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# EFFECTIVENESS OF STEEL FIBERS VERSUS SHEAR STUD REINFORCEMENT FOR PUNCHING SHEAR RESISTANCE IN SLAB-COLUMN CONNECTIONS SUBJECTED TO BI-AXIAL LATERAL DISPLACEMENTS

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**ABSTRACT :**

A study of the effectiveness of steel fiber reinforcement and headed shear studs for increasing punching shear resistance and deformation capacity of slab-column connections subjected to earthquake-induced deformations was conducted. Three large-scale slab-column subassemblies were tested under combined gravity load and bi-axial lateral displacements. The slab-column connection in the first two specimens was reinforced with hooked steel fibers in a 1.5% volume fraction, while the connection in the third specimen was reinforced with headed shear studs. All three connections were subjected to a gravity shear ratio of approximately ½ during application of lateral displacements. Test results indicate that the use of fiber reinforced concrete in the connection region is an effective design alternative to increase punching shear resistance and deformation capacity of slab-column connections. Maximum connection rotation, just before punching, was approximately 0.04 rad in the two fiber reinforced concrete connections. On the other hand, a peak rotation of 0.027 rad was measured in the specimen with shear stud reinforcement. Inspection of the connection after the test indicated a breakout failure of the concrete engaged by the second line of studs accompanied by severe bending of the bottom steel rail.

**KEYWORDS:** Fiber reinforced concrete, drift capacity, flat-plate construction, shear strength, hooked steel fibers, headed reinforcement

## 1. INTRODUCTION

Reinforced concrete frames that consist of slabs directly supported by columns are a popular structural system because of their low cost and reduced story heights. In high seismic risk regions, slab-column frames are often combined with either structural walls or moment resisting frames for lateral stiffness and strength. However, the slab-column frames must be able to maintain their gravity load carrying capacity while undergoing earthquake-induced lateral displacements.

Research conducted in the 1980s and 1990s (e.g. Pan and Moehle, 1992; Hueste and Wight, 1997) showed that drift capacity in slab-column frames decreases as connection shear due to gravity loads increases. To account for this reduction in drift capacity, the ACI Building Code (ACI Committee 318, 2008) includes a drift versus gravity shear ratio interaction diagram (Fig. 1) in order to determine whether shear reinforcement is required in slab-column connections. Gravity shear ratio is defined as the shear induced by gravity load divided by the punching shear strength of the connection. Alternatively, a stress-based check can be performed to determine the need for shear reinforcement. In this check, shear reinforcement is needed if the summation of the shear stress due to direct punching shear and the shear stress due to unbalanced moment at the design lateral displacement exceeds the nominal punching shear strength of the connection (typically at a stress of $0.33\sqrt{f_c'}$, MPa).

Although several types of shear reinforcement have been evaluated for use in slab-column connections (e.g. bent-up bars, shearheads, hoops), headed shear stud reinforcement (Fig. 2) is currently the most widely used slab shear reinforcement in the United States. Shear stud reinforcement has been reported to be as effective as hoop reinforcement in resisting punching shear stresses while being easier to install (Robertson et al., 2002). This type of reinforcement, however, does have some drawbacks, particularly its elevated cost. Further, it may cause interference problems and reinforcement congestion, as shown in Fig. 2. As a result, the use of fiber reinforcement was investigated as an alternative to shear stud reinforcement in slab-column connections.

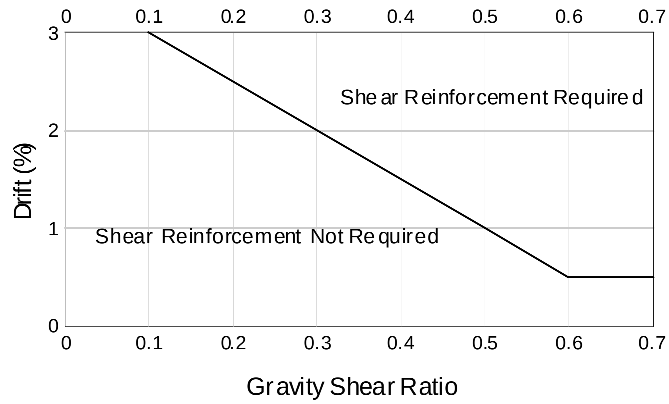



Figure 1 Drift versus Gravity Shear Ratio Relationship in ACI Building Code (ACI Committee 318, 2008)

Figure 2 Shear Stud Reinforcement in Slab-Column Connection (Courtesy of Eduardo Miranda)

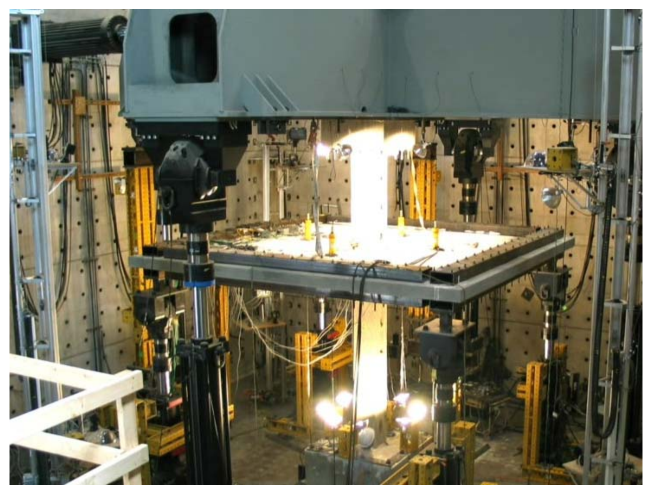


Figure 3 Slab-Column Connection Subassembly

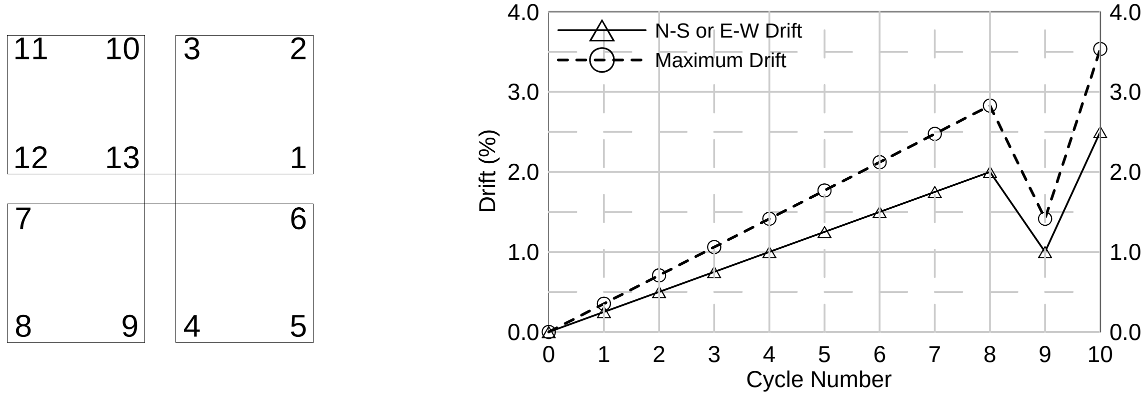
## 2. EXPERIMENTAL PROGRAM

The seismic behavior of slab-column connections with either fiber reinforcement or headed shear studs was evaluated through testing of three large-scale slab-column subassemblies under combined gravity load and bi-axial lateral displacements. The tests were conducted at the University of Minnesota NEES (Network for Earthquake Engineering Simulation) Laboratory, referred to as the Multi-Axial Subassemblage Testing (MAST) Laboratory. Each specimen represented a first floor interior connection. Thus, the subassemblies featured a full-story column underneath the slab, fixed at its base, and a half-story column above the slab, where lateral displacements were applied. Fig. 3 shows a photo of a test specimen.

The slab in each specimen was 5.2 m square, 15 cm thick, while the column was 40 cm square. The first story height was 3.25 m while the height of the column above the slab, including a block used for loading purposes, was approximately 1.9 m. Four vertical actuators were used to restrain vertical displacement at the slab corners while allowing lateral displacements and rotations. Steel double tube elements, “sandwiching” the slab along its perimeter, were used to restrain vertical displacements on the slab periphery.

A major challenge in tests of slab-column connections is the application of gravity loads, especially when moderate to high gravity shear ratios are to be achieved. In this research, gravity load in addition to the slab self-weight was applied through four steel strands, tensioned by hydraulic jacks, located 95 cm from the column faces. These hydraulic jacks, placed on top of the slab, can be seen in Fig. 3. The force in the strands was selected to induce a gravity shear ratio of ½. Because of load redistribution due to cracking and yielding in the slab, this force was adjusted throughout the test in order to keep the gravity shear ratio as close to the target as possible.

Lateral displacements were applied at the top of the column through a “rigid” steel crosshead. Each displacement cycle consisted of 13 steps in plan view, as shown in Fig. 4*a*. Displacement cycles of increasing magnitude were applied following a pre-determined pattern until punching shear failure of the slab-column connection occurred. A plot of peak drift versus cycle number is shown in Fig. 4*b*. Two drift values are shown per cycle, one corresponding to loading in each principal direction (N-S or E-W), and the other to the maximum resultant drift combining both loading directions. A small column axial load, corresponding to approximately 10% of the column axial load capacity, was applied throughout the test.




a) Bi-axial Displacement History per Cycle
b) Peak Drift History

Figure 4 Loading History

### *2.1 Slab Reinforcement*

The three slab-column subassemblies were designed to be nominally “identical”, except for the connection region. Fig. 5 shows the top and bottom flexural reinforcement layout for the slabs of Specimens 1 and 2. The flexural reinforcement layout in Specimen 3 was nearly the same as that in the other two specimens, except for some minor differences in bar spacing in the connection region in order to accommodate the headed shear stud reinforcement.

Specimens 1 and 2 featured fiber reinforcement in the connection, up to four slab thicknesses away from the column faces. Peak shear stresses at the interface between the fiber reinforced concrete and the regular concrete were expected to be below $0.17\sqrt{f_c'}$ (MPa). Two types of hooked steel fibers were evaluated, both at a design volume fraction of 1.5%. Due to a larger than expected volume of concrete in the ready-mix concrete truck, however, the fiber volume fraction used in Specimen 2 was estimated as 1.36%. Specimen 1 was reinforced with 30 mm long, 0.55 mm diameter (aspect ratio of 55) hooked fibers, made of a wire with a 1100 MPa tensile strength. On the other hand, 30 mm long, 0.38 mm diameter (aspect ratio of 80) hooked fibers, made of a wire with 2300 MPa tensile strength, were used in the connection of Specimen 2. The two fiber types were manufactured by Bekaert Corp. and are referred to as Dramix ZP305 and Dramix RC80/30-BP fibers, respectively. Fiber reinforced concrete was cast first, immediately followed by casting of regular concrete in the slab regions away from the connection. Thus, no cold joint existed at the interface between the fiber reinforced concrete and the regular concrete.

The connection of Specimen 3 featured headed shear studs, as shown in Fig. 6. The design of the shear studs was performed according to the 2008 ACI Building Code (ACI Committee 318, 2008) such that,

$$v_n = v_c + v_s \geq v_u \tag{2.1}$$

where $v_c$ is the assumed contribution of the concrete to punching shear strength ($v_c = 0.25\sqrt{f_c'}$ (MPa) for the configuration tested), $v_s$ is the shear strength provided by the headed shear studs, and $v_u$ is the maximum shear stress due to a combination of direct punching shear and shear induced by unbalanced moment, calculated as follows,

$$v = \frac{V}{b_o d} + \frac{\gamma_v M}{J_c} c \tag{2.2}$$

where $V$ is the connection shear, $b_o$ is the critical section perimeter, $\gamma_v M$ is the portion of the unbalanced moment $M$ transferred through eccentric shear ($\gamma_v$ = 0.4 in this research study), $J_c$ is a critical section property that is "analogous to polar moment of inertia", and $c$ is the distance from the centroid of the critical section to the section on the critical perimeter where the shear stress is calculated.

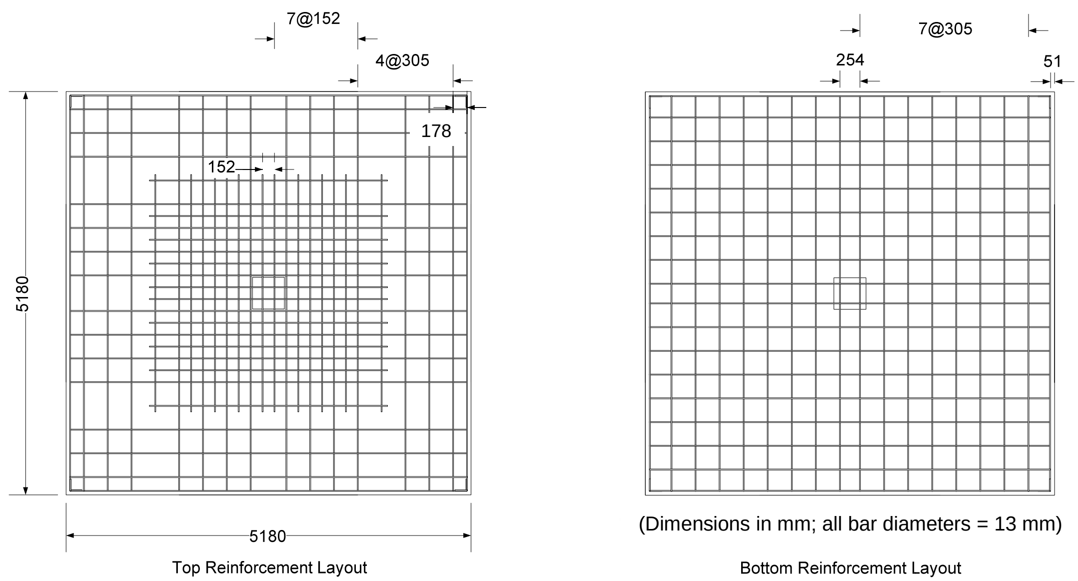



Figure 5 Slab Flexural Reinforcement Layout

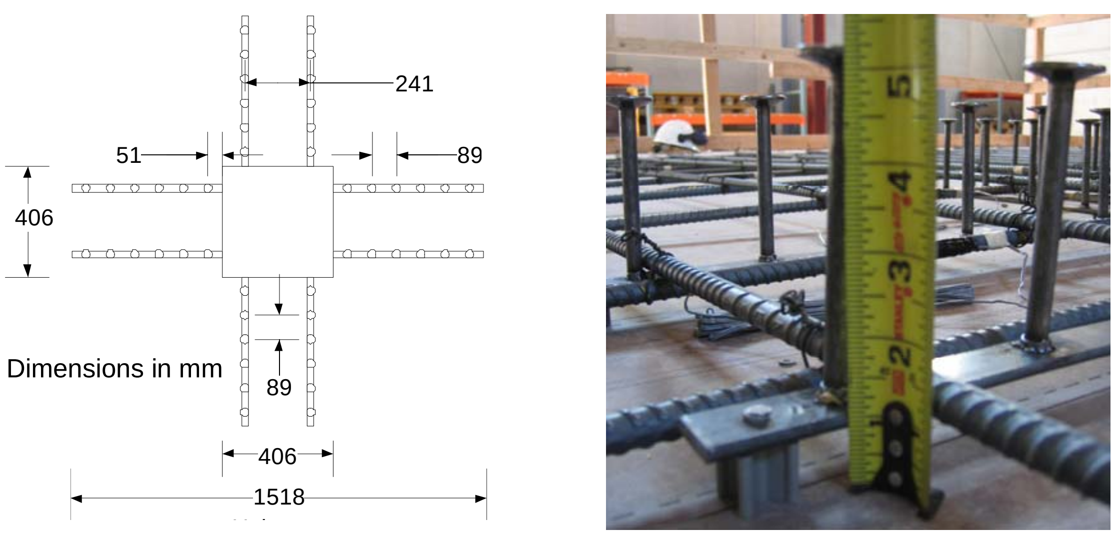



Figure 6 Headed Shear Stud Arrangement in Connection of Specimen 3

### *2.2 Material properties*

Ready-mix concrete was used in all three specimens. For the case of fiber reinforced concrete, hooked steel fibers were added to the truck at the site. Average compressive strengths of the fiber reinforced concrete (Specimens 1 and 2) and regular concrete (Specimen 3) used in the connection region, obtained from cylinder (100x200 mm) tests, were 37 MPa, 31 MPa, and 44 MPa, respectively. Third-point flexural tests of fiber reinforced concrete beams (150 mm square cross section and span length of 450 mm) were also conducted. Test results showed a deflection-hardening behavior for both types of fiber reinforced concrete. The material with high-strength hooked fibers, however, showed higher ductility compared to that of the concrete reinforced with regular strength hooked fibers.

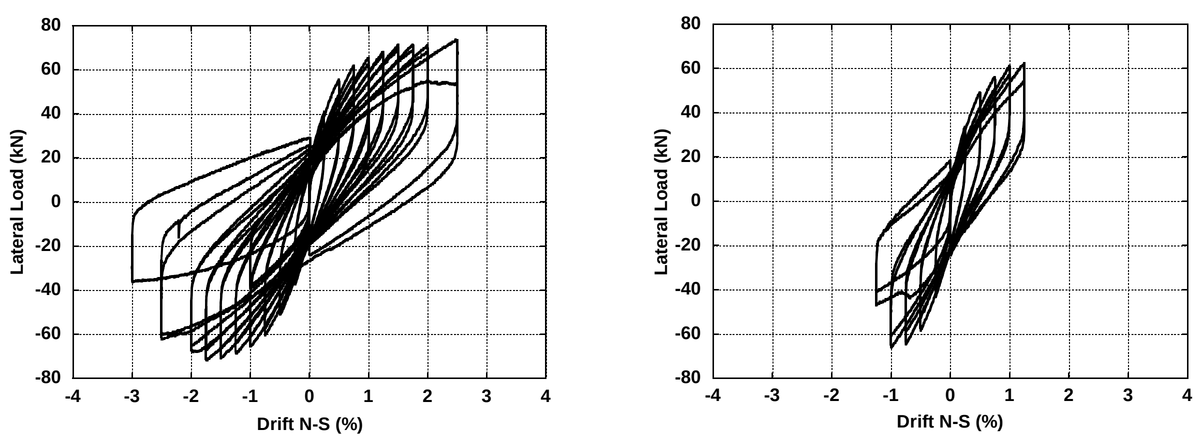
Grade 420M reinforcing bars were used for column and slab reinforcement. Measured yield and ultimate strengths for the slab flexural reinforcement were 424 MPa and 659 MPa for Specimens 1 and 2, and 451 MPa and 730 MPa for Specimen 3, respectively.

## 3. EXPERIMENTAL RESULTS

### *3.1 Overall Behavior*

The lateral load versus drift hysteresis responses for loading in the North-South direction for Specimens 1 and 3 are shown in Figs. 7*a* and 7*b*, respectively. Drift was calculated as the lateral displacement at the top of the column divided by the specimen height. The behavior of the test specimens, prior to punching shear failure of the connection, was governed by the flexural behavior of the slab. In all three specimens, flexural yielding of the slab was first observed during the cycle at 0.75% drift. During subsequent cycles, yielding of the slab spread over a width of approximately 80 cm (five slab thicknesses) for Specimens 1 and 3. For Specimen 2, slab yielding spread over a width of 60 cm (four slab thicknesses). Yielding at the column base first occurred during the cycle at 1.25% drift. By the end of the tests, the column of Specimens 1 and 2 had undergone moderate inelastic rotations (peak total rotation on the order of 0.015 rad). Column base yielding in Specimen 3, on the other hand, was very limited because punching shear failure occurred soon after yielding had initiated during the cycle at 1.25% drift.

Failure in the specimens with fiber reinforcement (Specimens 1 and 2) occurred during the cycle at 2.5% drift in each principal direction. It is worth mentioning that punching shear failure in Specimen 2 actually started during the cycle at 2% drift. However, a sudden drop in applied gravity load occurred early in the cycle at 2.5% drift. Punching shear failure in the specimen with shear stud reinforcement, on the other hand, occurred during the cycle at 1.25% drift in each principal direction. Before the development of a punching shear failure, damage in the connection region of all three specimens was relatively minor.



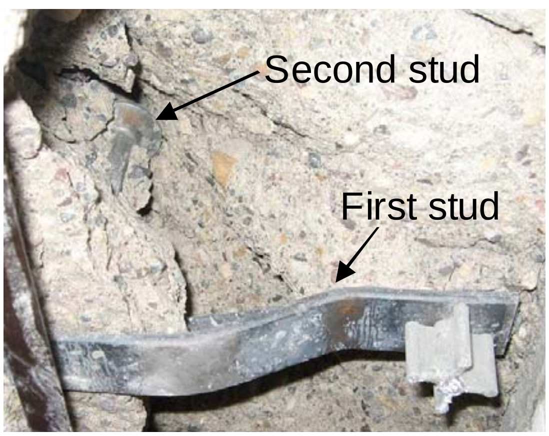

a) Specimen 1 (regular strength fibers) b) Specimen 3 (shear stud reinforcement)

Figure 7 Load versus Drift Hysteresis Response

The size of the punching shear cone was relatively similar, regardless of whether fiber reinforcement or headed shear studs were used. On average, the failure surface on the top of the slab was located at 1.5 slab thicknesses away from the column faces. Figs. 8*a* and 8*b* show the punching shear surface for Specimens 1 and 3, respectively. The punching failure mechanism in Specimens 1 and 2 was fiber pullout. On the other hand, failure in Specimen 3, with shear stud reinforcement, seemed to have been initiated with a breakout failure of the concrete engaged by the second line of studs (Fig. 8*c*). Once the studs were not able to bridge the critical diagonal crack, the bottom rails supporting the studs were mobilized in shear through dowel action, which led to severe bending of the rails, as shown in Figs. 8*c* and 8*d*.

a) Specimen 1

b) Specimen 3




c) Failure Surface and Shear Stud Reinforcement Damage in Connection of Specimen 3 with Regular Concrete

d) Bending of Rail and Slab Drop in Specimen 3 (After Removal of Damaged Concrete)

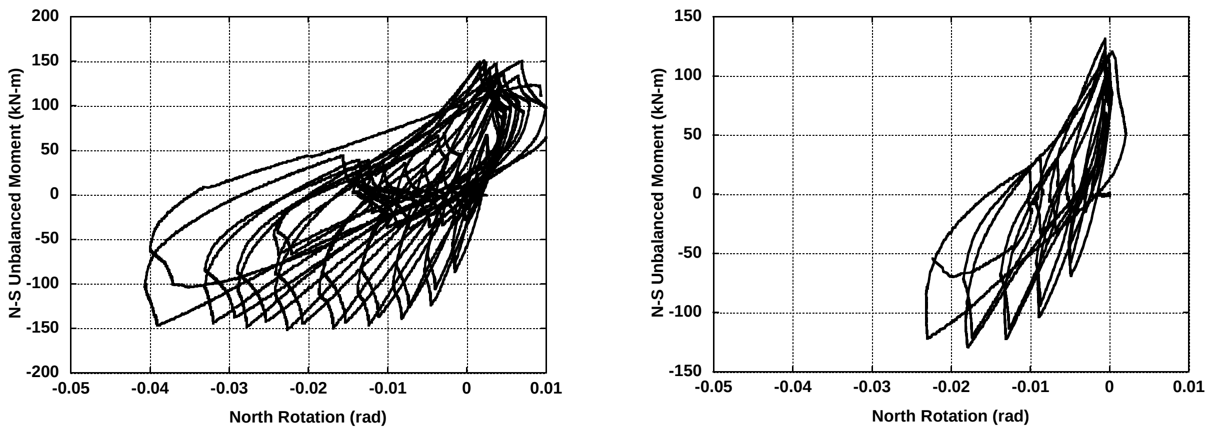
Figure 8 Punching Shear Failure in Test Specimens

### *3.2 Slab Rotations*

In typical slab-column subassembly tests representing a connection in an intermediate floor level, slab rotations and drift are generally comparable. In this research, however, in which a first-story connection subassembly with the column base fixed was tested, specimen drift was expected to be lower than slab rotations in the vicinity of the column. Slab rotations were measured over a distance equal to 1*d* and 2*d* from each column face, where *d* is the effective depth of the slab. Most of the slab inelastic deformations concentrated within one effective depth from the column faces. Figs. 9*a* and 9*b* show the unbalanced moment versus rotation response for Specimens 1 and 3, respectively. Unbalanced moment corresponds to loading in the North-South direction, while the rotations shown were measured over a distance of 1*d* on the North side of the connection.

As can be seen in Fig. 9*a*, Specimen 1 exhibited a stable moment versus slab rotation hysteresis response. A rotation slightly less than 0.01 rad corresponded to first yielding of the slab which, as reported earlier, occurred during the cycle at 0.75% drift. Maximum unbalanced moment corresponded to a rotation, on average, of approximately 0.015 rad. The peak unbalanced moment during subsequent loading cycles remained relatively constant up to a slab rotation of approximately 0.04 rad, when a punching shear failure initiated. The behavior of Specimen 2 was similar to that of Specimen 1, with rotations at punching shear failure between 0.036 and 0.044 rad for all four connections sides.

The behavior of Specimen 3 with headed shear studs was characterized by a limited connection rotation capacity, as shown in Fig. 9*b*. Rotation at failure on the North side of the connection was 0.023 rad. The rotation at failure on the other three sides of the connection ranged between 0.02 rad and 0.027 rad.




a) Specimen 1 (regular strength fibers) b) Specimen 3 (shear stud reinforcement)

Figure 9 Connection Unbalanced Moment versus Rotation Response

### *3.3. Peak Shear Stress*

Shear stresses due to combined direct punching shear and unbalanced moment were calculated using Eqn. 2.2. From this equation, which is used in typical design practice in the United States, unbalanced moments about two principal axes are considered separately. In Specimens 1 and 2, the peak combined shear stress for loading in each principal direction was approximately equal to $0.4\sqrt{f_c'}$ (MPa). For Specimen 3, with shear stud reinforcement, the peak shear stress was approximately $0.33\sqrt{f_c'}$ (MPa) and $0.36\sqrt{f_c'}$ (MPa) for loading in the North-South and East-West direction, respectively. If Eqn. 2.2 is modified to account for bi-axial bending rather than bending in one direction only, a peak shear stress of $0.54\sqrt{f_c'}$ (MPa) and $0.59\sqrt{f_c'}$ (MPa) is obtained for Specimens 1 and 2, respectively. For Specimen 3, the peak shear stress caused by gravity-induced shear and bi-axial bending was $0.47\sqrt{f_c'}$ (MPa).

As mentioned earlier, a shear stress of $0.33\sqrt{f_c'}$ (MPa) represents the maximum value for which shear reinforcement is not required in connections with the configuration tested. Thus, the limited drift capacity exhibited by Specimen 3, which was subjected to a peak shear stress only 10% greater than the shear stress limit for connections with no shear reinforcement, is concerning and suggests that the headed shear stud reinforcement was not effective in increasing the deformation capacity of the connection.

## 4. CONCLUSIONS

From the information presented, the following conclusions can be drawn,

- Hooked steel fibers in a 1.5% volume fraction were effective as shear reinforcement in slab-column connections subjected to combined gravity load and bi-axial lateral displacement reversals. No major difference in connection rotation capacity was observed between the connection with regular strength hooked fibers and that with high-strength hooked fibers. In both cases, average connection rotation prior to punching shear failure was approximately 0.04 rad. Compared to the connection with headed shear stud reinforcement, the fiber reinforced concrete specimens exhibited over 60% greater average rotation capacity. All three specimens were subjected to a gravity shear ratio of ½.

- Headed shear stud reinforcement did not seem to be effective in bridging the critical diagonal crack and thus, in increasing connection punching shear resistance and deformation capacity. Although peak shear stress due to direct punching shear and eccentric shear was only slightly greater than the upper shear stress limit for which no shear reinforcement is required according to the 2008 ACI Building Code ($0.33\sqrt{f_c'}$, MPa), punching shear failure occurred at a peak connection rotation of 0.027 rad. Failure was characterized by a breakout failure of the concrete engaged by the second line of studs accompanied by severe bending of the rail supporting the studs. Based on these results, further experimental research on the behavior of slab-column connections with headed shear stud reinforcement under combined gravity load and lateral displacement reversals is warranted.

## ACKNOWLEDGEMENTS

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