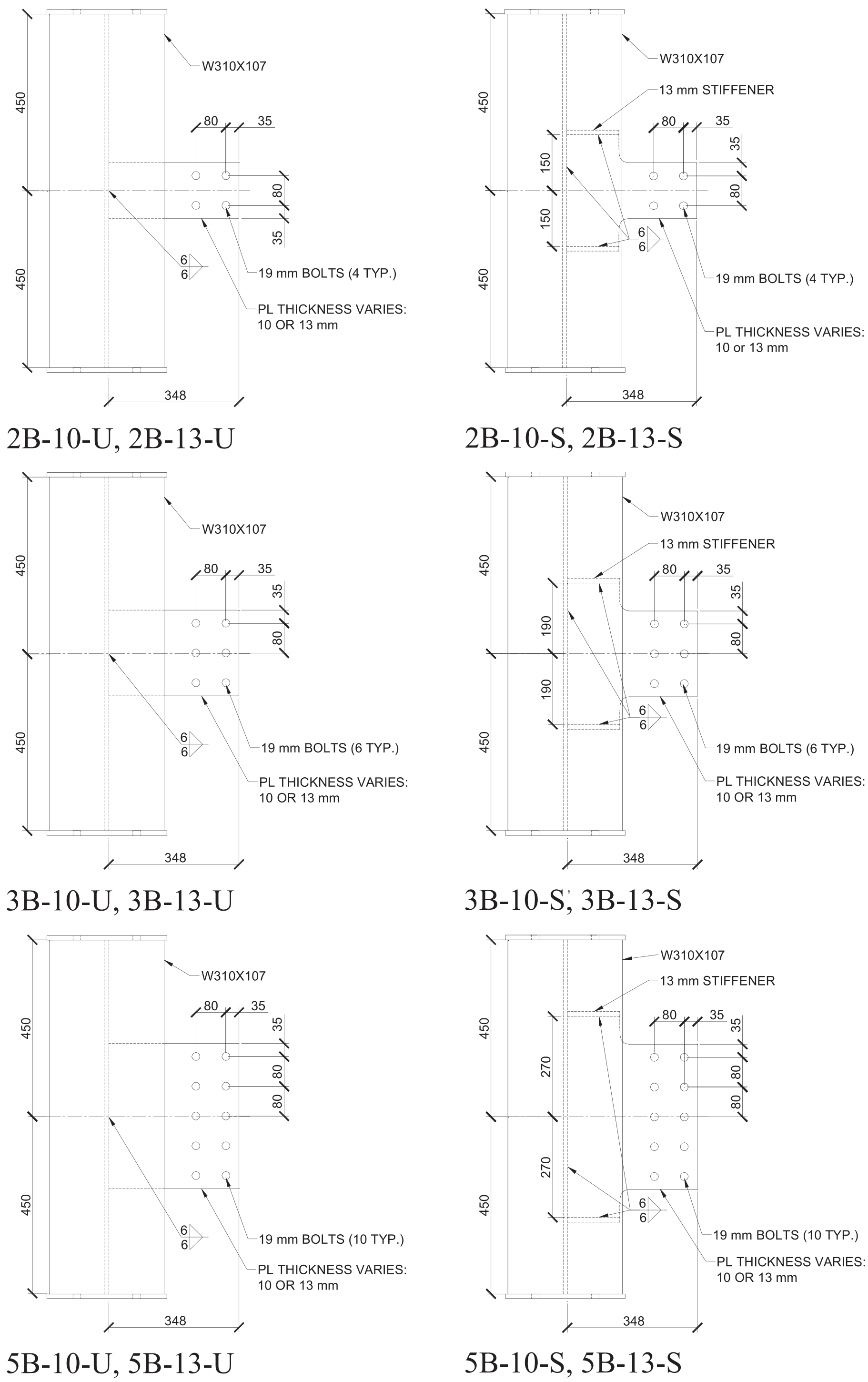

Fig. 2. Specimen Geometries.

number of horizontal bolt lines was varied not only to determine the influence of this variable on the capacity, but also to test different plate depth-to-length ratios. Two, three, and five horizontal bolt lines were selected to cover a reasonable range commonly used in buildings. Standard bolt holes for 19 mm (3/4 in.) diameter ASTM A325 bolts were used in all specimens, 2 mm (1/16 in.) greater than the nominal bolt diameter, and a consistent 80 mm (3–3/16 in.) spacing was used between both the vertical and horizontal bolt lines with 35 mm (1–3/8 in.) end and edge distances. The plate thicknesses, 10 mm (3/8 in.) and 13 mm (1/2 in.), are commonly used for extended single-plate connections. A 6 mm (1/4 in.) fillet weld was selected to connect the plates to the column even though it does not satisfy the minimum weld size for the 13 mm (1/2 in.) plates to see if this efficient-to-fabricate weld would negatively affect the connection's strength or ductility.

The plate length and eccentricity, e, are identical for all specimens. In practice, there are no typical values for these dimensions because they depend on the column size, the required gap between the beam and column flanges, the number of vertical bolt lines, the vertical bolt line spacing, and the edge distance. Therefore, typical sizes and dimensions were specified for these parameters and, with the exception of the gap distance, are shown in Fig. 2. A gap distance of 50 mm (2 in.) was selected to represent a “worst case” scenario—when fireproofing must be accommodated between the column and beam—and is a reasonable estimate of the maximum plate length typically found in practice. As a result, a 348 mm (13–11/16 in.) long plate was used, resulting in a geometric eccentricity of 273 mm (10–3/4 in.).

The stabilizer plate configuration was chosen to represent typical dimensions used in practice in industrial applications where the stabilizer plates also act as joint reinforcement for moment connections that frame into the column flange. The stabilizer plates had a 13 mm (1/2 in.) thickness and were offset from the top and bottom of the connection plates by a clear distance of 75 mm (3 in.). The radius of the re-entrant corners where the plate depth changes was 25 mm (1 in.) and the stabilizer plate width was 115 mm (4–1/2 in.). Double-sided, 6 mm (1/4 in.) fillet welds were specified to connect the stabilizer plates to the connection plate, the column web, and the column flanges. The stabilizers were welded to the column web as they also represent joint stiffeners for perpendicular moment connections. Note that it is common for there to be an offset between the stabilizer plates and the top and bottom edges of the connection plate when the beam framing into the column web is not the same depth as the space between the stiffeners used for moment connections to the column flanges; the 75 mm (3 in.) offset chosen was used as a representation of the worst case in a typical design situation.

Each extended single-plate connection was supported by a 900 mm long (35–7/16 in.), W310 × 107 (W12 × 72) column stub. A 13 mm (1/2 in.) column cap plate was welded onto the top and bottom of the column stub for installation of the specimen into the test set-up. The connection plates were centered vertically on the column web and offset horizontally by one-half of the total beam-web-plus-connection-plate thickness to keep the beam and column axes aligned. For the two and three horizontal bolt line specimens, a W310 × 129 (W12 × 87) beam was used, whereas for the deeper, five bolt line specimens, a W530 × 165 (W21 × 111) beam was used. The beams were reused within each family of specimens and were selected with thick webs to minimize beam web bolt hole distortion.

3.2. Test set-up

The test set-up was designed to allow any combination of vertical load, horizontal load, and rotation to be applied to the connection. As shown in Fig. 3, the set-up consisted of interchangeable column stub specimens (with the connection plates pre-installed in the fabrication shop), a beam, and three actuators. Actuator 1 was used to apply vertical load near the connection (203 mm (8 in.) from the center of the test connection bolt group), while load from Actuator 2 primarily controlled the rotation of the beam (1020 mm (40–3/16 in.) from the center of the test connection bolt group). The specified horizontal load for each test was applied using Actuator 3. Actuators 1 and 2 were anchored to the lab's strong floor, while Actuator 3 was connected to a stiff shear wall.

The column stub was bolted to heavy brackets through cap plates using four 25 mm (1 in.) diameter pretensioned bolts at the top and bottom to prevent the column stub from slipping. The supporting column was anchored to the strong floor and braced diagonally to the floor and also back to the shear wall to prevent movement. Bracing on each side of the beam near Actuator 1 combined with robust bracing of Actuator 3, which was actually two rigidly-connected actuators in parallel, prevented any lateral movement of the beam during the test.

3.3. Instrumentation

Load cells, clinometers, cable transducers, pressure transducers, and linear variable differential transformers (LVDTs) were used to monitor the behaviour of the extended single-plate connections and the testing assembly. In addition, a digital image correlation camera system was used to map the three-dimensional displacements and strains of the connection specimens.

A load cell, cable transducer, and clinometer were mounted on each actuator, allowing the forces from the load cells to be separated into vertical and horizontal components. By adding the three vertical and three horizontal load components, the total load on the connection in each principal direction was calculated. Redundant load measurements were taken by pressure transducers connected to the hoses of the hydraulic actuators. Beam rotation was measured both directly, using a clinometer on the beam web, and indirectly using the cable transducers and clinometers on Actuators 1 and 2. Another clinometer was placed on the connection plate as close to the weld line as possible to get an indication of the column web rotation. Overall movement of the column stub was also monitored, but found to be negligible.

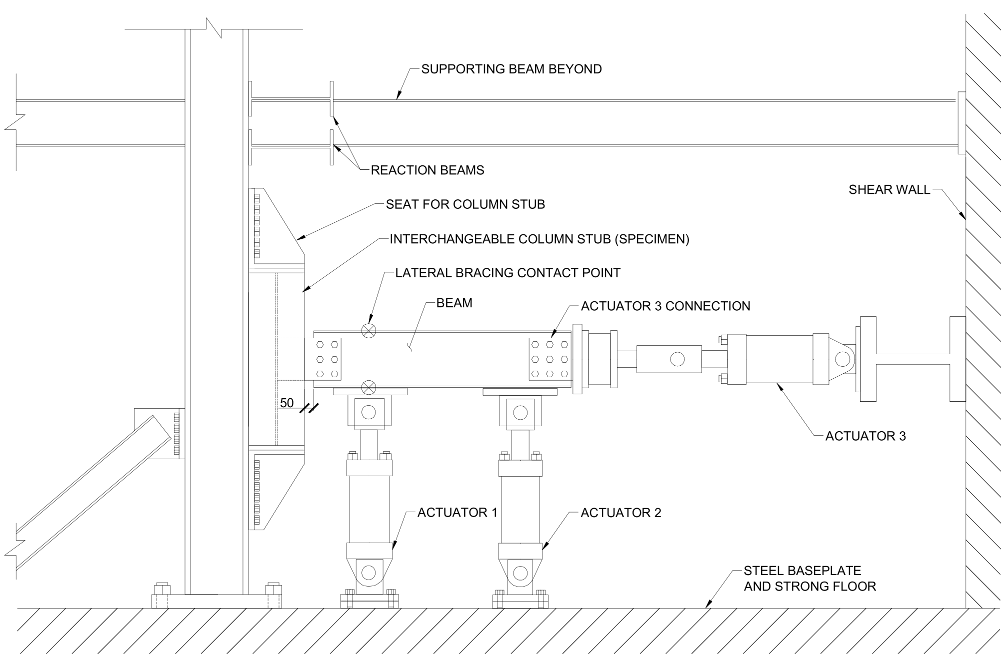

Fig. 3. Test Set-up.

3.4. Test procedure

Once the connection plate was bolted to the beam web to a snug-tight condition, the beam was rotated upward by 0.03 rad, since the connection was tested in an inverted configuration. While rotating the beam, the total horizontal and vertical loads on the connection were kept at or near zero, while the moment that developed remained small due to the flexibility of the connection. The horizontal load was then applied while the rotation was held constant and the vertical load was kept at or near zero. Once the desired horizontal load had been applied, both it and the beam rotation were held constant while upward vertical displacement was induced until failure occurred and the vertical load had decreased substantially from its peak value. All loads were applied quasi-statically. Throughout testing and after the load was removed, the specimen was examined and any visible deformations or failure modes were recorded. It is doubtful that there are many cases where a beam would be loaded in this manner in a real-world situation; however, this loading sequence was chosen as a reasonable worst case scenario for the variety of loading conditions found in practice and is consistent with previous research on beam shear connections.

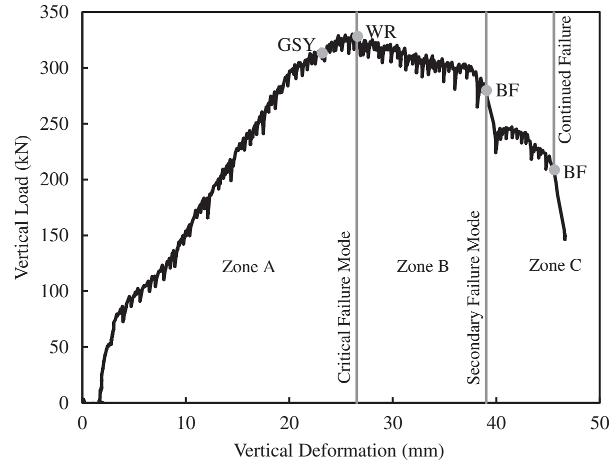

Fig. 4. Load–Deformation Response of Specimen 3B-10-U-0.

4. Results

4.1. Material tests

4.1.1. Plate coupon tests

Tension coupons were fabricated from plates that were cut from the same source plate as was used for the extended single-plate specimens. For each plate thickness, four material samples were taken and pairs were cut at 90° to each other to assess the material properties in the two principal directions, but the observed differences are not considered to be significant. Tests were conducted in accordance with ASTM Standard A370 [5]. The mean yield strengths were 455 MPa (66 ksi) and 418 MPa (61 ksi) for the 10 mm (3/8 in.) and 13 mm (1/2 in.) plates, respectively, and the mean associated ultimate strengths were 507 MPa (74 ksi) and 470 MPa (68 ksi). The rupture elongation was 24% for both plates on a gauge length of 50 mm.

4.1.2. Bolt shear tests

To determine the shear capacity of the bolts used to fasten the connection plate to the beam web, six bolt shear tests were conducted with the bolt in single shear to mimic the test conditions. To ensure the shear plane was in precisely the same location as it was during the tests, three tests were completed for each plate thickness. In both cases, the nut was against the beam web and the shear plane was in the thread run-out region of the bolt, but for the thicker plate it was near the minimum diameter and for the thinner plate it was on the boundary between the thread run-out and the unthreaded shank. On average, the bolt shear strength was 177 kN (40 kip) for the 10 mm (3/8 in.) plate tests and 159 kN (36 kip) for the 13 mm (1/2 in.) plate tests.

4.2. Unstabilized extended single-plate connection tests

The behaviour of the 13 unstabilized extended single-plate connections can be explained by dividing the vertical load–vertical deformation graphs into three zones—A, B, and C—delineated by the critical and secondary failure modes. A typical response curve (specimen 3B-10-U-0) is shown in Fig. 4. The deformation plotted in the figure is the vertical displacement of the bolt group centroid on the beam with respect to its original position.

In zone A, the 0.03 rad beam rotation is imposed, horizontal load is then applied (if required), and the vertical load is applied subsequently. During the beam rotation and application of horizontal load, a small amount of vertical connection deformation was recorded (approximately 2 mm (1/16 in) during beam rotation in Fig. 4). During the application of the 0.03 rad pure rotation, the bolts were seated into bearing against the bottoms of the connection plate holes nearer to the column and the tops of the holes farther from the column, and there was a small amount of deformation in the connection plate. No significant yielding occurred in either the connection plate or the column web. As the vertical load increased, a slight increase in stiffness was observed. As the deformation of the column web at this point was small, a change from yield line behaviour to a stiffer mechanism is not the reason for this change. While the reason is not known conclusively, since the system is flexible and in this case the vertical deformation following the rotation is only about 5 mm, it is surmised that it corresponds to the development of full engagement of the bolts against the tops of the holes in the connection plate. Subsequently, stiffness reductions occurred as the connection elements—primarily the column web—began to yield until, in most cases, the connection plate yielded through its depth causing the stiffness to diminish rapidly. Zone A ends at the point where the critical failure mode occurs, defined as the point where the vertical load begins to decrease after its peak value.

After initial failure occurred, load redistribution began in zone B. When weld rupture was the critical failure mode, the load decreased gradually as the rupture propagated, as is the case in Fig. 4. The vertical load tended to drop more suddenly when bolts fractured. In other cases, the load remained relatively constant until secondary failure occurred. The extent of deformation between the critical and secondary failure modes varied considerably among tests.

In zone C, the vertical load continued to drop while the bolts progressively failed and/or the weld rupture propagated. Again, the

Table 1
Maximum Recorded Vertical Loads, kN (kip) – Unstabilized Specimens.

Specimen group	200 T	Axial load, kN 0	00[a]	200C	300C
2B-10-U	–	188 (42)	197 (44)	159 (36)	–
2B-13-U	–	–	–	138 (31)	–
3B-10-U	270 (61)	330 (74)	–	339 (76)	278 (62)
3B-13-U	–	–	–	263 (59)	–
5B-10-U	612 (138)	762 (171)[b]	–	–	732 (165)
5B-13-U	–	–	–	–	613 (138)

[a] No axial load/no beam rotation.
[b] Actuator 1 capacity reached.

number and extent of the load drops in this zone varied. During the 3B-10-U-0 test, two bolts failed, shown in Fig. 4 as the secondary failure mode and "continued failure" points.

The maximum vertical loads recorded during testing are shown in Table 1 for the 13 extended single-plate connections without stabilizer plates. While testing specimen 5B-10-U-0, the capacity of Actuator 1 was reached and 142 kN (32 kip) of horizontal tension was then applied to induce failure, in this case tearing of the column web. As the connection had already displayed considerable softening by the time the actuator capacity was reached, the effect of the axial force on the vertical load capacity is considered to be small and the vertical load reported in Table 1 can be treated as a reasonable, but lower-bound, estimate of the connection's true capacity with no axial load.

As a point of comparison, design strengths can be calculated for the three cases without axial load for which there are associated design provisions in the *Steel Construction Manual* [1]. The design strengths of specimens 2B-10-U-0/2B-10-U-00, 3B-10-U-0, and 5B-10-U-0 are 60 kN, 119 kN, and 202 kN, respectively. The associated peak vertical loads from the tests are 188/197 kN, 330 kN, and 762 kN, respectively, indicating that these provisions can be quite conservative.

The capacities of specimens 2B-10-U-0 and 2B-10-U-00 (identical, but with and without rotation, respectively) had similar capacities, but the application of rotation appears to be a slightly more severe loading condition based on these tests. The capacities of the extended single-plate connections increased with increasing connection depth and decreased with increasing plate thickness. The peak vertical loads were lower for the thicker plate by 13%, 22%, and 16% for the two, three, and five horizontal bolt line connections, respectively, where direct comparisons can be made (with the same axial force). Although these results may seem counter-intuitive, the connection stiffness was increased by increasing the plate thickness, in turn reducing plate deformation and increasing the demands on both the welds and the bolt group. In addition to the increased demand on the bolt group, the shear plane through the bolts of specimens with the thicker plate was closer to the minimum diameter in the threads, as discussed previously, resulting in a 10% lower bolt strength.

In general, horizontal load tended to reduce the vertical load-carrying capacity of the connections. In the case of the 3B-10-U specimens, the addition of 200 kN (45 kip) of compression actually increased the shear capacity of the connection by 9 kN (2 kips), but when 300 kN (67 kip) was applied the capacity was reduced considerably. This result indicates that there may be a limiting value of compression below which the shear capacity of the connection is unaffected. In contrast, the addition of tension caused the largest decreases in peak vertical load because more demand was placed on the tension side of the weld, a location of failure for these connections.

The commonly observed failure modes for the extended single-plate connections tested were weld rupture, bolt fracture, and column web yielding. In nearly all cases, the connection plates underwent plastic deformations due to gross section yielding and/or limited deformations at the bolt holes due to bolt bearing. However, the critical failure mode (CFM)—considered to be that which caused the initial post-peak decrease in vertical load—tended to be bolt fracture or weld rupture. A detailed list of the failure modes observed during and after testing is shown in Table 2. Potential failure modes include weld rupture (WR), bolt fracture (BF), column web yielding (CWY), gross section yielding (GSY), net section fracture (NSF), bolt bearing (BB), and out-of-plane deformation (OPD). Whether or not gross section yielding occurred during test 3B-13-U-200C cannot be verified explicitly due to a problem with data collection from the camera system (as a result, it is marked "N/A" in Table 2), although gross section yielding is believed to have occurred. The failure modes of specimens 2B-10-U-0 and 2B-10-U-00 (with and without rotation) are identical.

Even if it did not cause a drop in vertical load, some extent of weld rupture was evident in every test specimen, except for the specimen that experienced column web tearing instead, i.e., 5B-10-U-0. The rupture would begin at the tension tip of the weld and in some cases propagate significantly towards the compression side, while in others it arrested after a very short distance. If the tip fracture did not propagate, it was considered a local effect and the weld performance was deemed acceptable. Weld rupture tended to occur when the specimens were tested without axial load or with axial tension. For both specimens subjected to tension, weld rupture was the critical failure mode and it should be noted that both of these cases, plus the two others where weld rupture was the critical failure mode, had the thinner plate for which the AISC minimum weld size criterion is met. Moreover, in none of the three cases with thicker plates, where the minimum weld size criterion is not met, did weld rupture occur as the critical or a secondary failure mode.

In many cases, the initial decrease in vertical load is attributed to the fracture of one or more bolts, as shown in Fig. 5. With the exception of the test of specimen 3B-10-U-200T, during which no bolts fractured, this failure mode occurred during every test, although in some cases it was a secondary failure mode. In these tests, after the primary decrease in vertical load the forces were redistributed in order for the connection to reach a new equilibrium state. This redistribution of forces eventually led to an increased bolt shear that caused failure, either immediately after the initial load dropped or after a plateau at a lower vertical force.

Column web yielding was observed during 12 of the 13 unstabilized extended single-plate tests and in one case, specimen 5B-10-U-0, the column web tore. The column webs were whitewashed prior to testing

Table 2
Observed Failure Modes – Unstabilized Specimens.

	Failure modes						
Specimen I.D.	WR	BF	CWY	GSY	NSF	BB	OPD
2B-10-U-0	✓	CFM	✓	✓	–	–	–
2B-10-U-00	✓	CFM	✓	✓	–	–	–
2B-10-U-200C	–	CFM	✓	✓	–	–	–
2B-13-U-200C	–	CFM	✓	✓	–	–	–
3B-10-U-0	CFM	✓	✓	✓	–	–	–
3B-10-U-200C	CFM	✓	✓	✓	–	–	–
3B-10-U-300C	–	CFM	✓	✓	–	–	–
3B-10-U-200T	CFM	–	–	–	–	–	–
3B-13-U-200C	–	CFM	✓	N/A	–	–	–
5B-10-U-0	–	✓	CFM[a]	✓	–	–	–
5B-10-U-300C	–	CFM	✓	✓	–	–	✓
5B-10-U-200T	CFM	✓	✓	✓	–	–	–
5B-13-U-300C	–	CFM	✓	✓	–	–	–

[a] Column web tearing.

Fig. 5. Bolt Fracture in Specimen 3B-10-U-0.

Fig. 6. Column Web Lüders' Lines at the Compression Edge of the Plate.

to determine the column web yield pattern. The whitewash revealed Lüders' lines extending radially on the column web from the top and bottom of the plate, as shown in Fig. 6. Specimen 3B-10-U-200T, which did not exhibit signs of column web yielding prior to the weld rupturing, was the only exception.

Although evidence of localized connection plate yielding was observed on all specimens, it was not responsible for a decrease in vertical load. Plate yielding occurred along the vertical bolt line closer to the end of the beam such that the plate was bending around the bolt group, causing a discrete kink in the plate at this location (visible in Fig. 5, although more pronounced in some other specimens). Transverse necking of the plate through its thickness was evident where it bent around the bolt group in five cases, 2B-10-U-0, 2B-10-U-00, 3B-10-U-0, 5B-10-U-0, and 5B-10-U-200T, indicating that net section fracture would have occurred along this line had it not been pre-empted by another mode. Because the necking deformation occurred in the region of the plate that was primarily subjected to tension, these deformations were not observed when the connection was subjected to horizontal compressive loads. Necking was not observed on specimen 3B-10-U-200T due to the high tensile force on the weld during this test; the only failure mode observed was weld rupture.

Gross section yielding is one mechanism that can be used in design to ensure that the connection has sufficient ductility. The vertical load at which the plate yielded through its full depth is given in Table 3 for each specimen other than 3B-10-U-200T, which experienced only localized yielding, and 3B-13-U-200C, for which insufficient data are available (both are marked “N/A” in Table 3). This point was determined by analyzing the strain field on the surface of the plate revealed by the camera system. The data shown in Table 3 verify that full-depth yielding of the connection plate occurred in eight of 11 (2B-10-U-00 has been removed from Table 3 due to its non-standard loading regime and 3B-13-U-200C is neglected due to lack of data) unstabilized connection tests prior to the peak load being reached, which is a desirable response as it promotes ductility. The yield line in the plate was always in the same location: along the vertical bolt line closer to the end of the beam.

Table 3
Vertical Load, kN (kip) Causing Yielding Through Plate Depth – Unstabilized Specimens.

Specimen group	Axial load, kN 200T	0	200C	300C
2B-10-U	–	160 (36)	127 (29)	–
2B-13-U	–	–	124 (28)	–
3B-10-U	N/A	311 (70)	302 (68)	273 (61)
3B-13-U	–	–	N/A	–
5B-10-U	569 (128)	670 (151)[a]	–	615 (138)
5B-13-U	–	–	–	595 (134)[a]

[a] Occurred after peak vertical load.

Varying degrees of bolt bearing deformation were observed on all the specimens without stabilizer plates. This was not a critical failure mode for any specimen, but rather contributed to the overall ductility of the connections. The maximum permanent bearing deformation observed was 8.6 mm (5/16 in.) in specimen 2B-10-U-0, which had a 10 mm (3/8 in.) connection plate. In contrast, the specimens with a 13 mm (1/2 in.) plate experienced less than 1 mm (1/32 in.) of deformation.

Out-of-plane deformation was not a common failure mode for the unstabilized extended single-plates tested. In previous testing programs, Moore and Owens [13] and Sherman and Ghorbanpoor [15] observed plate twisting. However, because the beams were braced next to the connection, plate twisting did not occur in this testing program. Specimen 5B-10-U-300C exhibited plastic out-of-plane deformation, indicative of the onset of plate buckling failure, not twisting.

4.3. Stabilized extended single-plate connection tests

The behaviour of the ten stabilized extended single-plate connections can be explained by dividing the vertical load–vertical deformation graphs into three zones—A, B, and C—delineated by the beginning and end of the load-carrying capacity plateau. A typical response curve (specimen 2B-10-S-0) is shown in Fig. 7. The deformation plotted in the figure is the vertical displacement of the bolt group centroid on the beam with respect to its original position.

Zone A contains the beginning of the test, when the beam was rotated to 0.03 rad and horizontal load was applied, if required. During the application of the 0.03 rad rotation the bolts were seated into bearing and there was a small amount of deformation in the connection plate. No significant yielding occurred in either the connection plate or the column components. Vertical displacement was then gradually applied and the plate began to yield, typically in the area around the radius in compression. As the plate continued to yield, it began to deform out-of-plane until the vertical load no longer increased. For some tests, the plate yielded along its full depth prior to deforming out-of-plane. The point of maximum vertical load marks the end of zone A.

After the peak vertical load was attained, the plate continued to deform out-of-plane while remaining at or near the peak load. The length of this load plateau varied among tests, but the vertical load eventually began to decrease, marking the end of zone B. During the tests of specimens 5B-10-S-300C and 5B-13-S-500C, the beginning of zone B was reached prior to reaching the capacity of Actuator 1, but the end of zone B was not. (The force in Actuator 1 needed to be increased to maintain the 0.03 rad beam rotation while the specimen deformed, even though the net vertical load on the connection was not increasing.) In

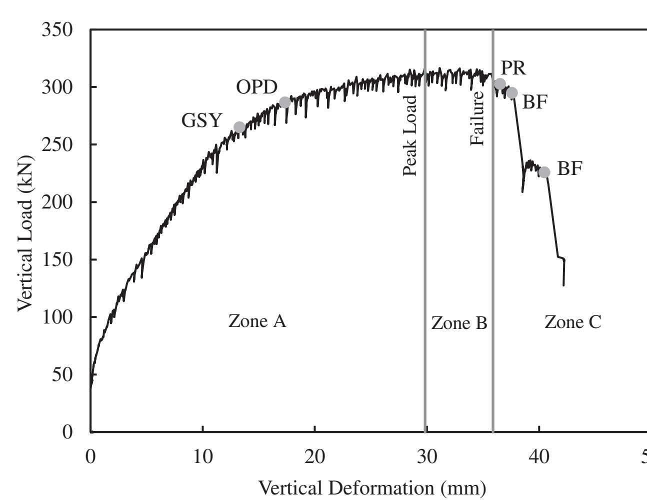

Fig. 7. Load–Deformation Response of Specimen 2B-10-S-0.

the case of specimen 5B-13-S-500C, an additional 50 kN (11 kip) of compressive load was applied to fail the connection.

Zone C begins when the vertical load starts to decline steadily. The decrease in vertical load was either gradual, and the test was then terminated when half of the peak vertical load was reached, or bolt fracture occurred, which caused a much more abrupt load decline. Zone C in Fig. 7 is an example of the latter; the load begins to decrease gradually until two bolts progressively fail. The maximum vertical loads recorded during testing of the ten extended single-plate connections with stabilizer plates are shown in Table 4.

In contrast to those without stabilizer plates, stabilized extended single-plate specimens obtained higher peak loads when the thicker plate was used. This higher capacity was achieved because the thicker connection plate was less susceptible to out-of-plane deformation, which was the critical failure mode for all of these specimens. The peak vertical load resisted by the stabilized extended single-plate specimens decreased significantly with the addition of compression. With the application of 200 kN (45 kip) of compression, the capacity of the 2B-10-S specimen group decreased by 59 kN (13 kip) or 19%, and the 3B-10-S group by 129 kN (29 kip) or 25%. Increasing the compressive horizontal load from 300 kN (67 kip) to 400 kN (90 kip) resulted in a decrease of 212 kN (48 kip) or 27% for the 5B-10-S specimen group. Because the stabilized specimens typically deformed out-of-plane, applying axial compression accelerated the deformation and decreased the shear capacity of the connections.

The influence of stabilizer plates on connection strength is shown in Table 5 by comparing specimens that differed only by the presence or absence of stabilizers. Both the unstabilized and stabilized maximum vertical loads, as well as the increase in capacity due to the addition of stabilizers, are presented. The “U” or “S” has been removed from the specimen I.D. in the table for ease of comparison. This comparison shows that stabilizer plates increased the capacities of the connections, especially for specimens with two horizontal bolt lines and/or the 13 mm (1/2 in.) plate. However, the increase in capacity decreases with the addition of horizontal compression. For example, for the 3B-10-300C geometry the addition of stabilizer plates did not increase the connection capacity significantly, although the critical failure mode did change from bolt fracture to out-of-plane deformation.

A summary of the failure modes observed for each stabilized extended single-plate specimen is given in Table 6. Potential failure modes include weld rupture (WR), bolt fracture (BF), column web yielding (CWY), gross section yielding (GSY), bolt bearing (BB), and out-of-plane deformation (OPD). No indication of net section fracture was observed on any specimen, but plate rupture (PR) was observed in the radius on the tension side of the plate in two instances.

Weld rupture did not occur during the stabilized extended single-plate connection tests, as a stress concentration at the tip of the weld was prevented by the stabilizer plates. Similarly, significant column web yielding was not observed during any of the stabilized extended single-plate connection tests.

The tests show that moment is attracted to the column end of the connection when stabilizer plates are present, and this moment is transferred to the column as a force couple through the stabilizers. As this moment can be considerable, it must be taken into account during design of the column. Further research is needed to help quantify the appropriate eccentricity to be used to evaluate the design moment, but the inflection point for the stabilized connection plate at the connection's capacity was typically half way between the tip of the stabilizer and the centerline of the bolt group. The extra moment being attracted to the column enabled the stabilized connections to reach a large enough load that plate out-of-plane deformation became the failure mechanism. The stabilizer plates in essence shorten the “span” of the plate, requiring a much more severe curvature than is required for the unstabilized case with a longer plate, for the same rotation and displacement. This sharp curvature cannot be accommodated through in-plane bending, so out-of-plane deformation is induced. Fig. 8 shows the typical mechanism that formed in the stabilized single-plate connections.

In six of the ten tests, bolt fracture was a secondary failure mode. As the plates deformed out-of-plane, the bolts were subjected to an increasing tensile prying action in addition to the horizontal and vertical loads. The prying force was a result of the double column of bolts imposing a clamped boundary for the out-of-plane deformation of the connection plate. It is this out-of-plane deformation of the laterally-stiff connection plate (always in the direction away from the beam web) that creates a severe bolt-prying condition, especially in the first vertical line of bolts. Therefore, with the exception of the three and five bolt line specimens with 10 mm (3/8 in.) plates and high axial load, at least some bolts failed. The bolts in the vertical line closer to the end of the beam typically failed, as they were subjected to a higher proportion of the tensile load caused by the plate deformation. The severity of this failure mode varied; for example, the 3B-13-S-200C test ended abruptly, with the entire bolt group rupturing at once, whereas during the 5B-13-S-500C test the bolts failed progressively.

Although not the critical failure mode, gross section yielding was observed in all cases. The specimens with two horizontal bolt lines yielded through their depth prior to the plate deforming out-of-plane, but the vertical load continued to increase. At the point where the out-of-plane deformation began, the load plateaued while the plate folded and, ultimately, the load began to decrease.

The specimens tested without horizontal compression—specimens 2B-10-S-0 and 3B-10-S-0—both tore at the radius cut of the plate on

Table 4
Maximum Recorded Vertical Loads, kN (kip) – Stabilized Specimens.

Specimen group	Axial load, kN				
	0	200C	300C	400C	500C
2B-10-S	317 (71)	258 (58)	–	–	–
2B-13-S	–	323 (73)	–	–	–
3B-10-S	511 (115)	382 (86)	279 (63)	–	–
3B-13-S	–	562 (126)	–	–	–
5B-10-S	–	–	798 (179)[a]	586 (132)	–
5B-13-S	–	–	–	–	861 (194)[a]

[a] Actuator 1 capacity reached on peak load plateau.

Table 5
Effect of Stabilizer Plates on Connection Capacity.

Specimen I.D.	Max. vertical load, kN (kip)		Percent increase
	Unstabilized	Stabilized	
2B-10-0	188 (42)	317 (71)	68.6%
2B-10-200C	159 (36)	258 (58)	62.3%
2B-13-200C	138 (31)	323 (73)	134.1%
3B-10-0	330 (74)	511 (115)	54.8%
3B-10-200C	339 (76)	382 (86)	12.7%
3B-10-300C	278 (62)	279 (63)	0.4%
3B-13-200C	263 (59)	562 (126)	113.7%
5B-10-300C	732 (165)	798 (179)	9.0%

Table 6
Observed Failure Modes – Stabilized Specimens.

Specimen I.D.	Failure modes						
	WR	BF	CWY	PR	GSY	BB	OPD
2B-10-S-0	–	✓	–	✓	✓	–	CFM
2B-10-S-200C	–	✓	–	–	✓	–	CFM
2B-13-S-200C	–	✓	–	–	✓	–	CFM
3B-10-S-0	–	✓	–	✓	✓	–	CFM
3B-10-S-200C	–	–	–	–	✓	–	CFM
3B-10-S-300C	–	–	–	–	✓	–	CFM
3B-13-S-200C	–	✓	–	–	✓	–	CFM
5B-10-S-300C	–	–	–	–	✓	–	CFM
5B-10-S-400C	–	–	–	–	✓	–	CFM
5B-13-S-500C	–	✓	–	–	✓	–	CFM

Fig. 8. Out-of-plane Deformation of Stabilized Specimen.

the flexural tension side, as shown in Fig. 9. Cheng [6] identified radius cuts as potential fatigue crack initiation sites in coped beams, where tears result from a combination of the stress concentration due to the cross-sectional discontinuity and tensile residual stresses if flame-cutting is used. A similar stress concentration exists in the stabilized extended single-plate connections and produces a location of potential tearing, even under non-cyclic loading. However, the tears in both specimens began after the peak load and significant deformation had already occurred. Compression in the connection mitigates the potential for tearing.

Varying degrees of bolt bearing deformation were observed on the stabilized connection plates. Many of the maximum deformations were less than or approximately equal to 1 mm (1/32 in.); the largest deformation, 5.7 mm (7/32 in.), occurred in specimen 3B-10-S-0.

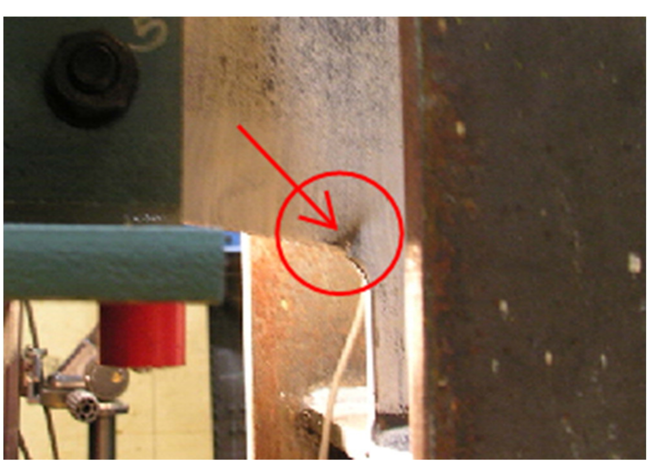

Fig. 9. Plate Tearing at Corner Radius.

5. Summary and conclusions

The capacity of an extended single-plate connection can, in general, be increased significantly by using stabilizer plates. If stabilizer plates are not used, the capacity can instead be increased by using a deeper plate with more bolts. However, increasing the plate thickness does not necessarily strengthen the connection, as it may reduce ductility and thereby trigger another failure mode; this concept is reflected in the design procedures provided in the *Steel Construction Manual* [1].

When stabilizer plates are used, additional shear capacity can be realized by increasing the plate depth and number of bolts, or the plate thickness (contrary to the observations for the unstabilized specimens). As out-of-plane deformation of the connection plate governed the ultimate strength of the stabilized extended single-plate connections, the reduction in shear strength with the addition of compressive load was observed to be more rapid than for the configuration without stabilizer plates, whose capacity may not decrease at all under small horizontal compressive loads.

Eight potential failure modes were studied in this research and should be considered during design: weld rupture, bolt fracture, column web yielding, plate rupture, gross section yielding, net section fracture, bolt bearing, and out-of-plane deformation. Weld rupture and bolt fracture were the dominant observed critical failure modes for extended single-plate connections without stabilizer plates, and for those with stabilizers it was out-of-plane deformation. For both stabilized and unstabilized single-plate connections, it is desirable to proportion the connection to ensure yielding of the plate cross-section prior to the ultimate load capacity being reached, as plate yielding plays a significant role in achieving ductile connection behaviour. This yielding occurs as a result of the combination of flexural and shear actions. While bolt bearing played a role in several of the tests, in no case was bolt tear-out observed. In addition to the failure modes observed in the tests, other potential modes such as block shear should be considered during design. Moreover, if adequate lateral beam bracing is not provided near the connection, modes such as plate twisting and lateral-torsional buckling of the overall member may govern.

It is noted in the paper that the capacities of unstabilized extended single-plate connections without axial load are substantially larger than those predicted by the design procedure given in the *Steel Construction Manual* [1]. The primary reason for these larger capacities is that the distribution of the moment on the connection plate, which has a large influence on the connection capacity, is determined by the overall stiffness of the structural system, which may or may not reflect the assumptions of the Manual's design procedure. Thus, while this paper presents results of the first phase of an ongoing comprehensive research program on the design and behaviour of extended single-plate connections, subsequent research will include additional full-scale connection tests with different boundary conditions, numerical simulations, and the development of improved recommendations for design.

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