Connection Specimens

All specimens were modeled after a typical 10 storey flat plate office or residential building having a typical center-to-center distance between columns of 5 m and 4 m in the two principal directions. A typical size of the column was 1200 mm by 240 mm and the overall depth of the slab was 200 mm. Inter-story height was 3 m. In addition to its dead weight, the slab was also designed for 1.0 kPa of super-imposed dead load and a 3 kPa of service live load. The panel centerlines or approximate lines of contraflexure during application of lateral loadings were taken as the boundaries of the slab specimens.

The actual test specimens would represent a 75% scale down of the portion of the floor plan to be modeled (Fig. 1). This would work out to the specimens having dimensions of 4.5 m by 3.5 m and column size of 0.9 m by 0.18 m and a slab thickness of 150 mm. The overall height of the specimen was 2.25 m and was assumed to terminate at the column mid-height in an actual prototype building. This was because the column mid height represented the lines of contraflexure in the column during lateral loadings. In addition, the size of each specimen was determined such that the stresses near the connections would be relatively unaffected by the boundary conditions near the slab edge. Since the panel centerlines in an actual building would only move horizontally under lateral loading (with very little relative deflections), the slab boundaries were assumed to be on rollers. A schematic diagram showing the boundary and support conditions for the single column test specimens are shown in Fig 2.

The roller supports were simulated by means of edge link supports with steel rocker simulating the pin support (Figs. 3a). Figure 3b shows the overall test set-up.

Details of the top and bottom reinforcement layouts in the slabs are shown in Fig 4a and 4b, respectively. The compressive strength of concrete was 40 MPa and the yielding strength for both flexural and shear reinforcements was 520 MPa for 10 mm bars (T10) and 530 MPa for the 13 mm bars (T13). The slab was reinforced with top flexural reinforcement ratio of 1.2% in regions of c+3h across the column width where c is the width of column transverse to the direction of the lateral loading and h is the overall depth of the slab. The c+3h width was used as specified in ACI 318-02 to prevent flexural failure due to transfer of unbalanced moments by flexural stresses.[1, 2] For better performance, the top bars in the y-direction (along the strong column axis) were placed at the outermost layer (Fig 4) because of the longer span in the y-direction. Clear concrete cover was 15 mm; therefore, the average effective depth was 122 mm. Bottom reinforcement ratio of 0.4% was provided as bottom flexural reinforcement. In addition, at least two bottom bars passed through the column continuously to prevent total collapse of the specimen during punching shear failure[2]. The column was reinforced with steel ratio of 2.5%, aligning with the "strong column and weak beam" concept in design.

Fig. 5 shows the arrangement of the SSR around the column. The shear studs used have an overall height of 150 mm equal to the overall depth of the slab and diameter of stem equal to 10 mm. Lack of concrete cover for the studs was not expected to result in any drastic loss in strength.[4,5] The arrangement and spacing of the studs around the column followed the recommendations of ACI Committee 421[3] and was designed such that the nominal shear strength of the connection would not exceed 0.5 $(f'c)^{1/2}$ MPa. The spacing and layout of the SSR were determined so that punching shear failure would not occur outside the shear-reinforced region. The spacing chosen between the face of the column and the first stud of 45 mm was less than the upper limit of 0.4d. Subsequent spacing of 90 mm was approximately equal to 0.75d where d is the average effective depth of the slab. The distance between the stud shear reinforcement on the long faces of the column exceeded 2d. This is because the critical zone for shear failure is around the short faces of the column and adding more SSR along the long faces did not necessarily lead to higher capacity of the connection.

Gravity Loading

The high gravity loading case corresponded to the slab being loaded with a live load of 4 kPa in addition to the 1 kPa super-imposed dead load and self-weight of the slab. The low gravity loading case corresponded to the slab being loaded with a 0.75 kPa of live load in addition to the imposed dead load and self-weight of the slab.

Using the above set of assumptions, the gravity shear ratio, V_g/V_o for high gravity loading cases worked out to be 0.28 while that for low gravity shear would be 0.17. The term V_g is the shear force transferred at the slab column connection due to gravity loads and is calculated using tributary area, while V_o is the punching shear strength of the connection in the absence of moment transfer. V_o was calculated and governed by the minimum value given by equations (1a), (1b), and (1c) as determined according to ACI 318-02. [1]

$$V_o = (1+\frac{2}{\beta_c})\frac{\sqrt{f_c'}b_o d}{6} \tag{1a}$$

$$V_o = (2+\frac{\alpha_s d}{b_o})\frac{\sqrt{f_c'}b_o d}{12} \tag{1b}$$

$$V_o = \frac{1}{3}\sqrt{f_c'}b_o d \tag{1c}$$

β_c is the ratio of the long side to the short side of the column; b_o is the perimeter of critical section located at 0.5d from face of column; d is the average effective depth of the slab; and α_s is 40 for interior connections, 30 for edge connections and 20 for corner connections

Gravity loading was simulated by means of the self-weight of the slab, placement of steel blocks each weighing 100 N on the slab and vertical force from a 200 tonne hydraulic jack at the bottom of the column. The placement of the steel blocks and application of the vertical force were done such that the shear stresses at the critical section 0.5d from the column face were equivalent to the desired gravity shear.

Lateral Loading

The lateral loading sequence follows that shown in Fig 6a. Lateral loading procedures were displacement based and the drift ratio parameter was used. Drift ratio is defined as the relative displacement between the top and bottom of the column divided by the column height. Two cycles of loadings were performed at each target drift ratio. After 1.5% target drift ratio was reached, an “unloading” to 1% drift ratio for one cycle was performed following each subsequent target drift ratio. The purpose was to study the behavior of the connection at service level after loading at higher drift ratios.

For specimens subjected to biaxial bending, the loading path for each cycle as shown in Fig 6b was followed. The entire loading path for one cycle consists of 16 loading stages, designed to simulate all possible loadings history. For the uniaxially loaded specimens, the lateral loading was applied in the weaker column direction (along x-axis) of the connection.

Instrumentation and Testing

Load cells were installed to measure the values of the applied loads (Fig. 3a). In addition, a total of 72 strain gauges were attached to both the flexural and shear reinforcements to record the strains readings in the reinforcements during all stages of loading. Line transducers were attached to the edges of the specimen and column to record the lateral movements. A total of 8 LVDT and 4 line transducers would serve to measure the vertical deflections of the slab.

Gravity loading from self-weight of the slab was simulated first during the experiment by first jacking up the specimen so that the rocker at bottom of column would carry the whole of the self-weight. Gravity loading due to the imposed dead load was then simulated by placing the steel blocks at the designated positions before the vertical force was applied at the bottom of the column to simulate the effects of the desired live loads. The vertical force was then maintained throughout the course of the experiment. Prestressing of the column using prestressing steel bars running through the column also simulated the uniform axial force in the column from the upper stories of the building. During lateral testing, the test was stopped at the end of every loading stage as shown in Fig 6b so that the development and propagation of cracks could be observed. The slab was deemed to have failed when the lateral loads dropped to less than 80% of the peak load.

GENERAL SPECIMEN RESPONSE

For specimens loaded with high gravity shear, flexural cracks on the top surface of the slab were observed at the slab to column interface after the application of the gravity loads, while there was no visible crack due to low gravity shear. Throughout the course of the experiment, more flexural cracks were formed and propagated towards the slab edges as the load increased. Cracks in the tangential direction crossing the cracks in the radial direction were also observed during the course of experiment. Existing cracks from gravity loads if any would widen and propagate as well. Flexural cracks at the bottom surface of the slab did not form until drift ratio of 2.5%. Very few new cracks were formed on top surface, if any, after the target drift ratio of 2% although the existing cracks continue to widen throughout the course of the experiment.

Specimen YL-L1 (low gravity shear, uniaxial lateral loading) failed in flexure mode and its overall hysteresis curve is shown in Fig 7. Crushing of the concrete started to occur at target drift ratio of 5% while the column started to crack at 6% drift ratio. From the hysteresis curve, it can be seen that the peak measured lateral force P_x (a long the weaker column direction or x axis) of 63kN was recorded at drift ratio of 3.97% and in the opposite direction (-x axis); -71.2kN at -5.57% drift ratio. The gentle decreasing slope after the peak values indicate a ductile behaviour while the severe pinching of the hysteresis curve indicates bond deterioration at the connection. The test was stopped at target drift ratio of 7% when the maximum stroke in the hydraulic jacks was reached. The crushing of the concrete coupled with the deep flexural cracks at column to slab interface and the results from hysteresis curve indicated that flexural failure had occurred. The cracking of the column also shows that inelastic deformation of the column had occurred.

Specimens YL-H2 (high gravity, biaxial lateral loading), YL-L2 (low gravity, biaxial) and YL-H2V (low gravity, biaxial, with SSR) failed in punching shear characterized by the sudden drop in load (See Figs. 8 to 10) and the spalling of the top concrete, exposing the top flexural bars. For all the biaxially loaded specimens, failure always occur after the application of the lateral loadings in both directions i.e after loading stage 2, 6, 10 or 14 of the loading path shown in Fig 6b. For specimen YL-H2 (Figs. 8a and 8b), the peak P_y (lateral force along the strong column direction or y axis) occurred at 1.45% drift ratio and had value of 51.6kN and, in the opposite direction, the peak P_y was -48kN at -1.4% drift ratio. On the other hand, P_x had its peak value of 43kN at the drift ratio of 1.96% just after loading stage 1 and -52.2 kN at -1.88% drift ratio just after loading stage 9 respectively. Punching shear occurred after loading stage 2 at target drift ratio of 2%. By the end of cycle 1 for target drift ratio of 2%, the applied lateral loads in both the x and y directions have dropped by more than half

Specimen YL-L2 (low gravity, biaxial lateral loading) also failed in punching shear at target drift ratio of 2%. Peak values P_x and P_y could be observed during first cycle of 2% drift ratio. See Figs 9a and 9b for the hysteresis curves. The highest P_x of 53kN occurred at 2.08% drift ratio and –53.7kN at 1.9% drift ratio. The highest value of P_y recorded is 70.2kN at 2.02% drift ratio and -66.2kN at –1.96% drift ratio. However unlike YL-H2, drastic drop of the peak value by more than half occurred after loading stage 6 and 14 for P_x while P_y shows significant drop only during the second cycle of loading at target drift of 2%.

Specimen YL-H2V also failed in punching shear although the widespread yielding of the flexural bars and crushing of the concrete might have indicated that flexure capacity of the slab had been reached. Crushing of the concrete started to occur at 4% target drift ratio when the slab failed in punching shear (Figs. 10a and 10b). A significant drop in load in both the P_x and P_y directions occurred only after loading stage 14 in the first cycle. Peak P_y of 119kN was observed at 4.29% drift ratio and –114kN at –4.24% drift ratio. Peak P_x of 64.7kN at 4.07% drift ratio and –62.5kN at –3.19% drift ratio were recorded. The punching shear surface exhibited in YL-H2V is different with respect to YL-H2 and YL-L2, with part of the critical section along the long side of the column not showing the "punching out" effect. Also it seems that the slope of the punching shear surface at the shorter side of the column is steeper than that at the long side.

Specimen YL-H1V (high gravity, uniaxial lateral loading, with SSR) was loaded to a maximum stroke at around target drift ratio of 8% but there was no significant drop in load (Fig. 11). Crushing of the concrete at bottom of slab occurs at 5% target drift ratio. Peak loadings, P_x of 78 KN were observed at 7.03% drift ratio and –80.7kN at –7.18% drift ratio. Severe pinching of the hysteresis curve shown in Fig 11 suggests that bond failure might have governed the behavior of the connection. While the applied loadings had not dropped beyond 80% of the peak values recorded at target drift ratio of 7%, there were clear indications from the deep flexural cracks and crushing of the concrete (Fig 12c) that flexure failure was imminent. Table 2 summarized the peak values recorded for all five specimens and their corresponding drift ratios.

Connection Strength

The vertical segments in the hysteresis curves for the biaxially loaded specimens show the drop or increase in strength and stiffness of the connection during loading in the prescribed direction (e.g P_x) when the loading in the other direction (e.g P_y) was applied or unloaded respectively. This is due to the effects of column rectangularity and biaxial loading. These effects of column rectangularity and biaxial loading can be best illustrated by the P_y vs P_x curves as shown in Figs 12a to 12c. Note that the curves shown in Fig 12a to 12c represent those from the first cycles only. If the column rectangularity and biaxial loading have no effects on the strength of the connection, the P_y vs P_x curve will closely resemble the shape of the loading path shown in Fig 6b, consisting of four fairly perfect squares with one in each quadrant on the x-y coordinate system. The P_y vs P_x curves show the interaction between the applied lateral forces, P_x and P_y, as the target drift ratio increases. For each target drift ratio, the curve consists instead of four skewed quadrilaterals, with the peak absolute values for each direction being recorded when there is no loading in the other direction. The first quadrant (consisting of loading stages 1, 2, 3, and 4) of the loading path shown in Fig 6b gives a good example. Using specimen YL-H2 as an example (Fig 12a), P_x will record the highest value of 39.7 kN for target drift ratio of 1.5% when P_y is not activated (after loading stage 1). When P_y is being applied and increased to 43.5 kN (after loading stage 2), P_x will decrease in value to 35.2 kN. Correspondingly, the second quadrant (consisting of loading stages 5, 6, 7, and 8) of the loading path in Fig 6b and again using Fig 12a as an example, when P_y is being unloaded from –36.5 kN (after loading stage 6) to 2.2 kN (after loading stage 7), the value of P_x will increase from 33 kN (after loading stage 6) to 37.2 kN after loading stage 7). This interactive relationship between P_x and P_y illustrate the biaxial effect. The effect of column rectangularity can be easily shown by the different values of P_x and P_y recorded at the corners of the loading path i.e. after loading stage 2, 6, 10 and 14. For the above example, compare the value of P_x of 35.2kN recorded after loading stage 2 and value of P_y of 43.5kN. The different values of the applied loadings, P_x and P_y, at the same loading point (after loading stage 2) prove that the strength of the connection in the y-direction is higher than that in the x-direction.

The ratio of the peak P_x to the peak P_y is about 0.95, 0.78 and 0.55 for specimens YL-H2, YL-L2 and YL-H2V respectively. It must be noted here that for YL-H2, the peak values for P_x and P_y were recorded at target drift ratio of 2% and 1.5% respectively.

Biaxial loading also reduced the peak values of P_x and P_y. The fact that all of the biaxially loaded specimens failed in punching shear failure at the corners of the loading path in Fig 6b concludes that the shear stresses are additive when P_x and P_y were applied simultaneously. For specimen YL-H2, the average reduction in strength for P_x due to biaxial effects was 11.5% and that for P_y was 14.4%. The values were calculated at 1.5% target drift ratio. For specimen YL-L2, the average reduction in strength for P_x due to biaxial effects was 11.2% and that for P_y was 17.0%. The values were also calculated at 1.5% target drift ratio. For specimen Yl-H2V, the average reduction in strength for P_x due to biaxial effects was 18.2% and that for P_y was 14.2%. The values were calculated at 4% target drift ratio. It is interesting to note this average reduction for YL-H2V at 1.5% target drift ratio was 11.9% and only 9.9% for P_x and P_y respectively. The above calculations seem to suggest that denser shear reinforcement placed in the short side of the column is also effective in reducing the biaxial effects at lower target drift ratio.

The effects of biaxial loading can also be observed by including the study of specimens YL-L1 and YL-H1V. In each case, both specimens were able to sustain higher drift ratio beyond 1.5% drift ratio without failing in the brittle punching shear mode.

The effect of gravity shear can be observed from Table 2 where comparisons can be drawn between specimens YL-H2 and YL-L2. The ratio for the average peak P_x between YL-H2 and YL-L2 is 0.89 and that for P_y is 0.73, indicating that higher gravity shear results in a bigger drop for loading about the strong column direction i.e. P_y.

The effect of the use of the shear studs is observed by comparing YL-H2 and YL-H2V. Without shear reinforcement, the peak values P_x and P_y for YL-H2 were only 74.8% and 42.7%, respectively, of the peak values of specimen YL-H2V. In addition, the applied lateral load for specimen YL-H1V did not drop beyond 80% of the peak load, a sufficient evidence to suggest the enhancement of the connection strength due to the presence of shear studs. Fig 13a and 13b shows the strain profile of some of the shear studs in YL-H2V. The high strain value or yielding of the shear studs near the column face shows that the shear studs are effective in resisting the shear stresses. The yield strain of the shear studs is about 0.0025.

Drift capacity

The definitions[7] of the ultimate drift ratio, DR_u, and yield drift ratio, DR_y, are shown in Fig 14. DR_u occurs when the load displacement envelope shows a drop of the peak load, P_{max} to 0.8 P_{max} or less. The difficulty in choosing DR_y lies with the changing gradient of the hysteresis curves. Thus there is no distinct point in which one can identify as the yield point. The main reason is that as the load increases, more flexural bars away from the column are engaged to carry the load. In this experimental study, the definition used was that suggested by Pan and Moehle[7]. The yield drift ratio is evaluated by assuming an elasto-plastic relationship for the hysteresis load displacement envelope curve. The initial gradient of the straight line is constructed such that it intersects the load displacement curve at 2/3 P_{max}. Using the above definition, the values of DR_u, DR_y, and ductility ratio μ are tabulated in Table 3.

As had already been described earlier, the specimens loaded uniaxially along the weaker direction (along x-axis) possessed the highest drift capacity, with specimens YL-L1 and YL-H1V having drift capacity of 7.32% and 8.14% respectively, far exceeding the commonly suggested[6, 7] drift limit of 1.5%. On the contrary, for YL-H2 and YL-L2 which were loaded biaxially, their corresponding drift capacity barely exceed 2%. From directly comparing the behavior of YL-L1 and YL-L2(X), it can be seen that the ultimate drift ratio for YL-L2(X) was only 28.4% of that of YL-L1. Even for specimens which were installed with shear studs, see for example YL-H1V and YL-H2V(X), the reduction of the drift capacity was as much as half. In fact, it must be noted here that for YL-H1V, there was no sign of obvious failure. The much higher drift ratios achieved by the uniaxially loaded specimens were due to the higher column flexibility in relation to that of the slab. Note also that the uniaxially loaded specimens were loaded along the weaker direction. Should the lateral loading be applied along the stronger direction, the drift capacity would most likely reduce.

The effect of column rectangularity cannot be evaluated here as the loading steps in the x and y directions were equal in each loading cycle. However, the behavior of

YL-H2 suggests that drift capacity of 1.45% for loading in the stronger y-direction is lower than that in the x-direction (1.96%).

The effect of gravity as shown by the above results is not as significant as expected. Comparing YL-H2 and YL-L2, it can be concluded that while the ultimate drift ratio for YL-L2 in the y direction was 1.39 times higher than that for YL-H2, the ultimate drift ratio in the x-direction of YL-L2 was only 1.06 times higher. Again, it must be stressed that for YL-L2, the slab only showed complete punching shear failure after the completion of the second cycle of target drift ratio of 2%. Failure in the second cycle could be due to the deterioration of the concrete under cyclic loading.

The presence of shear studs significantly increased the drift capacity of the slab column connection. Comparing the response of YL-H2 and YL-H2V, the increase in drift capacity was 2.96 times and 2.08 times higher for loading in the y and x directions respectively for YL-H2V. Again the better performance of the drift ratio capacity in the y direction can be attributed to the denser shear reinforcement along the short side of the column. The improvement in performance due to shear studs can also be observed from specimen YL-H1V which did not fail in punching despite the high gravity loading. Due to the performance enhancement effect of shear studs, the ultimate drift ratio of 8.14% for YL-H1V was achieved. Hence, it can be concluded that the provisions given by the ACI Committee 421[3] for installation and arrangement of the shear studs are effective in increasing the drift capacity beyond 1.5% even for biaxially loaded connections.

Fig. 15 shows the plot of DR_u with respect to the gravity shear ratio, including results from other studies[5-15], which are essentially interior connections with square columns and without shear reinforcement. It can be observed from Fig. 15 that the results of ultimate drift capacity from specimens YL-H2 and YL-L2 provide for a lower bound for all data points. The results from those two specimens suggest that the limit for gravity shear ratio of less than 0.4 in order for the slab column connections to sustain at least 1.5% drift ratio is perhaps unconservative for slab-rectangular column connections. Based on this current experimental study, a limiting gravity shear ratio of 0.28 for column with aspect ratio of five should be imposed if the connections are expected to undergo 1.5% drift ratio and loaded biaxially.

Connection Stiffness

The definition of stiffness is shown in Fig 16 and is defined[5] as the ratio of the absolute peak-to-peak load to the corresponding peak-to-peak drift ratio of the hysteresis curve. With the above definition and the hysteresis curves for each target drift ratio, the stiffness degradation curves can be plotted and studied. For the biaxially loaded specimens, two sets of stiffness degradation curves can be obtained for loading in any one direction. One will be Case A: the stiffness degradation curve when the load in the other direction has not been applied and Case B: when the load in the other direction has already been applied. The second case (Case B), where a drop in strength was observed, is expected to show lower stiffness values as well. As such, for each biaxially loaded specimen, there will be four stiffness degradation curves. And for the uniaxially loaded ones, there will be only one stiffness degradation curve.

The four stiffness degradation curves for the biaxially loaded specimens are shown in Fig 17a to 17c while Fig 17d and 17e show the stiffness degradation curves for all five specimens in the x and y directions respectively. The stiffness representing the biaxially loaded one will be the more critical ones i.e. Case B.

Using the stiffness degradation curves from Fig 17a to 17c, it can be observed that the stiffness of the connection for any direction is always higher when the loading in the other direction is not applied. Using YL-H2 as an example (Fig 17a), the stiffness of the connection in the stronger y-direction is higher when no loading is applied in the x-direction compared to that when loading is applied simultaneously in the x-direction. The same phenomenon occurs to stiffness in the x-direction. Thus it can be easily concluded that biaxial loading decreases stiffness. In addition, the difference is the largest during initial stages of lateral loading. The difference becomes smaller as the specimen is subjected to higher drift. This is because fewer new cracks were formed as the target drift ratio increased. Also, from the same set of curves, the stiffness during loading in the stronger y-direction is always higher than the stiffness of the connection for loading in the weaker x-direction. This is caused by column rectangularity. It can generally be stated for a column aspect ratio of five that the stiffness of the connection when loading is applied along the stronger column direction is approximately 1.6 times higher than the stiffness in the weaker column direction.

From Fig 17d and 17e, the stiffness in the x-direction for YL-L2 is only 1.05 times higher than the same stiffness for YL-H2. For the stiffness in the y-direction, the difference is greater, constituting to 1.16 times. The above two observations suggest that while the magnitude of gravity shear may not affect the stiffness significantly, high gravity shear still decreases the stiffness of the connection.

The stiffness degradation curves for YL-H2V and YL-H1V are almost always higher than those of the other specimens for all target drift ratio, signaling the effectiveness of the shear studs even at earlier stages of loading. Specimens with shear studs have stiffness on the average of 1.4 and 1.57 times higher in the x and y directions, respectively, than the ones without shear studs. Again, the higher stiffness in the stronger y-direction can be attributed to the denser shear reinforcement along the short side of the column.

Ductility

Using the ductility ratio calculated in Table 3, comparisons of ductility ratio for all five specimens can be made. It can be observed that biaxial lateral loading severely decreases the ductility for both loadings in the x and y directions. Figs 18a and 18b show the load displacement envelope for loading in the x and y directions respectively. For rectangular column-slab connections, a uniaxial loading along the weaker x-direction results in ductility ratio greater than 3, which is more than twice the ductility ratios for the biaxial loaded specimens without shear reinforcement. In fact, the ductility ratios for YL-L1 (3.54) and YL-H1V (3.28) are even higher than the ductility ratio in the x direction for YL-H2V (2.91). The effect of column rectangularity on ductility cannot be clearly shown for the three biaxially loaded specimens. The ductility ratios in the x-directions for YL-H2 (1.69), YL-L2 (1.45) and YL-H2V (2.91) are only slightly higher than those corresponding values for YL-H2 (1.61), YL-L2 (1.44) and YL-H2V (1.77) in the y-direction. Shear studs are also found to be effective in increasing the ductility of slab column connection. YL-H2V has ductility ratio in the x and y directions of 1.72 times and 1.1 times higher than those of YL-H2, respectively. The lower than expected value of ductility ratio for YL-H2V in the y-direction can be due to the load displacement envelope for P_y, which shows that at later drift ratios, the curve still increases significantly without any sign of flattening. The fact that P_y is still increasing at a significant rate even at later stages of loading (> 2% drift ratio) suggests that the shear reinforcements installed along the short side of the column continued to carry the stresses caused by the applied loading effectively. The effectiveness of the shear studs can also be attributed to the small spacing of the first studs from the column face, which is 0.35d, comparatively lower than the upper limit of 0.4d set by ACI Committee 421.[3]

Fig. 19 shows the ductility ratios for the five specimens from this present experiment together with experimental results from other researchers.[5-15] In comparison with past experimental results, it can be seen that most of the specimens are able to exhibit some form of ductility (μ >1.0) if the gravity shear ratio is kept below 0.4. Also, despite the low drift capacities for specimens YL-H2 and YL-L2, their ductility ratios are greater than 1.4, indicating some form of ductility and moment redistribution within the slab before punching shear failure.

ACI 318-02

The ACI 318-02 uses the eccentric shear stress model to calculate the shear capacity of the slab-column connections. In the eccentric shear stress model, part of the unbalanced moment, M_{ub} is transferred by shear, $\gamma_v M_{ub}$ and the rest is transferred by flexure, $\gamma_f M_{ub}$. Shear stress distribution caused by shear force transferred between the slab and the column is assumed to be uniform around the critical section calculated at 0.5d away from the column face. The total shear stress as the result of unbalanced moments about the x and y axes and shear force transferred are added using equation (2) and (3).

$$v_u = \frac{V_u}{b_o d} + \frac{\gamma_{vy} M_{uy} x_{max}}{J_{cy}} + \frac{\gamma_{vx} M_{ux} y_{max}}{J_{cx}} \tag{2}$$

where

$$\gamma_v = 1 - \gamma_f = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} \tag{3}$$

where b_1 is the width of critical section, measured in the direction of the span for which moments are determined; b_2 is the perpendicular width; γ_f is the proportion of unbalanced moments carried by flexure; M_{uy} and M_{ux} are the unbalanced moments transferred about the y and x axes, respectively; J_{cy} and J_{cx} are analogous to the polar moment of inertia of the critical section about y and x axes, respectively; x_{max} and y_{max} are the distances from the centroid of the critical section in the respective directions. In absence of reinforcement, $v_n = v_c$ = minimum of equations (4a) to (4c):

$$v_c = (1 + \frac{2}{\beta_c})\frac{\sqrt{f_c'}}{6} \tag{4a}$$

$$v_c = (2 + \frac{\alpha_s d}{b_o})\frac{\sqrt{f_c'}}{12} \qquad (4b)$$

$$v_c = \frac{1}{3}\sqrt{f_c'} \qquad (4c)$$

In presence of shear reinforcement:

$$v_n = v_c + v_s \qquad (4d)$$

where

$$v_c = \frac{1}{6}\sqrt{f_c'} \qquad (4e)$$

$$v_s = \frac{f_{vy} A_v}{b_o s} \qquad (4f)$$

where s is the spacing of shear reinforcement, f_{vy} = yield strength of shear reinforcement, and A_v is the area of one peripheral perimeter of shear reinforcement.

Table 4 shows the results of the calculations using the eccentric shear model. The calculated v_u is compared with v_n calculated using equations (4a) to (4f). In addition, sufficient flexural reinforcements should be placed across a width of c_2+3h to enable the transfer of unbalanced moment by flexural stresses. c_2 is the width of column transverse to loading direction and h is overall depth of slab.

Columns 2 and 3 of Table 4 show the experimental ultimate unbalanced moments transferred about the y and x axes, respectively. Column 4 shows the shear stress calculated using equations (2) and (3). Column 5 shows the nominal shear strength of the connection calculated using equations (4a) to (4f), and Column 6 shows the shear strength ratio. Columns 7 and 8 show the unbalanced moments transferred by flexure about the y and x axes, M_{fy} and M_{fx}, respectively. Columns 9 and 10 show the nominal flexural strength of the slab strip of c_2 + 3h wide in y and x directions, M_{sy} and M_{sx}, respectively. Columns 11 and 12 show the ratios of Columns 7 and 9 and Columns 8 and 10, respectively.

The higher value between columns 6, 11, and 12 will govern the behavior of the connection. For YL-L1, the eccentric shear model is able to predict the behavior correctly as it predicts that the slab will fail by flexure first indicated by a higher ratio of 1.15 in Column 11, which is also close to unity. For YL-H2 and YL-L2, while the eccentric shear model is able to predict punching shear failure for the specimens correctly, it overestimates the ratio (underestimate the actual strength) by 49% and 63% respectively. The reason can be in the form of equation (2), which suggests that the connection will fail once the shear stress at a single point in the critical section exceeds v_n. For YL-H2V, the ACI method actually predicts that flexural failure might occur first although, punching shear was observed during the experiment. This can be explained as follows. While the unbalanced moment to be transferred by flexure has exceeded the flexural capacity of the slab for width, c_2 + 3h, the flexural behavior of the slab would allow for moment redistribution to occur across a larger width, hence a immediate flexural failure across c_2 + 3h width was avoided or delayed. The same can be said of YL-H1V. Note however