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Experimental Performance of Steel Moment Frame Connections with Replaceable Shear Fuses

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Abstract

Many steel moment frames are designed so that the beams act as structural fuses during earthquake loading. This approach, while effective for life safety, has poor resilience because it is difficult to unlock residual drifts when the beams have yielded or to replace damaged beams in a building.

A repairable steel moment frame connection has been developed that may be more attractive to engineers and constructors. The connection has a fuse plate bolted to the beam bottom flange which experiences shear yielding during severe earthquakes, protecting both the column and the beam. The strength of the fuse plate can be precisely controlled. After an earthquake, the fuse plates can be removed to unlock residual frame deformations and replaced to prepare the building for another event.

Full-scale experiments have been conducted to validate the performance of this innovative connection. Connections have been tested with beams ranging in depth from 350 to 915 mm. Beam and column pairs have completed multiple tests without damage. The paper will discuss the connection in detail and present results from the experimental program. Results for Specimens B3.1 and B3.2 are presented.

Keywords: steel moment frames, shear fuses, experimental testing

1. Introduction

Steel moment frames are commonly used to resist earthquake loads on buildings. Moment frames offer more architectural flexibility than braced frames and, when specially detailed, are considered to have more ductility than other lateral force resisting systems. Most steel moment frames achieve high system-level ductility for earthquakes through controlled yielding in the beams. Figs. 1a and 1b illustrate the type of beam yielding that is expected in ductile moment frames during severe earthquakes. This type of beam yielding should enable buildings to undergo large deformations without collapsing.

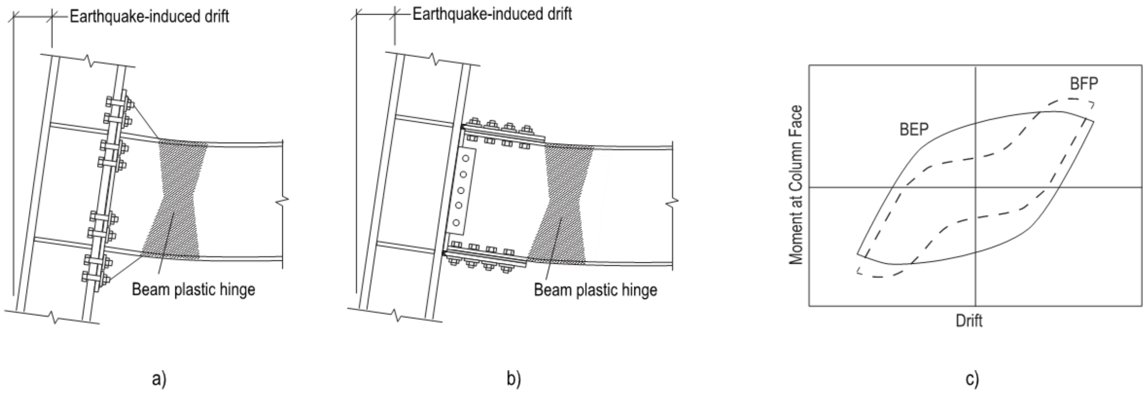


Fig. 1 – . Expected behavior of steel moment frame connections during severe earthquake loading: a) Bolted End Plate (BEP) moment connection; b) Bolted Flange Plate (BFP) moment connection; c) hysteretic behavior.

The design of special moment frame connections for buildings in the U.S. is governed by provisions published by the American Institute of Steel Construction (AISC). AISC 358 [1] lists eight prequalified connections for steel buildings that rely on beam yielding to achieve inelastic rotation capacity. Fig. 1(a) shows a bolted end plate (BEP) connection and Fig. 1(b) shows a bolted flange plate (BFP) connection, with the expected beam plastic hinge regions noted. Beam plastic hinging results in stable hysteretic behavior, but the behavior is different for the different bolted connections [Fig. 1(c)]. Moment connections that experience bolt slip, like the BFP, have pinched hysteretic behavior [2], [3], while field welded connection and those with tension bolts like the BEP have fuller hysteretic behavior. All moment connections that force beam plastic hinging will experience strength loss at large drifts because of flange and web local buckling. For a given beam size, moving the plastic hinge further from the column face will result in greater moment at the column face.

Steel moment frames that rely on beam yielding for ductility may not be very resilient following a severe earthquake because yielded beams are difficult to replace. Even if buildings with yielded beams [Fig. 1 (a), (b)] could be economically repaired, it would require months or years to complete such work.

Alternative moment connections have been developed that provide ductility and energy dissipation without sacrificing the beam. One family of connections rely on friction for energy dissipation. The sliding hinge joint [4] is perhaps the most researched of these friction connections. The resisting force in friction connections can be difficult to predict precisely because it is highly sensitive to the tensioning force and surface conditions of the bearing plates. Friction connections have been discussed in the United States, but not been adopted into practice.

Another family of alternative connections rely on yielding fuses that are replaceable after an earthquake. For example, Balnut and Gioncu [5] suggested moment frames with replaceable elements that are either link-beams with end plates or two channels. Shen et al. [6] investigated these two concepts with four full-scale tests, two for each type of connection. All of the specimens completed cycles beyond 0.04 rad story drift and exhibited stable hysteretic behavior. Castiglioni et al. [7] investigated a similar connection with a composite slab. Another example is the implementation of slit dampers into moment connections. Oh et al. [8] investigated three specimens with split dampers installed at the bottom flange level. The specimens completed cycles at 0.04 rad story drift, but under a protocol that was less demanding than the AISC standard protocol. Some connections have been investigated with yield plates with hour-glass shapes at the top and bottom flange level [1 ,9]. And tee connections which are designed such that yielding occurs in the tees, rather than the beam [10] also fall under the category of connections with replaceable yielding elements.

The DuraFuse Frames connection [11] also relies on a fuse element, but has a fuse that yields primarily in shear, rather than flexure or tension-compression. The DuraFuse Frames connection [Fig. 2(a)] has a steel fuse plate at the beam bottom flange that limits the amount of force that can develop at the face of the column, protecting both the column and beam. The shear yielding mechanism in the fuse is not susceptible to local buckling, so large rotations can be accommodated without strength degradation [Fig. 2(b)]. Richards and Oh [11] described prototype testing of seven reduced-scale specimens with the DuraFuse Frames connection.

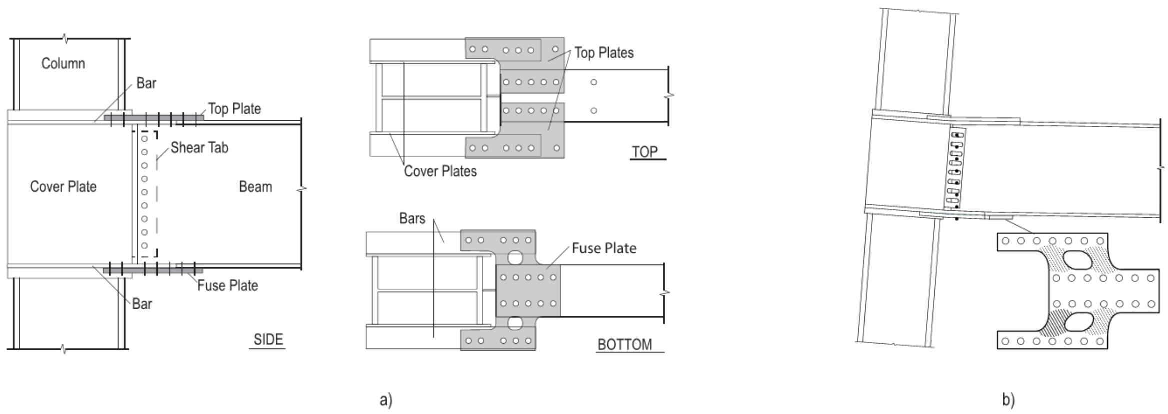


Fig. 2 – DuraFuse Frames connection: a) parts; b) expected fuse plate yielding during severe earthquake loading.

2. Experimental Program

2.1 Set-up

Full-scale sub-assembly testing has been performed to investigate the performance of the DuraFuse Frames connection. Two tests will be described in this paper, but more than twenty full-scale tests have been performed. The test specimens that will be described were designated B3.1 and 3.2. An overview of the set-up is shown in Fig. 3. The specimen was a sub-assembly from a moment frame, tested with the column in a horizontal position and the beam cantilevering up. Throughout the paper, the west flange of the beam is referred to as the “bottom” flange and the east is referred to as the “top”. The specimen had a distance of 4.578m between column supports and a distance of 4.578m from the column centerline to the actuator line-of-action. To simulate inflection points, the ends of the column were mounted on short sections of W360×314 positioned to experience weak-axis bending. A corbel was bolted to the “free” end of the beam and attached to one end of a servo-controlled actuator. The other end of the actuator was mounted to a reaction frame.

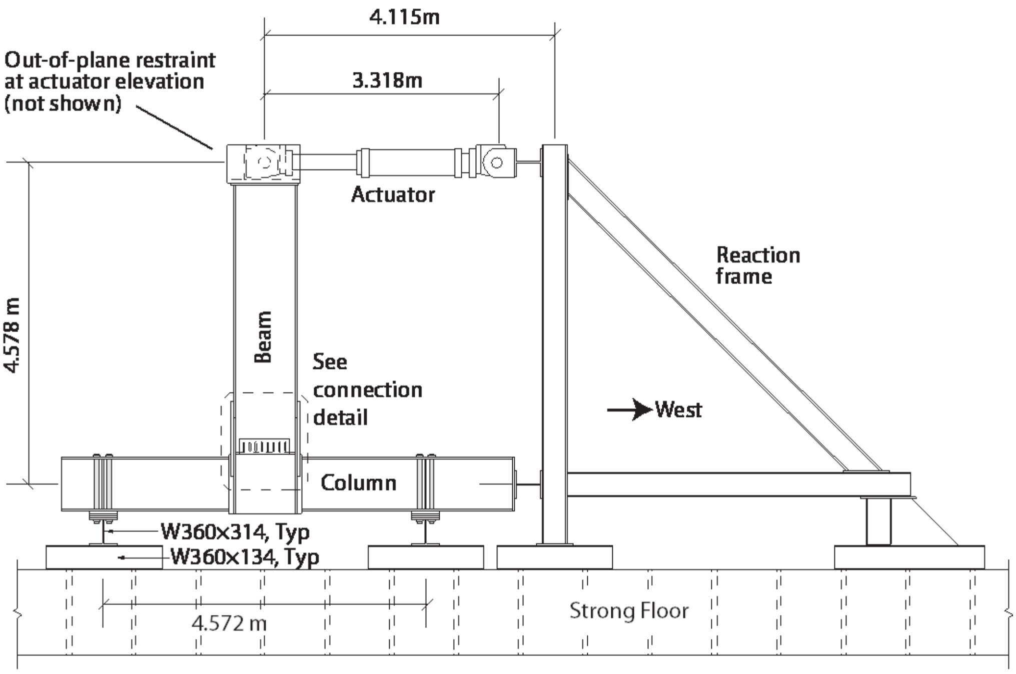


Fig. 3 – Test set-up.

2.2 Specimens

The shop drawings for the connections are shown in Fig. 4. The same beam and column were used for both tests. The beam was W920×223 and the column was a W920×223. The shear tab was 16 mm and the cover plates were 22 mm thick. All of the other bars and plates were 25 mm. The fuse plates had cut-outs that resulted in four "fuse" regions with dimensions of 130×105 mm [Fig. 4 right]. The fuse plates that were used for the two tests were nominally identical.

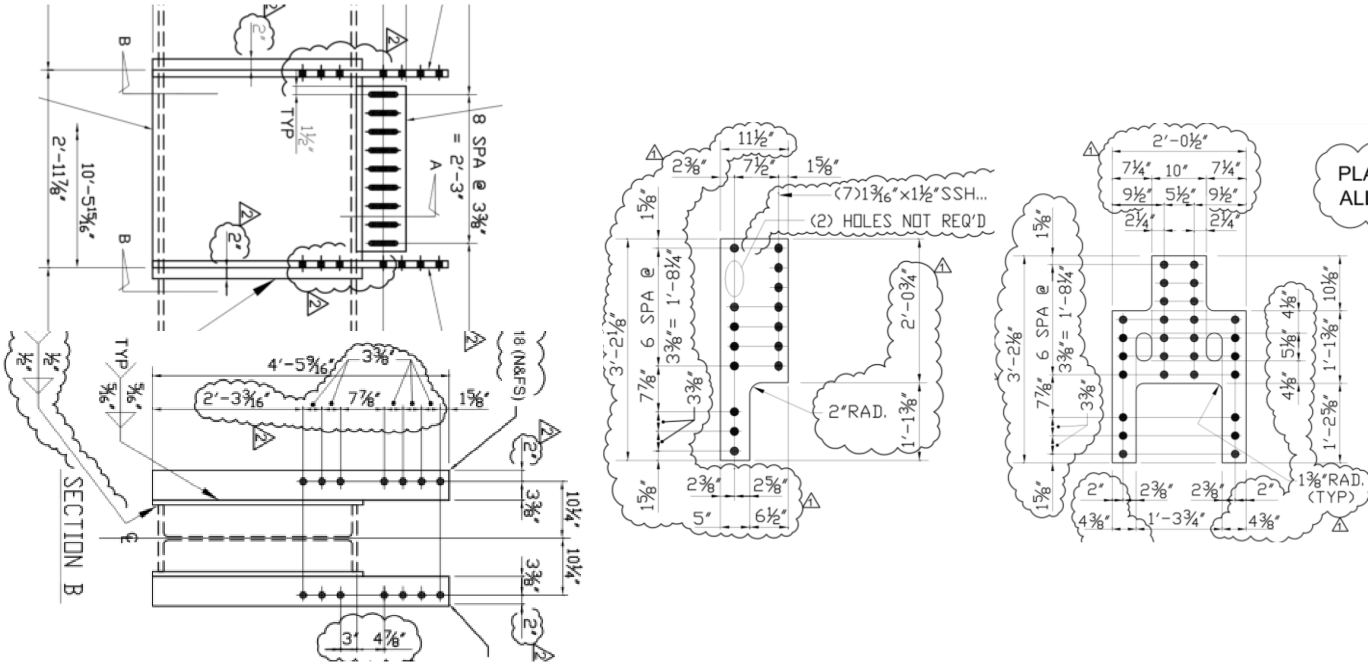


Fig. 4 – Connection detail.

The beam and column were ASTM A992 steel (nominal yield of 345 MPa), while the plates and bars were ASTM A572 Gr. 50 (nominal yield of 345 MPa). Material properties for the fuse plates were determined from independent testing. The properties are summarized in Table 1. The bars and top plates were made from the same heat as the fuse plates. All of the fuse plates were fabricated with a plasma cutter (holes in fuse plates were burned).

Table 1 – Material Properties for Shapes and Plates

Component	Heat No.	F_y (MPa)	F_u (MPa)	F_y/F_u	Elong. (%)
Beam Flange	A142884	403	502	0.80	31
Beam Web	A142884	399	507	0.79	29
Column Flange	480020	402	509	0.79	34
Column Web	480020	430	517	0.83	29
Bars	823Y64350	406	591	0.69	27
Top Plates	823Y64350	406	591	0.69	27
Fuse Plates	823Y64350	406	591	0.69	27

The bolts were 1-1/8 in. dia. (29 mm) ASTM F3125 Gr. F2280 bolts. The bolt holes in the beam and bars were standard sized while the bolt holes in the fuse plates were oversized. The bolt holes in the top plates were oversized on the outside lines, and short-slotted on the inside lines. The web bolts were the same diameter and grade as the flange bolts and were pre-tensioned.

The weld electrode was Lincoln UltraCore 70C, which meets the requirements for demand-critical welds (6). All of the welds were performed in the horizontal position (2F) in the shop.

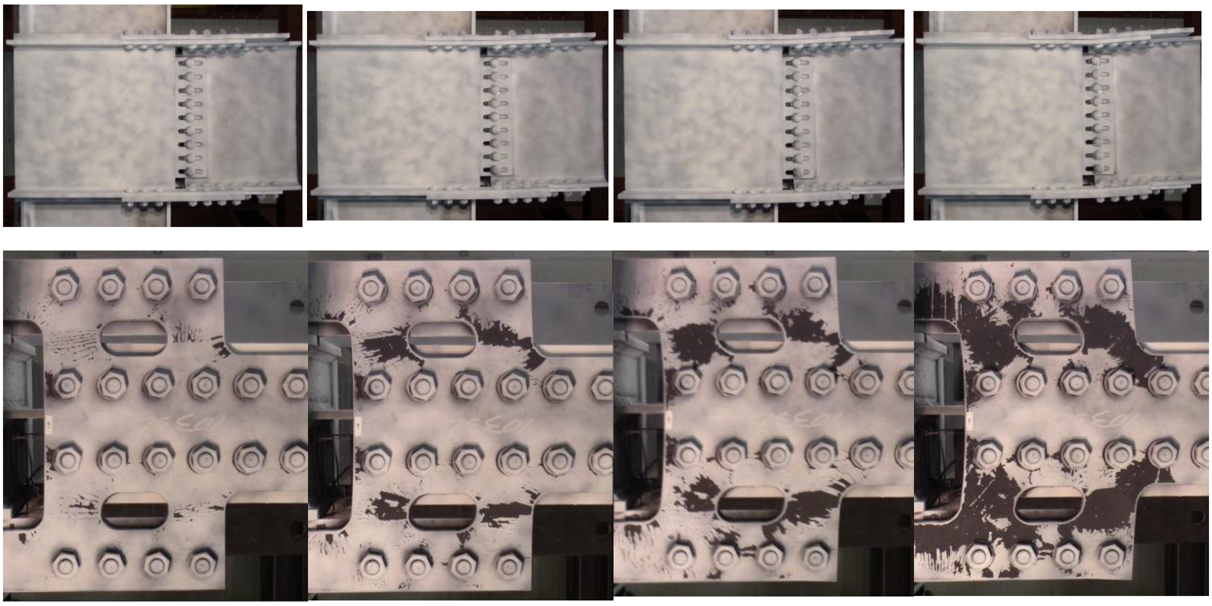
2.3 Loading Protocol

The loading sequence in AISC 341 §K2.4b (AISC, 2016) was used for the tests. The specified loading was 6 cycles at 0.00375 rad story drift, followed by 6 cycles at 0.005 rad, 6 cycles at 0.0075 rad, 4 cycles at 0.01 rad, 2 cycles at 0.015 rad, 2 cycles at 0.02 rad, 2 cycles at 0.03 rad, 2 cycles at 0.04 rad, and 2 additional cycles at each additional 0.01 rad increments up to failure. The displacement corresponding to each specified drift level was calculated by multiplying the target rotation by the distance from the column centerline to the actuator line of action.

3. Results

The observed response was similar for both specimens. Specimen B3.1 will be described in some detail. Specimen B3.1 appeared to respond elastically throughout the 0.00375, 0.005, 0.0075, and 0.01 cycles. During the 0.015 rad cycle, minor yielding occurred in the fuse plate, as evidenced by flaking of the mill scale in the lower yielding regions. During the first 0.02 rad cycle, two bangs indicated bolt slip.

During the 0.02, 0.03, 0.04 rad cycles, yielding of the fuse plate became more pronounced (Fig. 5). During the first cycle to 0.04 rad, the fuse regions appeared to be fully yielded. During the 0.05 rad cycles, yielding spread to locations on the fuse plate outside the fuse regions – below the edge bolts in the middle of the plate and into the arms [Fig. 5(d)]. During the first negative excursion to 0.05 rad minor tearing was observed in the northwest bar-to-cover plate weld. On the second negative excursion to 0.05 rad the tearing progressed and there was noticeable inelastic bending of the cover plate. Similar, but less severe tearing and bending were observed at the northeast bar on the same cover plate. The test was stopped to preserve other components for future tests.

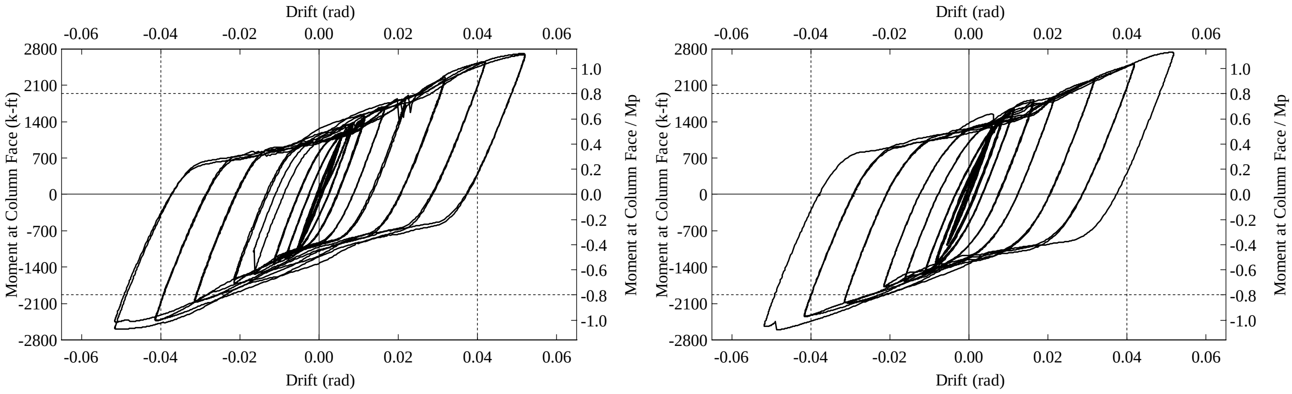


a) b) c) d)

Fig. 5 – Side view and fuse plate view at the first cycle of various deformation levels: a) 0.02 rad; b) 0.03 rad; c) 0.04 rad; d) 0.05 rad.

Specimen B3.2 used the same beam and column as Specimen B3.1. The cover plate was straightened and the bar-to-cover plate welds were reinforced prior to the installation of the new fuse plate for B3.2. Specimen B3.2 re-used the same top plates as were used in B3.1, but a new fuse plate was installed. The new fuse plate was nominally identical to the one used for B3.1. The performance of B3.2 was similar to B3.1. Testing was continued through one cycle at 0.05 rad; when it was stopped to preserve the components for future tests.

The moment at the column face versus the story drift was plotted for each of the specimens (Fig. 6). The moment at the column face was calculated as the actuator load multiplied by the distance from the actuator line of action to the column face (4.311m). The moment at the column face was normalized by the nominal plastic moment of the beam, Mp, on the right side of each plot. Horizontal dashed lines were added at 0.8Mp, which is the strength degradation threshold when determining rotation capacity [12]. The drift was calculated as the displacement at the actuator line of action divided by the distance from the actuator line of action to the centerline of the column (4.578m). Vertical dashed lines were added to the plots at 0.04 rad drift, which is the qualification criteria for special moment frames [12]. Fig. 6 shows that both of the specimens completed several cycles beyond what was required for qualification, and none of the specimens experienced strength degradation. The hysteretic behavior for the DuraFuse Frames connections is similar to that of a bolted flange plate (BFP) connection (Fig. 1c)



a) b)

Fig. 6 – Moment at column face versus story drift: a) Specimen B3.1; b) Specimen B3.2.

4. Discussion

The experiments indicate that the DuraFuse Frames connection can be repaired following a severe earthquake. Specimen B3.1 and Specimen B3.2 had the same column, beam, top plates, shear tab, and bars. The only thing that varied for the two specimens was the fuse plate. After Specimen B3.1 was tested, the fuse plate was replaced and the test was run again as Specimen B3.2. The results in Figure 6 show that Specimen B3.2, the repaired connection, had more-than-sufficient strength and ductility to still qualify as a special moment frame connection.

In addition to improve repairability, the DuraFuse Frames connection has several features that reduce the up-front cost as compared to commonly-used moment frame connections. The first feature is simplified fabrication and welding. Most moment frame connections require complete joint penetration (CJP) welds at the beam end. In the U.S., CJP welds have much stricter inspection requirements than fillet welds. The DuraFuse Frames connection does not have any CJP welds. All of the connection plates are connected to the column assembly with fillet welds.

Another feature that reduces up-front costs is elimination of beam lateral bracing. The AISC Seismic Provisions have a prescriptive requirement for lateral bracing for other moment frame beams because the beams are susceptible to lateral torsional buckling when plastic hinging occurs (Figure 1). The beams in DuraFuse Frames do not have the same problem because the fuse plates prevent beam plastic hinges from forming. The experiments reported in this paper did not have any lateral bracing along the length of the beam. By using the DuraFuse Frames connection, most of the beam lateral bracing can be eliminated.

A final feature that reduces up-front costs is high stiffness efficiency. For many steel moment frames, the member sizes are dictated by stiffness considerations and the strong-column weak-beam criteria. The fuse plates in DuraFuse Frames limit the column flexural demands, making it possible to satisfy the strong-column weak-beam criteria with lighter columns. In addition, the cover plates are efficient in reducing panel zone deformations, and make it possible to achieve required frame stiffness with lighter beams and columns.

5. Summary and Conclusions

Most steel moment frames are designed to experience beam yielding during severe earthquakes. The DuraFuse Frames connection uses a yielding fuse plate to prevent beam yielding. More than twenty full-scale tests have been performed to validate the DuraFuse Frames connection. Two such tests were described in this paper. The test specimens were sub-assemblies from a moment frame with story heights of 4.57m and bay widths of 9.14m. The beam and column size were W920×223 and W920×223 for both tests. The loading protocol from AISC 341-16 was used for the testing.

The test results support the following conclusions:

- The fuse plate in each DuraFuse Frames connection specimen was successful in preventing beam yielding.
- Full-scale DuraFuse Frames connections achieved rotations of 0.05 radians, exceeding AISC requirements for special moment frames.
- DuraFuse Frames connections could be repaired by replacing the fuse plate. The other components of the connection could handle multiple earthquakes.

In addition to improved repairability, the DuraFuse Frames connection reduces up-front costs through simplified fabrication and welding, elimination of most beam bracing, and lower beam and column weights.

6. References

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