SEISMIC BEHAVIOR AND DESIGN OF STEEL SHEAR CONNECTIONS WITH FLOOR SLABS

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SUMMARY

The 1994 Northridge earthquake raised many questions regarding welded steel moment frame structures. The simple connections are typically ignored as far as lateral resistance of these structures, but may have more lateral capacity than traditionally assumed. This project has, as its main goal, the task of determining whether or not these connections can or should be considered for use in lateral load-resisting systems. Cyclic tests on various connection details have been conducted, both with and without the floor slab. A closer look at just a few of the tests done on bolted, single-plate connections shows much about their basic cyclic behavior. This behavior is characterized primarily by slip, yielding, deformation of the bolt holes, and eventually, fracture at large rotations of drift. Tests with and without the floor slab indicated that the contribution of the slab to the lateral resistance was lost by 0.04 radians of drift, but that while the slab was effective, the capacity was practically doubled. Finally, the addition of a supplemental seat angle was very effective in increasing the moment capacity of the connection. Typical moment capacities ranged from 20% of the plastic moment capacity of the beam (Mp) for the bare-steel shear tab connection, to roughly 50% Mp for the shear tab with slab, and 80% Mp for the shear tab with supplemental seat angle and slab.

INTRODUCTION

In the Northridge earthquake, January 17, 1994, many welded, steel moment frame structures sustained brittle fractures in their welded moment connections. There has been a large effort since then to determine both the causes for and solutions to the problems with these connections. Many studies focus on the localized problem of the welded moment connections; still others evaluate the global behavior and design of these welded steel moment frame buildings.

This project attempts to address the question of the contribution of the simple connections to the lateral resistance of moment frame structures. Simple, or shear, connections typically comprise a large percentage of the connections in a welded steel moment frame building. However, these simple connections are traditionally assumed to be pin connections and are ignored as contributors to the lateral load resistance. However, with the composite action of the floor slab, it is possible that these connections actually act as partially-rigid connections, with more lateral load-resisting capacity than previously recognized.

The primary objective of this project is to determine if and when these shear connections, with the contribution of the floor slab, can be used to resist seismic loads. If so, they may provide a cost-effective alternative for repair or retrofit schemes for damaged welded moment-frame buildings. Another objective is to explore the use of the lateral resistance of these composite, partially-rigid connections in new construction. This project will fulfill these objectives through an investigation that includes sixteen full-scale cyclic tests and associated analytical studies. Various connection details were studied in the experimental investigation; this paper presents some of the observations and results related to the cyclic behavior of single plate, or shear tab connections.

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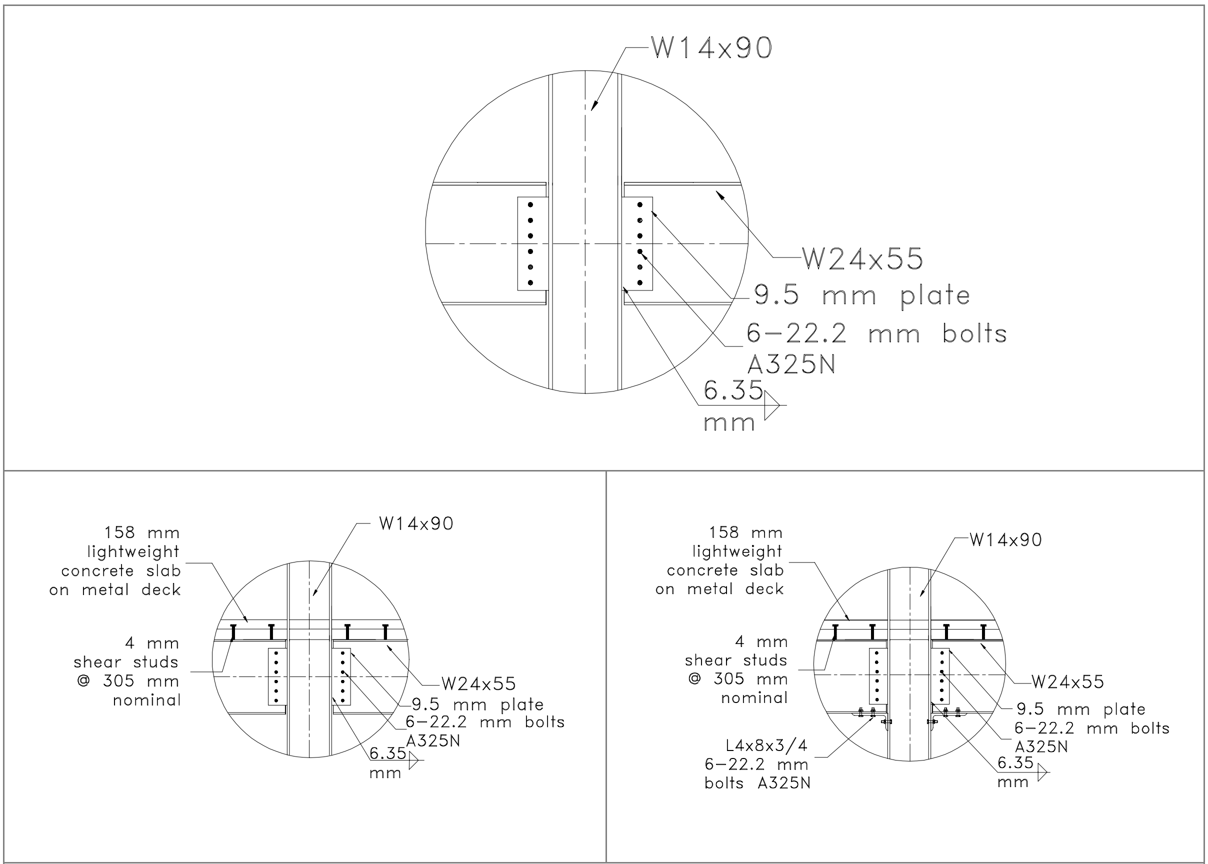
TEST SPECIMENS

The entire test program for this project consisted of sixteen full-scale tests. These sixteen tests were divided into two series. The first series (Series A) included mostly designs based on current practice. The second set (Series B), based on experimental results from the first series, looked towards the improvement of existing details, the effects of different concrete properties, and the performance of some older, standard details. Most of the connections were designed strictly for gravity loads; some others were considered to be partially-rigid connections.

Many of the specimens were based on typical single plate, or shear tab, connections, either in the strong-axis direction of the column or the weak-axis direction of the column. Both the strong-axis shear tab and the weak-axis shear tab were tested with and without the concrete slab, and with lightweight concrete or normal weight concrete. Both were tested with an additional mesh of reinforcement, intended to add negative bending moment capacity and prolong the composite action of the slab. The strong-axis shear tab was also tested with a supplemental seat angle and with a column web cavity void of concrete. An older standard for both strong-axis and weak-axis single plate connections was also investigated. These older connections were both tested without the slab; the strong-axis detail was also tested with the slab. Other specimens included a typical stiffened seat connection, a top and bottom angle connection, and a deeper shear tab connection. This discussion is limited to some of the tests done with a typical, strong-axis shear tab. Details are shown in Figure 1.



Counter-clockwise, from top: Bare-steel strong-axis shear tab specimen (2A), shear tab with floor slab (6A), shear tab with slab and supplemental seat angle (8A).

Figure 1: Details for some of the connections tested

Each specimen was constructed as if it were a section from a prototypical building (Figure 2). This building has W14x90 columns at 7.62 m (25 ft) spacing, and W18x35 beams framing into W24x55 girders. The W-shapes are A572 Grade 50 steel, and the connection plates and angles are typically A36 steel. The floor slab is a 160 mm (6.25 in) lightweight concrete slab on 1 mm (20 gage) metal decking with 74 mm (3 in) ribs at 305 mm (12 in) spacing. The lightweight concrete has a specified compressive strength of 20.7 MPa (3000 psi). Reinforcement for the floor slab is nominal, limited to welded wire mesh and D10 (#3) reinforcing bars at 305 mm (12 in) spacing across the girders for crack control. The beams are partially composite with the floor slab; the use of shear studs is intended mostly for construction or serviceability reasons, but the end result is that the beams and girders are 20 - 30 % partially composite. On the specimen itself, the column is 305 cm (10 ft) from pin to pin, and the beam is 762 cm (25 ft) from pin to pin (Figure 2). Providing the appropriate effective width, the dimension across the slab is 244 cm (8 ft).

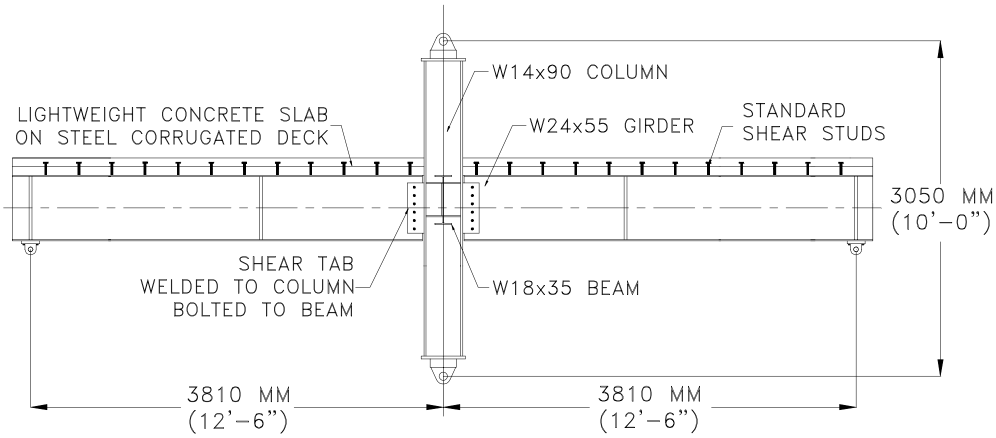


Figure 2: Typical Specimen with Slab

TEST SET-UP

The test set-up, shown in Figure 3, was designed so that both lateral drift displacement and gravity loads could be applied. The boundary conditions were pinned reactions at the top and bottom of the column and at the ends of the beams. The beams were supported vertically by pin-ended struts, which were also instrumented to act as load cells. The boundary conditions were appropriate for the lateral displacement, considering traditional portal frame assumptions of zero bending moment at mid-span of beam and mid-height of column for a frame under lateral loads only. Lateral, or out-of-plane, restraint was provided mainly by the vertical legs of the reaction frame, as well as the bracing mechanism symmetrically located on the opposite side.

The lateral load was applied as a cyclic drift displacement at the top of the column [SAC Joint Venture, 1997]. The drift angle was measured as the displacement at the top of the column divided by the height of the column, pin to pin. This displacement started at very small values of interstory drift and increased gradually until failure of the specimen, or as in some cases, the limit of the testing equipment, typically 0.15 radians. At predetermined points within the loading history, the specimen was unloaded from a zero displacement to a zero load condition, and two service-level cycles of 0.005 radians of drift were applied. The purpose of this procedure was to help determine the stiffness degradation of the specimen over the history.

Previous research on shear tab connections had indicated that the initial shear and rotation on the connection due to the gravity loads would have a significant effect on the response [Astaneh-Asl, 1989]. Although this application of gravity load violated some of the traditional portal frame assumptions used in the design of the test set-up, the effect of this load on the response of the connection was considered to be the critical parameter. Furthermore, it was not practically possible to apply a more realistic distributed load to the beams and girders. Therefore, as a best approximation, two actuators on each beam, each located at 170 cm (5 ft 6 in) from the centerline of the column, were used to create the appropriate shear and rotation at the joint. This load was applied monotonically at the beginning of the test and held constant for the duration. Any theoretical rotation that was still lacking was supplied by a forced deflection at the struts at the ends of the beams.

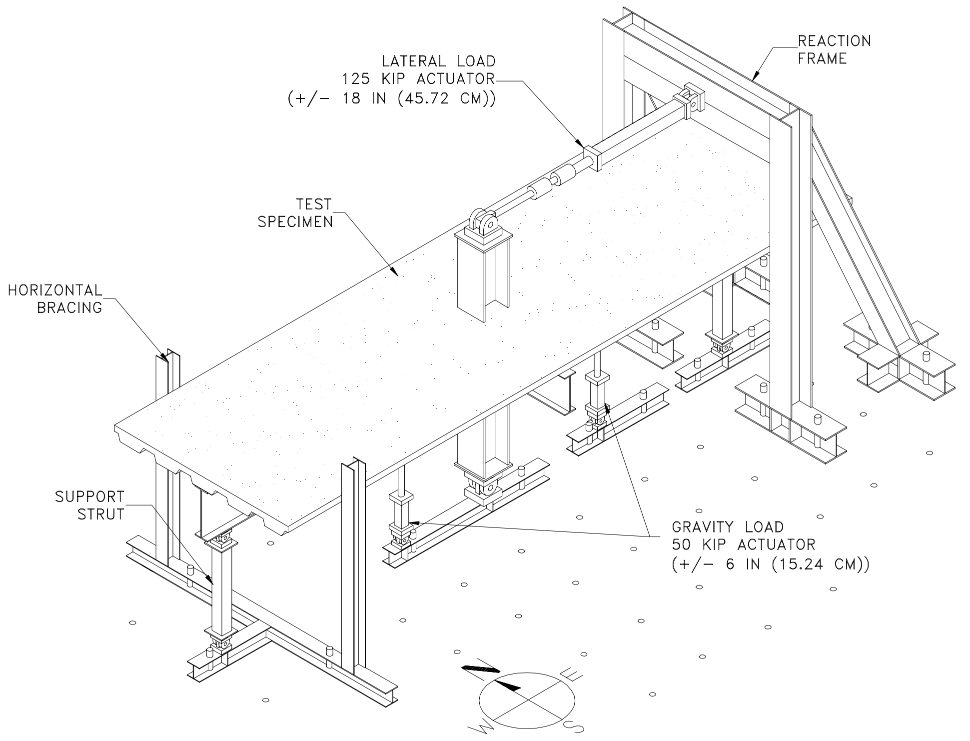


Figure 3: Test Set-up

EXPERIMENTAL OBSERVATIONS

Although this project deals with various types of connection, the main focus was on shear tabs. Over half of the test specimens represented a typical shear tab detail or some variation of it. Presented here are some experimental observations from a typical strong-axis shear tab detail, the same detail tested with the floor slab, and the same specimen with slab and also with a supplemental seat angle. These details were shown in Figure 1.

Cyclic behavior of the shear tab, not considering the floor slab

The test on the bare-steel specimen (2A) gave a baseline for the cyclic behavior of this shear tab connection. Generally, the behavior was characterized by slip and yielding and deformation of the bolt holes, as well as fracture at large levels of drift. Very early in the test (0.005 radians of drift), yield lines began to appear at the top and bottom bolts of the shear tab. Following this, slip between the shear tab and the beam web became quite evident from visible scraping of steel at the top and bottom edges of the shear tab. At 0.02 radians, yield lines had spread throughout the depth of the tab. At 0.03 radians, deformation of the bolt holes was beginning to show, particularly around the top bolts, where the steel in the edge distance of the shear tab was deforming and bulging outward significantly. Some local buckling of the shear tab also became apparent. At 0.07 radians, a crack formed at the top of the east shear tab, near the weld. By this time, there were significant losses in stiffness due to bolt hole deformation and buckling and fracture of the shear tabs. This, however, was counteracted at extreme rotations with a sharp increase in stiffness and load due to contact of the top and bottom beam flanges with the column. Meanwhile, as the crack on the east tab propagated down to the second bolt, the west shear tab followed with a nearly identical crack at 0.08 radians drift. At 0.09 radians, there a sudden, brittle fracture in the bottom edge distance of the west shear tab (Figure 4). The test was ended there.

The maximum capacity of the shear tab was approximately 20% of the plastic moment capacity of the beam (Mp). This capacity, however, came from the bearing of the beam flange on the column. The previous maximum was roughly 15% Mp. Regardless, these values demonstrated that, even without consideration for the slab, these connections have more moment resistance than traditionally assumed in design.

Figure 4: Deformation of bolt hole and fracture, Specimen 2A

The contribution of the floor slab

Tests of the shear tab specimen with floor slab (6A) clearly showed the contribution of the floor slab to the lateral capacity of these connections. Initially, the floor slab added both stiffness and strength, as seen in a comparison of the load versus drift for both the strong-axis shear tab specimen with a lightweight concrete slab and the same specimen without the slab (Figure 5). The maximum capacity was on the order of 50-60% Mp for the specimen with slab.

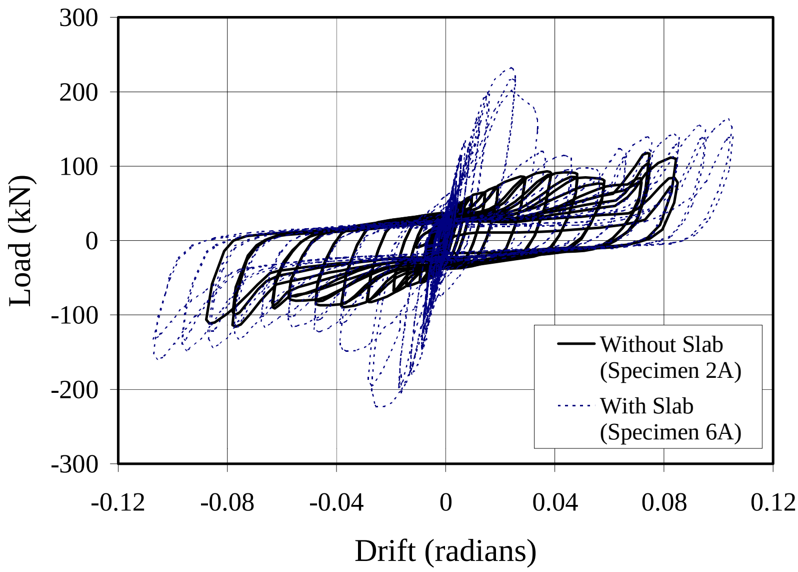


Figure 5: Comparison of load-drift response for specimens with and without the floor slab

In general, the cyclic behavior of the shear tab with the slab was rather similar to that of the bare-steel specimen. The main difference was, of course, in the response of the concrete slab and the shift in the neutral axis, initially forcing more deformation and fracture into the lower portion of the shear tab. For this specimen with the slab, slip and yielding of the shear tab began early in the test, primarily towards the bottom of the tab. By 0.015 radians, deformation of the bolt holes became apparent, as well as some minor local buckling in the tab. Cycles at 0.03 radians saw much more yielding throughout the depth of the beam and out-of-plane deformation. Spalling of the concrete and many small cracks appeared near the column. In the first cycle of 0.04 radians, a crack started to open up just below the bottom bolt on the west tab. This fractured completely through the edge distance on the second cycle. From the drop in lateral load capacity, it was also clear that the composite action and contribution of the slab had been lost. As the test continued, the behavior and capacity of the specimen resembled that of the bare-steel specimen. The existing crack propagated with each new cycle, and a new crack appeared on the east tab. At 0.08 and 0.09 radians of drift, brittle fractures occurred in the top edge distances of each tab. Finally, by 0.11 radians, both crack lines on the east tab had nearly connected and the test was ended. It should be noted that the initial gravity load applied from the beginning was being successfully supported throughout the test.

The same detail with slab was tested with normal weight concrete (6B) rather than lightweight concrete. For this connection detail, the type of concrete did not seem to make much difference in the lateral capacity or behavior. Initially, the appearance of the slab suggested less damage as compared to the lightweight concrete slab and there was increased panel zone yielding. However, the normal weight concrete specimen exhibited a maximum capacity that was also on the order of 50-60% Mp, with only a very slightly higher lateral load. Furthermore, the composite action of the slab was also lost by 0.04 radians drift.

The addition of a supplemental seat angle

In an effort to examine a potential retrofit detail, an identical specimen was also tested with a supplemental seat angle (8A). Not surprisingly, the supplemental angle significantly increased the lateral stiffness and resistance of the connection. The maximum resistance was roughly 80% of the Mp of the beam. The composite action of the slab still ceased to be effective at around 0.04 radians drift. The moment capacity, however, remained at roughly 50% Mp; this was primarily due to the contribution of the seat angle. The comparison of the load versus drift for this specimen and the specimen without the angle is shown in Figure 6.

The cyclic behavior, although similar to that of previous specimens, was certainly affected by the addition of the angle. By 0.0075 radians, there was clearly yielding in the column panel zone. Some plastic hinging in the angle was noticeable by 0.02 radians. By 0.03 radians, there was yielding throughout the depths of the shear tabs, mostly along the bolt lines and in the edge distances. By 0.04 radians, the concrete at the column was being crushed, and the composite action was lost. At 0.05 radians, a fracture occurred in the top edge distance of the west tab and continued down almost to the 3rd bolt. 0.06 radians saw a matching fracture on the east tab. By 0.08 radians, there was a crack extending almost all the way across the east seat angle, on the underside, at the "k" area. This fractured completely through at 0.09 radians drift, marking the end of the test (Figure 7).

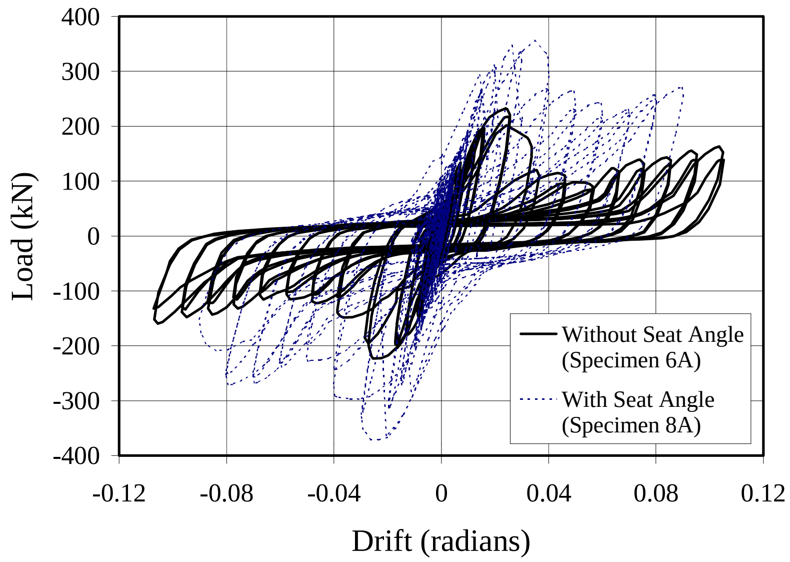


Figure 6: Comparison of load-drift response for specimens with and without supplemental seat angle

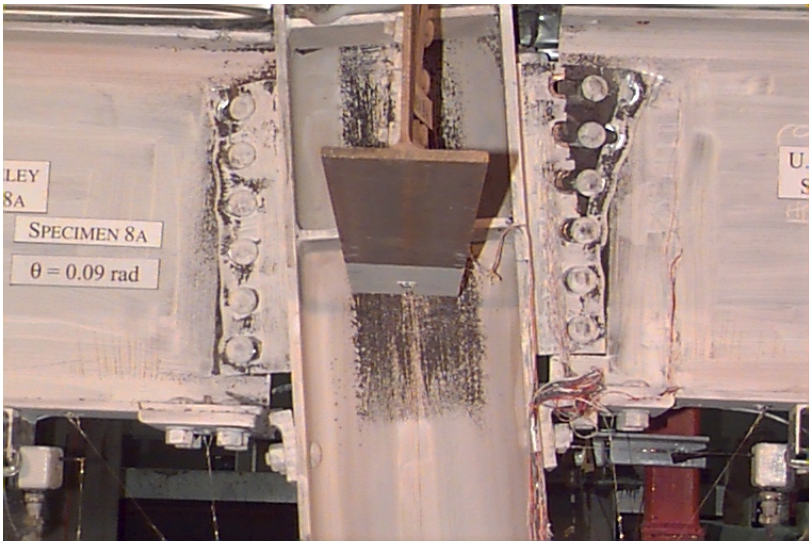


Figure 7: Specimen with supplemental seat angle at the end of the test

SUMMARY AND CONCLUSIONS

The 1994 Northridge earthquake raised many questions regarding welded steel moment frame structures. Meanwhile, the simple connections, although typically ignored as far as lateral resistance, may prove to be an integral part of such systems. This project has, as its main goal, the task of determining whether or not these connections can or should be considered for use in lateral load-resisting systems.

Cyclic tests on various connection details have been conducted. Some details may be considered partially-rigid connections, but the bulk of the tests were concerned with typical, bolted, single plate connections. A closer look at just a few of the tests done on shear tab connections to the strong-axis direction of the column shows much about their basic cyclic behavior. This behavior is characterized primarily by slip, yielding, deformation of the bolt holes, and eventually, fracture at large rotations of drift. Tests with and without the floor slab indicated that the contribution of the slab to the lateral resistance of the connection was lost by 0.04 radians of drift, but up to this point, the capacity was effectively doubled. Finally, the addition of a supplemental seat angle was very effective in increasing the moment capacity of the connection. Typical moment capacities ranged from 20% Mp for the bare-steel shear tab connection, to 50-60% Mp for the shear tab with slab, and 80% Mp for the shear tab with supplemental seat angle and slab.

These simple connections clearly have more lateral resistance than traditionally assumed in design. However, conclusions about how to incorporate them into the lateral design of steel structures deserves further analysis and requires the development of both local and global models of these connections and structures. This experimental work, in conjunction with analytical models, will culminate in a set of design recommendations on the use of simple connections for resisting lateral loads.

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