# INFLUENCE OF FLEXURAL REINFORCEMENT RATIOS ON FAILURE MODES OF SLAB-COLUMN CONNECTIONS: AN EXPERIMENTAL INVESTIGATION

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***ABSTRACT:*** *This paper presents an experimental investigation on the influence of flexural reinforcement ratios on failure modes in interior slab-column connections. Five specimens with same slab depths of 150 mm and reinforcement ratios varying between 0.75 and 1.25 percent are tested under concentric loading. The test results reveal that specimens containing low reinforcement ratios exhibit a more ductile flexural failure than those with high reinforcement ratios. The results compared with ACI 318-11 code equations reveal that ACI code underestimates the punching shear strength of slab-column connections and this underestimation becomes more pronounced with increase in the reinforcement ratios. The higher ratios of measured peak loads to the flexural capacities attribute to the low reinforcement ratios.*

**Keywords:** punching, shear, flexure, reinforcement ratio, slab-column connections, failure modes.

## 1. INTRODUCTION

Flat-plate floor system is currently used in a wide range of concrete structures. The absence of beams, ease of construction, less total building height, shorter construction time, and modularity in floor partitioning are just some of the advantages of using flat-plate floor system. Unfortunately, this system is associated with a high risk of brittle failure at the slab-column connection regions. This risk is higher whenever there are errors in design and construction, damage to concrete due to deterioration or corrosion of the flexural steel reinforcement, increase in service loads due to changes in the use of structures [1-3].

The reinforced concrete slab-column connection shown in Fig. 1 can reach its shear or flexural capacity and fail in two modes: punching shear or widespread yielding of flexural reinforcement. Prevailing of the failure mode depends on the amount of flexural reinforcement, and thus on the flexural capacity of the slab. If flexural capacity of the slab is much higher than the shear strength of concrete, then a brittle punching shear failure will occur and vice versa. Failure of a slab-column connection in punching shear will be brittle; whereas the connection exhibiting large deformations before failure will fail in flexure. In the absence of shear reinforcement, the slabs with relatively small reinforcement ratios $\rho$, fail in flexural mode when $V_{flex} < V_n$, where $V_{flex}$ is flexural capacity and $V_n$ is nominal shear strength provided by the concrete. In most tests, experimental value of the actual shear strength is higher than the conservative nominal value given by the design codes; a difference of 25% is not uncommon. Thus, to design a test specimen that would fail by flexure or by punching, the reinforcement ratio $\rho$ should be, respectively, well below or well above the ratio that gives $V_{flex}$ equal to $V_n$. Otherwise, mode of failure will not be clear from the tests because it can be a combination of the two modes. A slab with low flexural reinforcement ratio for example, $V_{flex} \leq 0.6\, V_n$ is expected to fail at a load that can be predicted by the yield line theory [4] and the failure mode will be ductile. A slab-column connection with low or medium flexural reinforcement ratio tend to fail in flexure rather than in punching shear. The deformations preceding the punching shear failure is a function of flexural and shear reinforcements. Punching may then occur as a secondary phenomenon after the slab reaches its flexural strength. A slab-column connection with a high flexural reinforcement ratio fail in brittle manner. In slab-column connections, with normal reinforcement ratios, some flexural rebars located in the vicinity of connection regions, especially crossing the columns, yield before occurrence of punching shear failures.

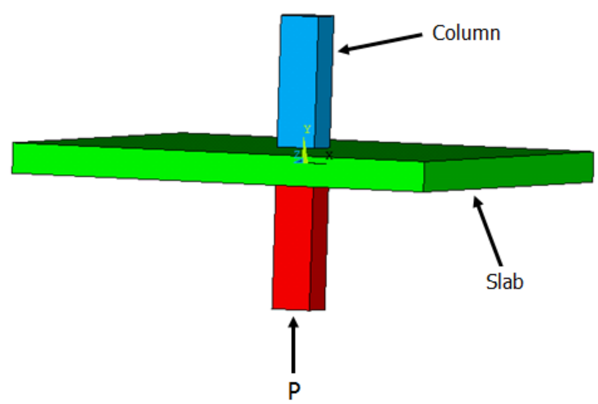



**Fig.1: Slab-column connection under concentric loading**

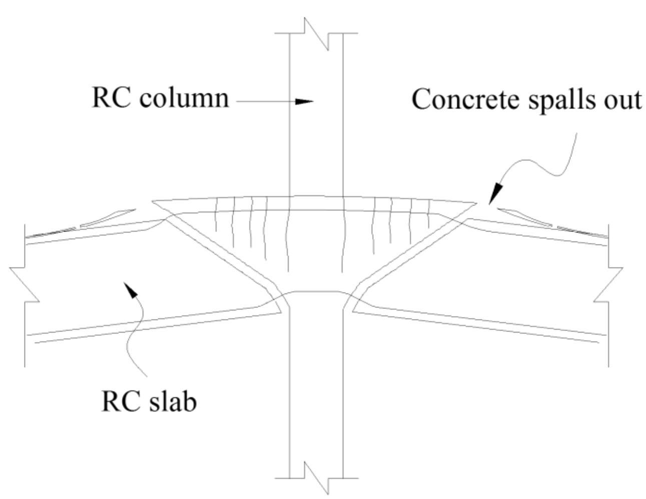



**Fig.2: Behavior of slab-column connection during a punching shear failure**

The cover to flexural reinforcement at tension zone of the slab is pushed outwards with spalling of the concrete as shown in Fig. 2. However, deformation is limited, and the failure is sudden and without any warning. The punching shear behavior is particularly crucial because it may occur at a load lower than the flexural capacity of the structure. Many factors influence this brittle punching shear failure such as distribution of cracks, reinforcement ratio, compressive stresses in the concrete, and depth of the neutral axis. In particular, reinforcement ratio usually plays an important role in determining the failure mode of slab-column connections. Heavily reinforced concrete slabs typically display abrupt punching shear failure, contrary to lightly reinforced concrete slabs that fail in flexure with the excessive deformations.

## 2. RESEARCH SIGNIFICANCE

A flat-plate floor system in a modern building should be able to reach its flexural capacity and undergo substantial additional deflection without capacity loss to prevent a brittle failure in case of accidental loading. The aim of this study is to investigate the failure behavior of slab-column connections under varying reinforcement ratios and examine the deformation, load-carrying capacity, and the failure modes.

## 3. LITERATURE REVIEW

Here, focus has been oriented towards difference between flexural and punching shear failure modes. As would be logically expected, both load-carrying capacity and punching shear strength of RC flat-plates tend to increase with increase in the flexural reinforcement ratios. Due to this fact, many researchers included the reinforcement ratio as a variable in their research programs, especially between the 1930's and 1970's [5]. The experimental results obtained by Menétrey [6] on octagonal slabs (thickness: 120 mm; radius of support: 550 mm) differing only in their flexural reinforcement ratios demonstrate this difference. The influence of addition of steel reinforcement on ultimate punching capacity was also studied by Marzouk and Hussein [7]. They examined behavior of two-way slabs through experimental testing. They found that by increasing reinforcement ratio from 1.1% to 2.3% and from 1.2% to 2.4% for slab thickness of 150 and 120 mm the ultimate punching shear loads were increased. They also concluded that the degree to which yielding spread in the reinforcement varies with the reinforcement ratio. In their tests, yielding of the steel reinforcements of reinforced concrete slabs with high reinforcement ratios occurred at comparatively higher loads, but were localized to the stub columns. On the other hand, in lightly reinforced concrete slabs, yielding initiated at stub columns and gradually spread throughout the whole length of the tensile reinforcements. Another study [8] was performed on 2 x 2 x 0.2 m slab specimens with a centrally located column 0.3 x 0.3 m in cross-section. The tensile flexural reinforcement used in the first and second series was 1.23 and 1.53 percent, respectively. Measurements for loads, deflection, and strains were recorded. Researchers found experimentally that ACI code provisions for computation of shear strength considering the reinforcement contribution is justifiably conservative at low reinforcement ratios (up to about 0.6 percent).

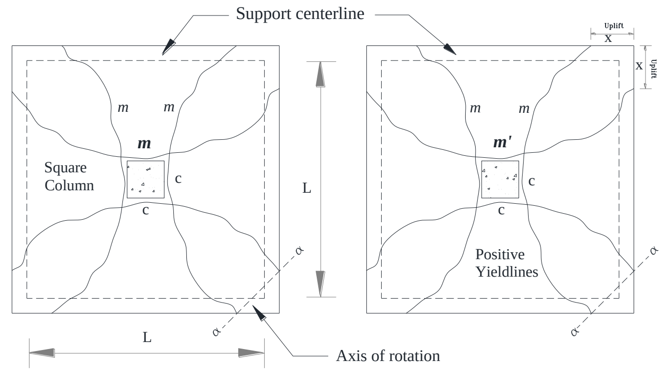



**Fig.3: Typical yield-line patterns [9].**

The study performed by [9] outlined a solution to the flexural collapse load in terms of the concentrated load or column reaction, $V_{flex}$ using the principle of virtual work which states that, if a rigid body is in a static state of equilibrium under a system of forces giving a virtual displacement, $\delta$ then sum of the virtual work done by the forces is zero. In other words, when a slab is subjected to an arbitrary displacement, the work done by the internal forces (moment capacity per unit width of slab section multiplied by the length of yield-line multiplied by rotation of the segments on either side of the yield line) must equal the work done by the external forces (total applied load multiplied by the distance). Thus; in Fig.3,

Internal virtual work = $\Sigma\, m\, l\, \theta$ (1)

$$m\,(L-2\,x)\,\frac{2\delta}{L-C}+4\,m\,x\sqrt{2}\,\frac{\sqrt{2}\,\delta}{L-c-x} \quad (2)$$

Neglecting slab self-weight and size of loading area:

External virtual work = $V_{flex}.\,\delta$ (3)

where, $V_{flex}$ is the concentrated vertical load or column reaction. Assuming a unit deflection, $\delta$ at the stub columns and equating internal virtual work to the external virtual work, we can write:

$$V_{flex}=4\,m\,(L-2x)\,\frac{2}{L-c}+4\,m\,x\,\frac{2}{L-c-x} \quad (4)$$

Solving for $x$ in terms of minimum value of $V_{flex}$:

$$\frac{\partial V_{flex}}{\partial x}=0 \quad (5)$$

$$x=\left(1-\frac{1}{2}\sqrt{2}\right)\,(L-c) \quad (6)$$

$$V_{flex}=8\,m\left(\frac{1}{1-\frac{c}{L}}-3+2\sqrt{2}\right) \quad (7)$$

Equation (7) is applicable to predict the flexural capacity (or column reaction) that causes a flexural collapse in squared reinforced concrete flat-plates incorporated with a square central column, assuming the reinforcement ratio as constant across the entire slab width and with no reinforcement concentration over the column. Alternatively, for concentrated reinforcement over the column, the radial moment resistance in the 'column strip', $m_c$ and the radial moment resistance in the 'outer strips', $m_o$, shall give flexural capacity or column reaction, $V_{flex}$ as:

$$V_{flex}=8\,m\,\left(\frac{1}{1-\frac{c}{L}}-3+2\sqrt{2}+\frac{\frac{m'}{m}-1}{\frac{L}{c}-1}\right) \quad (8)$$

where, $m$ is nominal flexural resistance or bending moment per unit length at yielding of the flexural reinforcement determined by ACI 318-11 code provisions by performing a sectional analysis in a unit slab width (i.e. 1 $m$).

$$m = \rho\, f_y\, d^2 \left\{1 - 0.59\, \left(\rho\, \frac{f_y}{f_c'}\right)\right\} \quad (9)$$

In above equations, $\rho$ is the tensile flexural reinforcement ratio, $L$ is the side dimension between supports of a square slab, $c$ is the side dimension of a square column, $d$ is the effective slab depth, $f_y$ is the yield strength of the flexural reinforcement, and $f_c'$ is concrete cylinder strength of the slab.

The provisions of building design code ACI 318-11 [10] section 11.11.2 require that in the design of slab-column connections (non-prestressed slabs and without shear reinforcement), the nominal shear strength carried by concrete at the centroid of the shear critical section is computed from the smallest of equations (10) to (12).

$$v_c = 0.333\sqrt{f_c'} \quad (10)$$

$$v_c = 0.167\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'} \quad (11)$$

$$v_c = 0.083\left(\frac{\alpha_s . d}{b_o} + 2\right)\sqrt{f_c'} \quad (12)$$

where,

$v_c =$ shear strength of concrete (MPa)

$f_c' =$ concrete compressive strength (MPa)

$b_o =$ perimeter of the shear critical section located at a distance $d/2$ away from the column faces, concentrated loads, or reaction areas (mm).

$d =$ slab depth equal to the distance from compression face of slab to a layer midway between centroids of flexural reinforcements running in x and y directions (mm)

$\beta_c =$ ratio of longer to shorter sides of the column.

$\alpha_s =$ modification factor for support type (40, 30 and 20 for interior, edge and corner columns, respectively).

The ultimate concentric load (in N), required to fail the slab column connection into punching shear, shall be equal to:

$$V_u = \phi\, v_c\, b_o\, d \quad (13)$$

## 4. EXPERIMENTAL PROGRAM

Properties of the structural materials (concrete, and steel) used in the construction of slab-column specimens, were measured by performing standard tests. For construction of specimens, concrete was ordered from a local ready-mix company with a target compressive strength of 25 MPa to simulate the concrete strength in existing reinforced concrete flat-plate structures built in mid-20th century. During casting of slab-column specimens, many 150 x 300 mm concrete cylinders and 150 x 150 x 600 mm un-reinforced concrete beams were also cast to perform compressive, split cylinder and bending tests. Some of the concrete cylinders were placed in lime saturated water for standard curing whereas remaining concrete cylinders and beams were cured under ambient conditions for equivalent days as slab-column specimens. The concrete cylinders cured under standard conditions were tested at 28 days to assertion compressive strength of the supplied concrete while those cured under ambient conditions were tested on the day of test of the corresponding slab-column specimens for determination of compressive, and tensile strengths. The average concrete cylinder strength $f_c'$ cured under standard conditions was measured as 28.2 MPa, slightly higher than the specified concrete compressive strength. The concrete specimens cured under ambient conditions, and tested on the day of testing the corresponding slab-column specimens, yielded average compressive $(f_c')$, tensile $(f_{ct})$, and flexural $(f_b)$ strengths of 25.7, 2.9, and 8.3 MPa, respectively. A summary of the concrete strengths is presented in Table 1.

**Table 1: Concrete strengths of tested specimens**

| Specimen's designation | Steel ratio $(\rho_x = \rho_y)$ | Concrete strength (MPa) | | | Age on test date (Days) |
|---|---|---|---|---|---|
| | | $(f_c')$ | $(f_{ct})$ | $(f_b)$ | |
| FP-1 | 0.0125 | 25.3 | 2.9 | 8.0 | 36 |
| FP-2 | 0.0114 | 25.8 | 3.0 | 8.7 | 38 |
| FP-3 | 0.0101 | 25.2 | 2.7 | 8.4 | 39 |
| FP-4 | 0.0088 | 26.0 | 2.9 | 8.3 | 40 |
| FP-5 | 0.0075 | 26.2 | 2.8 | 8.1 | 42 |

The reinforcing steel of Grade 60 consisted of Ø20, Ø14 and Ø8 mm deformed bars was used in the construction of slab-column specimens. The steel was delivered to the Structural Engineering Laboratory in one shipment. In order to determine actual stress-strain characteristics of the steel reinforcements, samples were tested in accordance with ASTM A615 standards [11] by using an Instron tensile test machine. Three samples of each rebar size were selected randomly and were subjected to uniaxial tensile testing. A summary of mechanical properties of tested rebars is presented in Table 2.

**Table 2: Mechanical properties of tested rebars**

| Size (mm) | Steel area ($mm^2$) | Strength | | Strain | |
|---|---|---|---|---|---|
| | | Yield | Ultimate | Yield | Ultimate |
| 8 | 50 | 444 | 678 | 2755 | 104750 |
| 14 | 153 | 433 | 695 | 2469 | 121898 |
| 20 | 314 | 412 | 673 | 2878 | 108594 |

The test specimens presented herein are representative of the interior slab-column connections in a reinforced concrete flat-plate prototype structure. The dimensions of the test specimens were determined on the basis of regions of negative bending moments around an interior slab-column connection (approximately 0.4 times the span). A total of five reinforced concrete flat-plates (FP1, FP2, FP3, FP4, FP5) with dimensions 2200 x 2200 x 150 mm were designed, constructed and tested under concentric gravity loading. A stub column of squared cross-section of 200 x 200 mm and height of 650 mm was extruded from each top and bottom face of the slabs. Loading was applied through lower stub column; thus the tension reinforcement was placed on top side of the slab. The specimens were designed with varying reinforcement ratios. The flexural capacity, $V_{flex}$ was computed using yield-line theory and shear capacity, $V_n$ was obtained using ACI 318-11 code equations. The details of steel reinforcements and dimensions of the slab-column specimens are shown in Fig. 4.

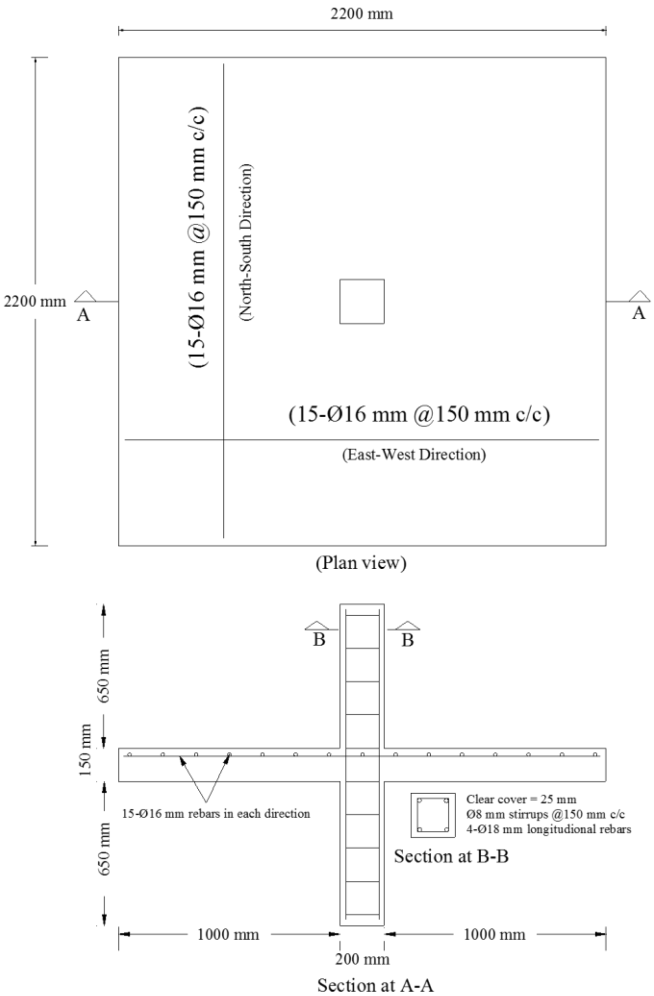



**Fig. 4: Typical dimensions and reinforcement details**

## 5. DISCUSSION OF RESULTS

The test observations are described in terms of recorded deflections, loads, strains, and failure modes.

1. Ratio $\phi$, of predicted shear strength ($V_n$) obtained by using ACI 318-11 code equations to the flexural strength ($V_{flex}$) computed by the yield-line analysis ranged between 0.7 and 1.1. This ratio is often used to divide, theoretically, the punching failure into those due to shear ($\phi$ < 1) and those due primarily to flexure ($\phi$ ≥1). Using this criterion, all specimens, except FP5, were predicted to fail in punching shear. The measured ultimate loads, $P_u$ equal to 356.4, 394.1, 340.3, 359.3 and 335.3 kN, correspond to the specimens FP1, FP2, FP3, FP4 and FP5, respectively. The ratios $\phi_{o,}$ of measured ultimate loads to the flexural capacities in case of FP1 is 0.87 and for the remaining specimens are above the unity. This shows that the specimen FP1 was weaker in shear, and failed before reaching the flexural capacity without any yielding of reinforcement, indicating a punching shear failure. The measured ultimate loads in case of FP2 through FP5 are higher than their predicted shear and flexural loads. Although, a complete ductile behavior with a longer yield plateau was not achieved, however it reveals that the failures took place due to yielding of reinforcements. This agrees with the research results reported by Elstner and Hognestad [11], and Criswell [12] in which yield-line strengths of the slabs were attained, thus, punching failures took place due to the lack of over-strength (reserve strength) and ductility needed for a yield mechanism to occur. In their experiments, the ultimate loads for flexural mechanisms were observed to be about 10 to 25% above the yield-line loads and they reported that this difference was attributed to the possible strain hardening and membrane action that occurred during a flexural mechanism but are not accounted for in the yield-line analysis. The results obtained in this study are presented in Table 3 which shows that ratio of, measured ultimate load to flexural capacity increased with the decrease in reinforcement ratio. Slab corners were free to lift, as would occur in an actual structure. The average uplift of slab corners was measured through LVDTs. In general, the slab corners lift-off was measured to be one-fifth to one-sixth of the upward deflection.

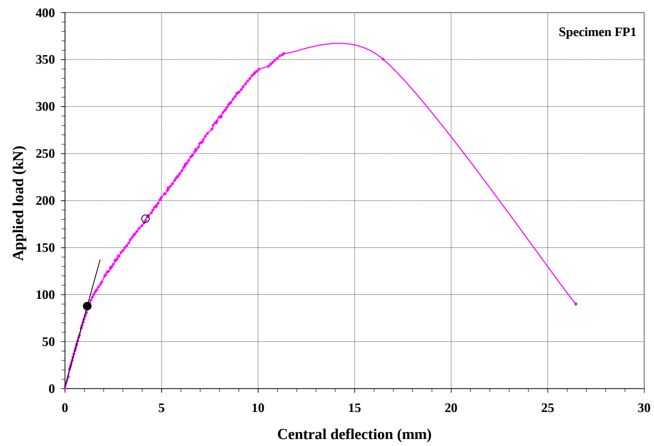
**Table 3: Ultimate loads of tested slab-column specimens**

| en Desi | Predicted ultimate loads (kN) | | | Measured ultimate loads, $P_u$ | $\frac{P_u}{V_{flex}}$ | ure Typ |
|---|---|---|---|---|---|---|
| | $V_n$ | $V_{flex}$ | $V_{flex}/V_n$ | | | |
| FP-1 | 283 | 406 | 0.70 | 356.4 | 0.87 | PS* |
| FP-2 | 283 | 374 | 0.76 | 394.1 | 1.05 | PF† |
| FP-3 | 283 | 337 | 0.84 | 340.3 | 1.00 | PF |
| FP-4 | 283 | 298 | 0.95 | 359.3 | 1.20 | PF |
| FP-5 | 283 | 257 | 1.10 | 335.3 | 1.30 | PF |

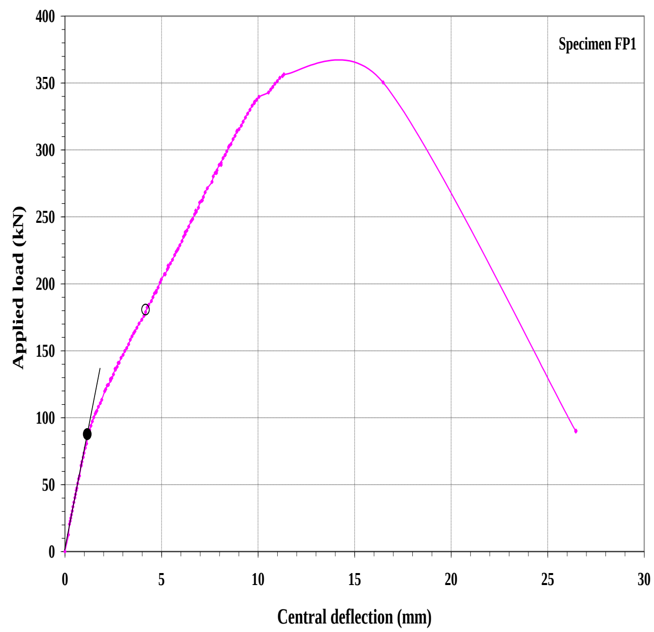
*PS = punching shear; †PF = punching flexure

2. Slab corners were free to lift, as would occur in an actual structure. The average uplift of slab corners was measured through LVDTs. In general, the slab corners lift-off was measured to be one-fifth to one-sixth of the upward deflection.

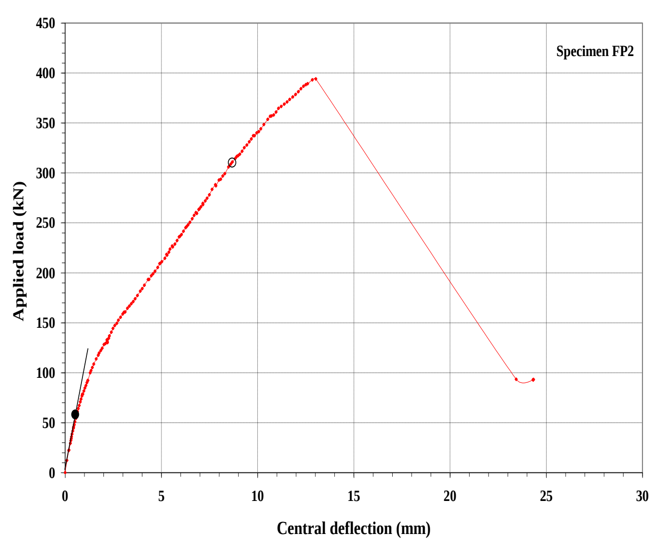
3. The relative deflection is calculated by subtracting the average deflection measured at edges from the average deflection measured at center of the slab. Applied load versus relative deflection plots for tested specimens are shown in Figs. 5 through 9. The load-deflection plots can be used in classifying the punching failure type: punching shear and punching flexural failures. Punching flexural failure takes place in slabs in which a longer plastic deformations plateau is obtained by yielding of most of the flexural reinforcement and vice versa. At ultimate loads $P_u$,the average deflection calculated for specimens FP1 through FP3 ranged from 11.3 to 13.4 mm shows that these specimens failed abruptly (in punching shear) without exhibiting longer plastic deformations. The specimens FP4 and FP5 showed an increased plastic deformation plateaus about 40% more than their companions, and are characterized as punching flexure failures. For all specimens, pre-cracking behavior and stiffness are almost similar, but the post-cracking behavior is different.



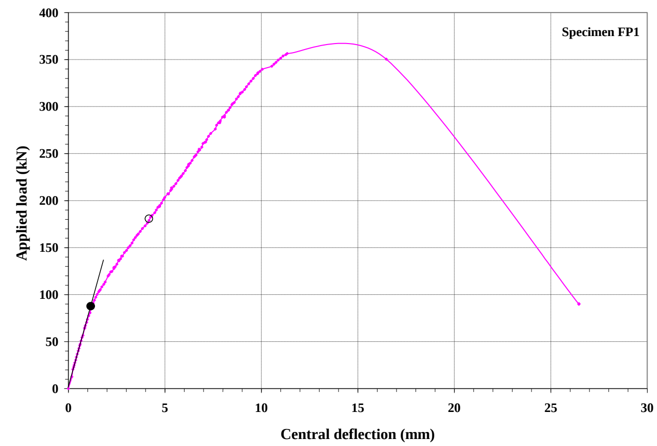

Specimen FP1



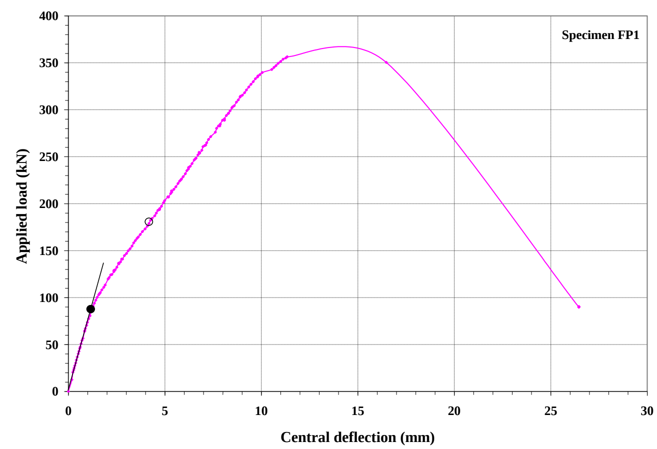

Specimen FP2




Specimen FP3




a) Specimen FP4




b) Specimen FP5

**Fig. 4: Load-deflection diagrams of tested specimens**

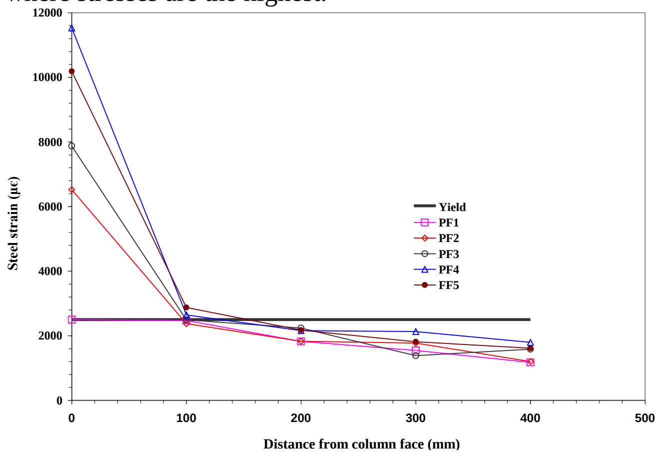
4. Initial tangents to load-deflection plots are drawn and the point where load-deflection behavior deviated from the initial elastic response gave an indication of the point of first cracking. Cracking, yield and ultimate failure loads along with corresponding deflections are all summarized in Table 4. It is observed that, in load-deflection plots, after a short linear elastic branch, tangential and radial cracking strongly reduces the stiffness of the specimens significantly. From the data presented in Table 4, no information is deduced due to wider variation.

**Table 4: Deflection characteristics of tested specimens**

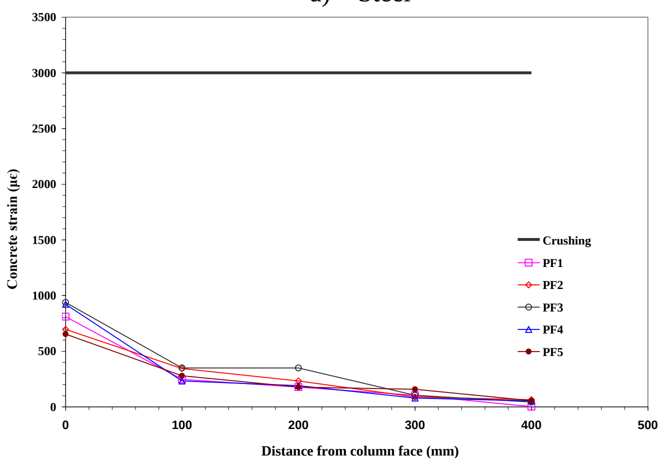
| Spe cim en's | First crack | | Yield | | Ultimate | |
|---|---|---|---|---|---|---|
| | d ( | t i o | d ( | t i o | d ( | t i o |
| FP-1 | 86 | 1.2 | 355.2 | 11.3 | 356.4 | 11.3 |
| FP-2 | 58 | 0.6 | 310.9 | 8.7 | 394.1 | 13.0 |
| FP-3 | 87 | 0.9 | 205.7 | 5.0 | 340.3 | 13.4 |
| FP-4 | 108 | 1.5 | 256.3 | 8.1 | 359.3 | 16.3 |
| FP-5 | 64 | 0.7 | 251.5 | 8.4 | 335.3 | 15.7 |

5. Fig. 5(a) shows strain measured on the flexural reinforcement at several locations. The horizontal line in Fig. 5(a) illustrates the yield strain (2469με) for rebars, determined from uniaxial tension tests. The measured steel

strain ranged between 0.001 and 0.01, indicating that yielding of flexural reinforcement exceeded from the yield strain for all tested specimens except FP1. In case of FP1, the yield load (355.2 kN), is almost equal to the measured ultimate load of 356.4 kN, which shows that specimen failed without any yielding of the flexural reinforcement, thus punching shear failure occurred. Most of the yielding in flexural reinforcements occurred, as expected, at the column faces where stresses are the highest.




a) Steel




b) Concrete

**Fig. 5: Strain Profile**

6. Fig. 5(b) shows concrete strain profile measured on top surfaces of the specimens to observe the strain levels during loading and at failures. The horizontal line in Fig. 6(b) illustrates average value of crushing strain of concrete taken as 3000με. The measured compressive strain is much less than the concrete crushing strain. The measurement of very low concrete strain suggests that the punching failure was of the shear-compression type. \

## 6. CONCLUSIONS

Based on the observed behavior and test results, following conclusions have been drawn:

1. The specimens attaining their flexural capacities with extensive yielding, cracking, and deflections lead to punching flexure failures.
2. Steel and concrete strains decrease with the increasing distances from the column face.
3. As expected, the ultimate punching-shear load increased as the reinforcement ratio was increased.
4. With the increase in the level of reinforcement, the punching strength of the slabs is also increased.

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