8th Canadian Conference on Earthquake Engineering / 8ième Conférence canadienne sur le génie paraséismique
Vancouver – 1999

Cyclic Behavior of Steel Shear Connections Including Floor Slab

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ABSTRACT

This project, part of the SAC[*] *Program to Reduce the Earthquake Hazards of Steel Moment Frames, Phase 2,* focuses on the cyclic behavior of simple connections with floor slabs. The problems with welded moment connections in the Northridge Earthquake, 1994, gave incentive to examine the actual contribution of the simple, or shear, connections to the lateral resistance of the building. With the composite action of the floor slab, it is possible that these connections have more lateral resistance than traditionally assumed, and therefore could be used for repair or retrofit of damaged welded steel moment-frame buildings. Furthermore, such connections might be incorporated into lateral load resisting systems for new construction. In order to evaluate these possibilities, two series of full-scale, cyclic tests on shear connections with floor slabs are being conducted. Of these tests, the first series has been completed, and the second series is underway. Together with analytical studies, models, and an extensive literature review, these tests will yield valuable information regarding the use of simple connections in lateral load resisting systems of steel moment-frame buildings.

INTRODUCTION

In the Northridge Earthquake, 1994, over 200 welded steel moment-frame buildings experienced brittle fractures in their welded moment connections. Since this event, there has been a significant effort to study many aspects of welded steel moment-frame structures, including the contribution of the existing simple connections to the seismic resistance. Although they typically comprise a large percentage of the beam-to-column connections in these structures, simple connections are traditionally ignored as far as the lateral resistance. However, with the effect of the floor slab, these connections may have more lateral resistance than traditionally assumed.

The primary objective of this project is to determine if and when the shear connections, with composite action of the slab, can be used to resist seismic loads. If so, they may be considered for use in cost-effective repair or retrofit schemes for welded steel moment-frame buildings. A secondary objective looks towards the use of such composite semi-rigid connections in new construction. For this purpose, two series of eight tests were planned. The first series of tests, based on current design practices, has been completed. The second series was designed based upon the results from the first. This discussion will focus on experimental observations from the first series.

TEST SPECIMENS AND TEST SET-UP

For the first series of tests, the connections were mainly typical, single plate shear tabs (AISC, 1994), (SAC, 1996). Also included were a stiffened seat connection and a supplemental seat angle connection. Half of the specimens were designed to test the cyclic behavior in the direction of the strong axis of the column and the girders (W24x55); the other half was designed to test the cyclic behavior in the direction of the weak axis of the column and the beams (W18x35). For a clearer assessment of the contribution of the floor slab, the first two tests were of shear tab connections without the concrete slab. One specimen in the weak axis direction of the column was tested to investigate the effect of having more steel reinforcement in the slab in the area around the column. Another, loaded in the direction of the strong axis of the column, addressed the consequences of not having concrete within the column web cavity. Details for the connections can be seen in Figure 1.

Each test specimen was built as if it were from a building with W14x90 columns at 7.62 m (25 ft) spacing, W24x55 girders and W18x35 beams. The specimens were full scale and represented a portion of a structure extending from mid-story-height to mid-story-height of column, and from mid-span to mid-span of beams. Each test specimen measured 7.6

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[*] SAC Joint Venture is a partnership of Structural Engineers Association of California (SEAOC), Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe)

m (25 ft) long, 3.0 m (10 ft) high, and 2.5 m (8 ft) wide with the floor slab. The floor slabs were made of lightweight concrete, 159 mm (6-1/4 in) deep on 20 gage (0.04 in) metal decking with 76 mm (3 in) ribs. Typically, the specimens with slabs included welded wire mesh for temperature and shrinkage reinforcement and nominal reinforcing bars across the girders for crack control under gravity loads. The beams were partially composite, with roughly 30% composite action. Figure 2 shows a typical elevation of a specimen with a floor slab.

The test set-up was designed for both gravity loading and a cyclic application of drift (Figure 3). Cyclic loading was based upon increasing drift displacements and was conducted following provisions of the SAC testing protocol (1997). The gravity loading was based on previous research (Astaneh-Asl et. al, 1989), which showed the initial shear and rotation due to gravity loads to be two of the most important factors affecting the behavior of shear tab connections. Therefore, prior to cyclic loading, gravity loads were applied and held constant for the duration of the test. Two hydraulic actuators, one on each beam, were used to load the specimen such that the initial shear and rotation at the joint best approximated that present in the theoretical building. Any additional rotation required was supplied by a forced vertical deflection at the support struts at the ends of the beams.

EXPERIMENTAL RESULTS

As stated, the first series of tests has just recently been completed; some preliminary results from each specimen are presented here. Envelopes of the load-drift response to the west (Figure 3) are shown in Figure 4.

- Specimen 1A demonstrated the cyclic behavior of a typical 4-bolt shear tab, without slab, in the weak-axis direction of the column. This specimen was very ductile, reaching large drift rotations of 0.14 radians before a complete fracture of the shear tab. The behavior was marked primarily by slip and yielding of the shear tab and significant deformation or elongation of some of the bolt holes. Ductile fracture began at the top of the shear tab at 0.11 radians of drift. The maximum moment resistance was roughly 15% of the plastic moment capacity of the beam (Mp).

- Specimen 2A demonstrated the cyclic behavior of a typical 6-bolt shear tab in the strong-axis direction of the column, without the slab. This tab was also very ductile up to large rotation. Behavior was characterized by slip, yielding, and deformation of bolt holes, followed by ductile fracture at 0.07 radians. The maximum moment resistance was approximately 20%Mp, and the test was stopped at a drift rotation of 0.09 radians.

- Specimen 3A showed the behavior of the 4-bolt shear tab, including the effect of the floor slab. Noticeably stiffer than the specimen without the slab, this connection also exhibited some slip and yielding before crushing of the concrete slab around the column and damage to the metal decking caused a loss of the composite action after 0.04 radians of drift. The behavior then resembled that of the specimen without slab (1A), although at a slightly higher moment capacity, with significant bolt hole deformation and then ductile fracture of the shear tab (0.12 radians). The maximum moment resistance was approximately 30%Mp.

- Specimen 4A was tested to investigate the potential benefits of additional reinforcement in the slab around the column for the 4-bolt shear tab. The behavior of this specimen was similar to that of Specimen 3A; in one direction of loading, loss of composite action still occurred at around 0.04 radians of drift due to the crushing of the concrete immediately around the column, and particularly at the flange tips. In the other direction of loading, however, the specimen was noticeably more brittle; the composite action was lost much earlier (Figure 4). Still, the noticeable benefit of additional reinforcement was a decrease in the extent of damage to the concrete around the column.

- Specimen 5A was a stiffened seat connection. The yielding in this connection occurred primarily in the beam flanges. Fracture of the erection angle at the top of one beam occurred at 0.05 radians; this was followed by fracture of the two bolts connecting the bottom flange of the beam to the stiffened seat on the opposite side at 0.06 radians. The maximum moment capacity was roughly 50% Mp.

- Specimen 6A demonstrated the cyclic behavior of the 6-bolt, strong-axis shear tab with the slab. Again, the slab roughly doubled the strength of the connection until loss of composite action due to crushing of the concrete, at 0.03 to 0.04 radians of drift. After this point, the strength dropped to something slightly higher than that of the specimen without slab. A sudden fracture of the bottom edge distance of one shear tab also occurred at 0.04 radians, initiating a crack that would extend almost the depth of the shear tab by 0.11 radians. The maximum moment resistance was roughly 60%Mp.

- Specimen 7A was nearly identical to Specimen 6A, but for the fact that there was no concrete in the column web. The cyclic behavior of 7A was similar to that of 6A, but with less strength and seemingly higher ductility. Fracture of the shear tab did not begin until 0.07 radians, but loss of composite action still commenced at around 0.04 radians, and by 0.11 radians, the shear tab was almost completely fractured through.

- Specimen 8A demonstrated the benefit of the addition of a supplemental seat angle to a structure that otherwise resembled Specimen 6A. This specimen was the strongest connection, and unlike any of the other test specimens, there was panel zone yielding, which began early in the test. There was also little slip or yielding in the tabs or beams or seat angles before plastic hinging in the angles, which was evident by 0.03 radians. Again, the contribution of the slab was essentially lost by 0.04 radians. This was followed by fracture in the shear tabs, beginning at 0.05 radians, and eventually, fracture of one of the seat angles at 0.09 radians, marking the end of the test. The maximum moment capacity of this connection was estimated at 80% Mp; the capacity after loss of composite action was roughly 50%Mp.

SUMMARY AND CONCLUSIONS

The specimens without slabs demonstrated that these theoretically pinned, simple, connections do have some moment capacity on their own. The 4-bolt shear tab on the W18x35 beam (1A) developed, on average, 15% of the plastic moment capacity of the beam. The 6-bolt shear tab for the W24x55 girder (2A) developed 20%Mp on average. The connections also showed very ductile behavior up to large rotations of 0.09 radians, characterized primarily by slip and yielding.

In both cases (W24x55 and W18x35 specimens), the addition of the floor slab to the test specimen resulted in roughly twice the maximum lateral load resistance. At approximately 0.04 radians drift, however, the composite action was basically lost due to damage to the concrete and the decking, and the specimen with the floor slab would revert towards the behavior of the specimen without the slab, with slightly higher capacities. Meanwhile, the addition of a seat angle proved to be effective in increasing the overall moment capacity of the connection, although still with a loss of the composite action by 0.04 radians of drift.

While the first test series has yielded much valuable information regarding the cyclic behavior of the shear tab connections, it has also raised some more questions. Among these questions are inquiries into the behavior of such connections with normal-weight concrete, the behavior when subjected to a near-fault loading history, and the behavior of shear tabs designed to an older standard and other connection details. These questions will be addressed by the second test series, currently in progress. The end result, including analysis and an extensive literature review, will be a comprehensive look at the use of these so-called simple connections for lateral resistance in steel welded moment-frame structures.

ACKNOWLEDGMENTS

The impetus and means for this project are credited to the sponsor, SAC Joint Venture. Meanwhile, much valuable technical assistance has been provided by the SAC technical advisory panel on connections (in particular, the input by Jim Malley, Professor Steve Mahin, Professor Charles Roeder, and Ron Hamburger is sincerely appreciated), the American Institute of Steel Construction (AISC), and the Herrick Corporation. Verco Manufacturing donated the metal decking, and additional assistance was given by Garcias Metal Specialties for installation of the deck. John Wolfe of Steven Tipping and Associates provided some typical details for the shear tabs and seat connections.

REFERENCES

American Institute of Steel Construction 1994. AISC Manual of Steel Construction Load & Resistance Factor Design.

Astaneh-Asl, A., Call., S.M., and McMullin, K.M. 1989. Design of Single Plate Connections. AISC Engineering Journal, First Quarter 1989. 21-31.

Liu, J. and Astaneh-Asl, A. 1998. Behavior of Simple Connections in Frames Subject to Large Lateral Deflections. Proceedings, Structural Engineers World Congress (SEWC)1998.

Liu, J. and Astaneh-Asl, A. 1999. Studies and Tests of Simple Connections, Including Slab Effects. Report No. UCB/CEE-Steel-99/01, Department of Civil and Environmental Engineering, University of California at Berkeley. (in progress)

SAC Joint Venture 1996. Brief Summary of Anticipated SAC Phase 2 Experimental Investigations, Request for Proposals. SAC Joint Venture, Sacramento, CA.

SAC Joint Venture 1997. Protocol for Fabrication, Inspection, Testing and Documentation of Beam-Column Connection Tests and Other Experimental Specimens. SAC/BD-97-02, SAC Joint Venture, Sacramento, CA.

FIGURES

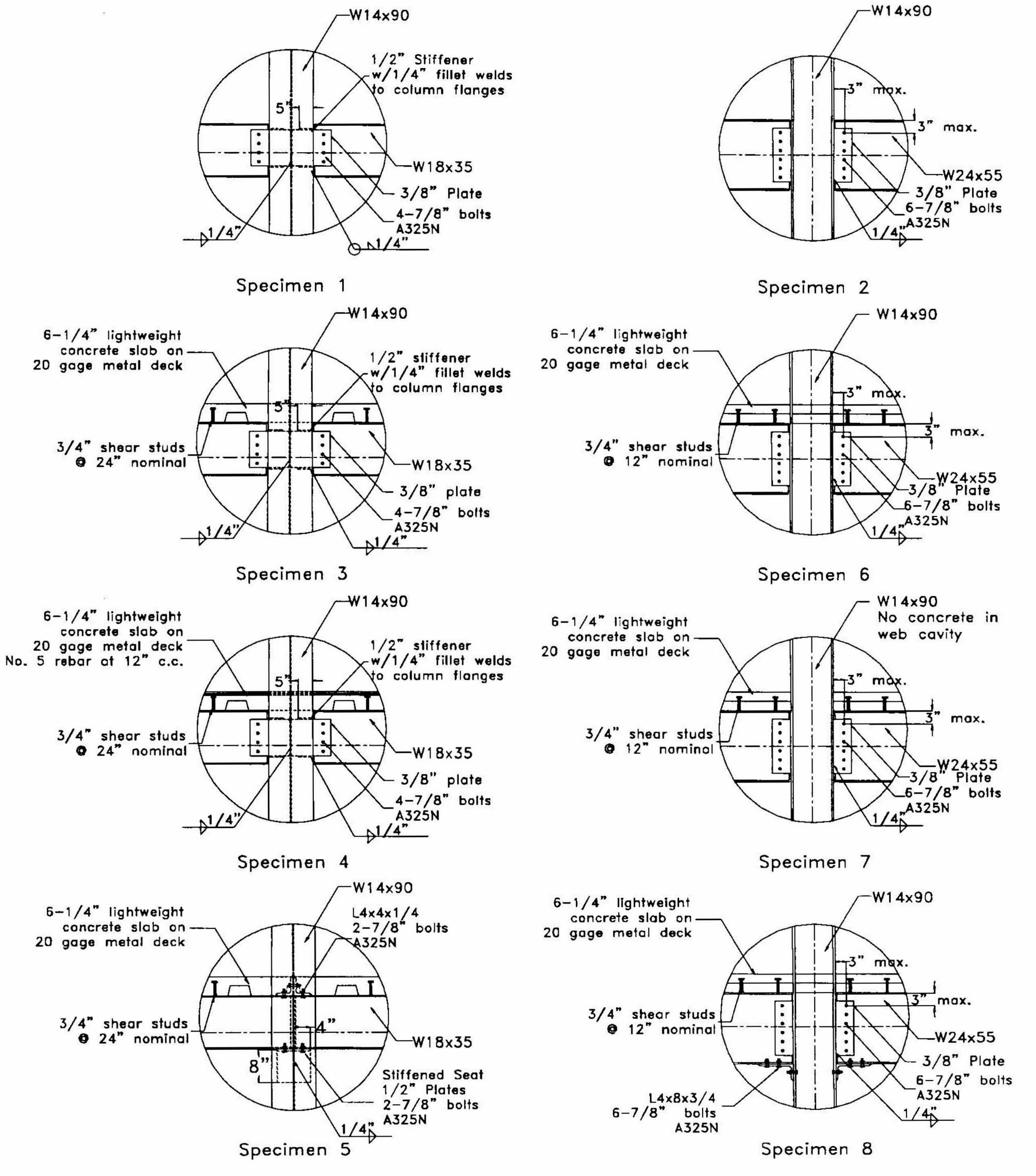

Figure 1: Connection details of test specimens

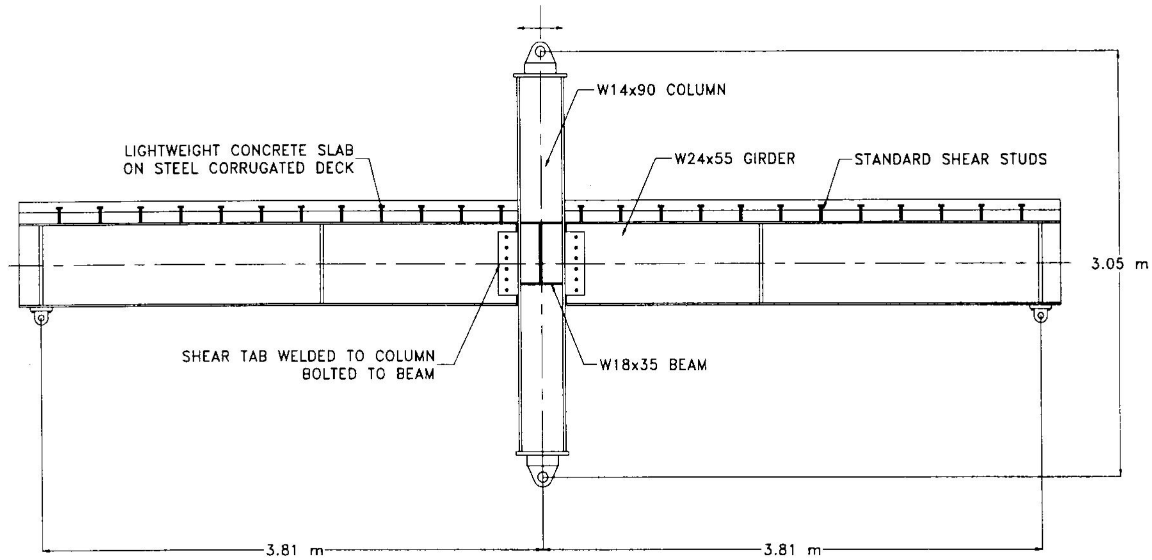

Figure 2: Elevation of a typical test specimen

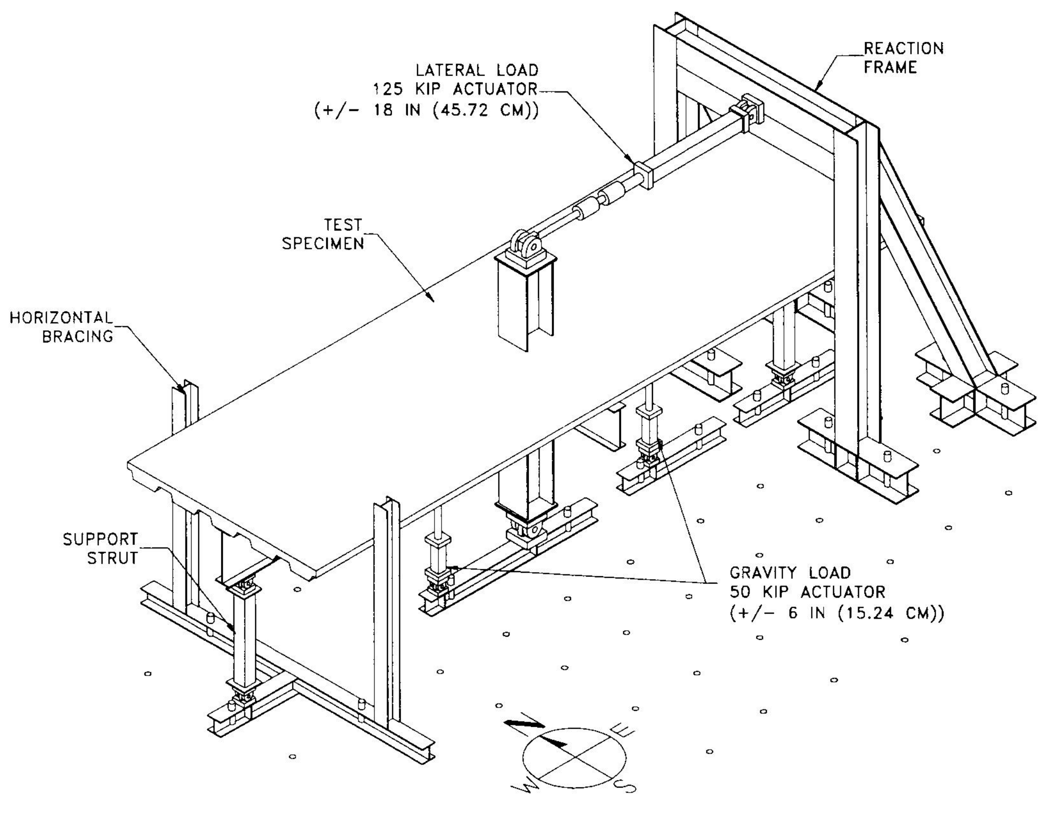

Figure 3: Isometric drawing of test set-up

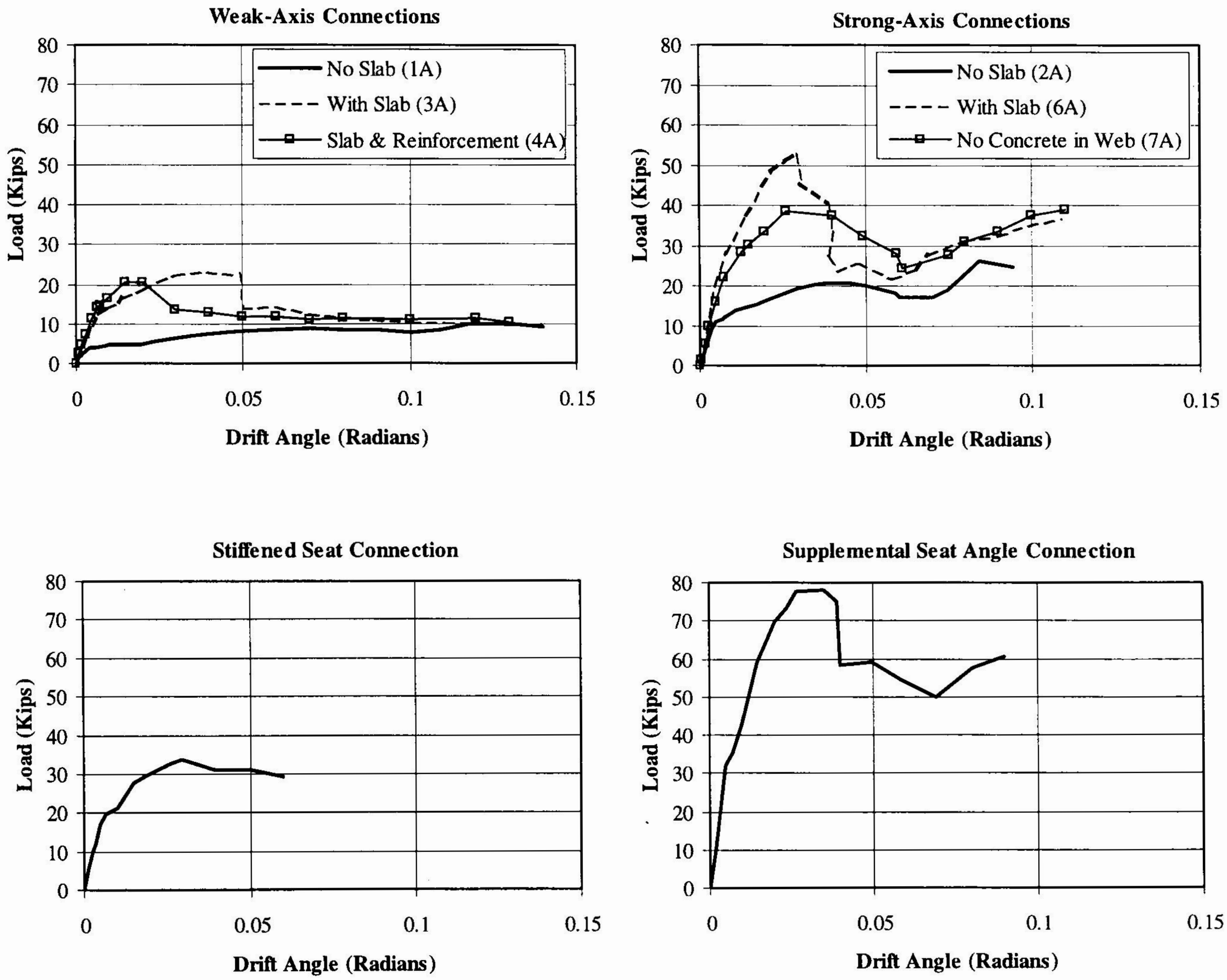

Figure 4: Envelopes of Load-Drift Response