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PUNCHING SHEAR BEHAVIOR AND STRENGTH OF FLAT SLABS

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Abstract

Prestressed and reinforced concrete flat plates are susceptible to failure due to punching shear. Modeling the punching shear strength of prestressed concrete slabs poses conceptual challenges, as it involves various factors, including the bonding of tendons, stress in tendons during punching failure (which can be indeterminate), tendon banding, and the presence of draped tendons that yield a beneficial vertical component of the prestressing force. Additionally, prestressed concrete slabs typically require supplementary bonded reinforcement.

The objective of this research is to comprehensively investigate the physical behavior of each test specimen, considering parameters such as deflections, crack patterns, crack widths, mode of failure, and to compare the punching shear resistance of interior slab-column connections with various code provisions. The study involved testing a total of five interior slab-column connections, each measuring 1600x1600 mm in size, 80 mm in thickness, with square columns of 180x180 mm and a height of 300 mm.

Keywords: Slab, Punching, Shear, Design, Codes, prestressing.

1. Introduction

Structural system consisting of slabs with uniform thickness supported directly on column without beams is called a flat slab system. Flat slabs are an economical structural system for medium height residential and office buildings. Prestressed concrete flat slabs have additional advantages such as reducing deflection problems and possibility of using larger span - thickness ratios. However, prestressed concrete and reinforced concrete flat slabs are susceptible to failure by punching shear. Punching shear is an undesirable mode of failure in that it occurs without warning and can lead to progressive collapse of large area of slab, or even of complete structures. Punching shear always occurs in regions of large moment, and flexural cracks are observed around the periphery of the loaded area or support. The objective of this paper was to study the following parameter on the behaviour of interior presterssed flat slabs connection:

1- Number of cables in specimens

2- Prestrissing force in slab

2. Building Code Requirements

The code provisions of (ACI, 1989); (ACI-ASCE, 1989); (BS, 1997); CSA A23.3-94, ; (CEB-FIP., 1990), Model code (Comite Euro-International du Beton and Federation international de la Precontraite) are similar in that the punching shear strength of a concrete slab is checked by calculating a nominal shear stress on a control perimeter some fraction of the slab depth away from the column. The mean shear stress acting over this control surface is related usually to the strength of concrete. The existing design procedures and code provisions are based primarily on empirically derived equations that do not necessarily model the mechanism of failure but have been chosen primarily for their simplicity in use and the wide range of conditions over which they produce acceptable results.

The nominal shear capacities of the tested slabs are compared with the nominal shear capacities predicted by the different codes of practice. A detailed account for the provisions of the different codes is given below. The comparison is beneficial for determining the possible conservatism or non-conservatism of some codes.

- **ACI 318 - 02**

ACI- 318-02 specifies that the shear capacity be calculated on the minimum perimeter located at a distance *d/*2 from the periphery of the column or the concentrated load. These provisions follow from the work of ACI-ASCE Committee 423. The punching shear strength around interior columns of two-way prestressed concrete slabs can be predicted by:

$$Vc = 0.083(\beta_p \sqrt{f_{ck}} + 0.3 f_{pc})b_0 d + V_p \tag{1}$$

- **BS 8110-97**

The British Code used a rectangular control perimeter 1.5*d* from the loaded area for both circular and rectangular loaded areas. For reinforced concrete flat slabs. The available shear force can be calculated from.

$$\frac{V_{eff}}{b_0 d} < V_{CBS} = 0.79\left(100\rho \frac{f_{cu}}{25}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4} < 5.0 < 0.8\sqrt{f_{cu}}\,MPa \tag{2}$$

- **Gardner's method**

(Gardner, 1996) proposed a prediction equation for the punching shear strength of interior slab-column connections of reinforced and prestressed concrete flat slabs, by extending the work of (Shehata & Regan, 1989); (Shehata, 1990), Gardner examined the dependence of the punching shear strength to the concrete strength and the strength for reinforced and prestressed concrete slabs using a control perimeter at the periphery of the loaded area and a (Shehata & Shehata Lidia, 1989) type strength enhancement expression. Columns with circular or rectangular cross sections were analyzed as square columns of the same cross-sectional area. A sensitivity analysis, using the coefficient of variation of the equation coefficient as the criterion of goodness, was used to confirm that the one-third power of concrete strength and steel force was close to optimal. For the unbonded post-tensioned slabs, the prestressed reinforcement ratio was calculated from the initial precompression at the column. i.e., $\rho_p = f_{pc}h/f_{se}d_p$.

The shear force V_d, appropriate to the decompression moment. Was calculated by $V_d \simeq 2\pi\rho_p d_p f_{ps}\,(d_p - h/3)$ (developed from the yield line expression for punching shear $V_{yield} = 2\pi M$; $M=$ yield moment per unit width, (Shehata & Shehata Lidia, 1989). This avoids the problem of determining the inclination of the prestressing tendons crossing the failure surface required to calculate the vertical component of the prestressing force. For design purposes the decompression shear force is multiplied by 0.75 to represent the lower 95 percent bound.

$$V_c = 0.55\lambda u d_{eff}\left[1+\left(\frac{250}{h}\right)^{0.5}\right]\left(\frac{h}{4c}\right)^{0.5} \times \left(\frac{\rho f_y d^3 + \rho_p f_{ps} d_p^3}{d_{eff}^3}\right)^{1/3}\left[f_{ck}^{(1/3)}\right] + 0.75V_d\left[\frac{u}{4c}\right] \quad (3)$$

3. Description of Model Test Specimen

The slab-column connections were tested in this study is one-third scale models. The slab has 160x160-mm side length and 80 mm thickness and is used to represent full-scale flat slab with 9000 mm span and 240 mm thickness. The dimensions were chosen to represent the zone of negative bending moments or lines of contraflexure around an interior column in a prestressed flat plat.

The materials used in this work were locally produced ordinary Portland cement, natural aggregates and tap drinking water. The compressive strength, setting times, expansion and fineness of cement satisfied the requirements of the Egyptian standard specifications. Graded crushed natural dolomite stone of maximum nominal size of 20 mm had been used as a coarse aggregates and natural siliceous sand had been used as fine aggregates. The ratio between coarse and fine aggregates used in this work was 2:1. The cement content was 350 kg/m^3 and W/C ratio was 0.5.

Dry components (gravel, sand and cement) were firstly mixed mechanically for one minute to ensure uniformity of the mix. Then water is added and mixed thoroughly. As soon as mixing completed,, concrete is cast in the forms and moulds and then compacted by using standard methods. The forms were designed in such a way to ensure water tightness and easy stripping. After 7 days all specimens were removed from moulds, and cured for 28 days. The resulted compressive strength of the produced concrete was between 300 and 320 kg/cm^2.

4. Test setup and Loading Frame

The loading frame of the structural and materials testing laboratory was applying monotonically increasing vertical static load. The slab was simply supported on four sides, resting on strips of neoprene rubber of 80-cm width and 15 mm thickness placed on the center of the flange of the supporting steel beam (See Fig 1).

The loading frame consists mainly of the following parts:

- Three I-beam fixed at right angle and fixed by angles and bolts.
- A small I-beam fixed vertically to horizontal I-beam on the floor to rest dial gauges.

The force was applied on to the column using a hydraulic jack of 250 kN capacities. The loading parts consist of two plates having dimension 300x300 mm and 20 mm thick placed at a distance 500 mm from the c.g of column to achieve uniform loading along all column width. The load cell, which was connected to the digital reading device, was placed on top of the upper plate.

5. Measuring Devices

5.1. Deflection

Dial gauges with an accuracy of 1/100 mm and total displacement of 50 mm were used for deflection measurements at the top face of the slabs. Four dial gauges were fixed at the level of the top of steel beam fixed at the right of the loading frame to measure the deflection of the slab.

5.2. Strain measurements

Strain gauges type KFG-5-120-C1-11 of 5-mm length was used to measure the strains on the strand in two directions and flexural reinforcement. For each slab, the electrical strain gauges were connected to a strain reading device. The accuracy of the strain indicator is 1x10-6. The locations of the strain gauges are shown in Figs 2. (a, b, c, d). Two-strain gauge were at the face of column in the tension side of reinforcement, one was at distance 50 mm from the column face and one at 150 mm from the face of column. Two strain gauges wear connected with the strands in each direction; one was in middle strand at distance 50 mm from column in two directions. And one in anther strand which beyond the middle strand in two direction. The steel and strand strain was measured and recorded using strain indicator connector connected to the strain gauge by wires and the reading were taken at each increment of loading.

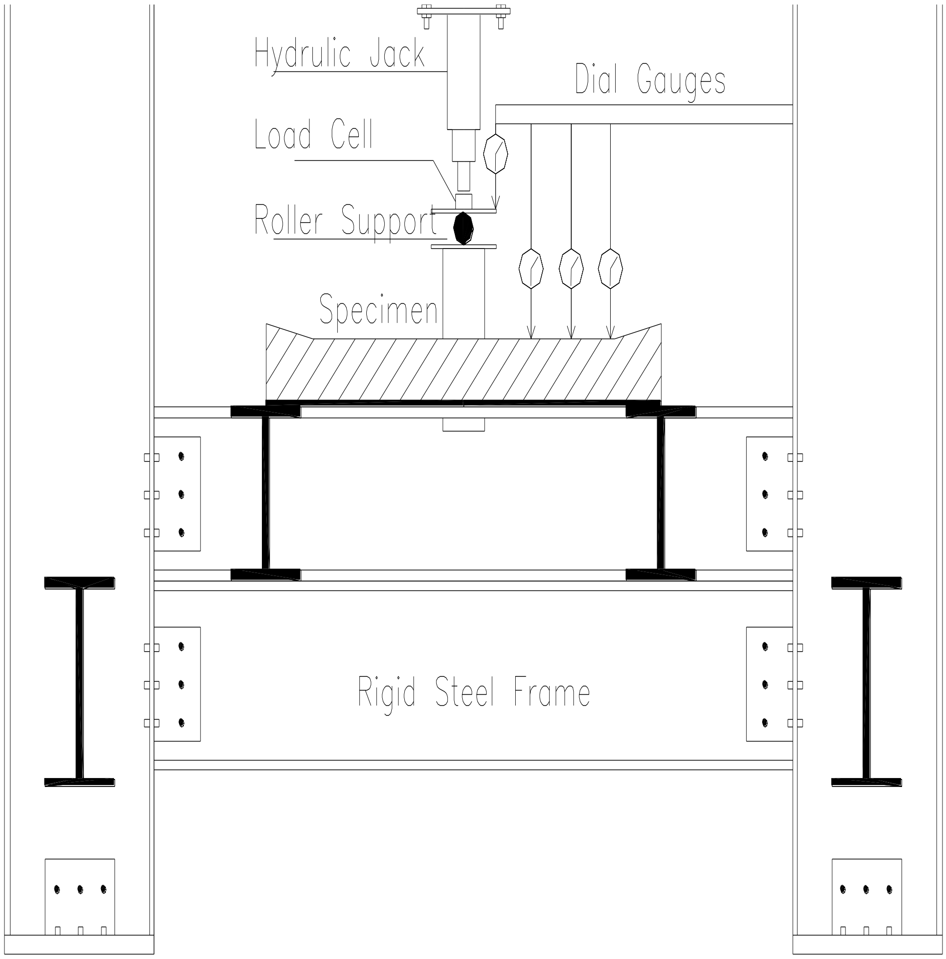

Fig 1: Test set up

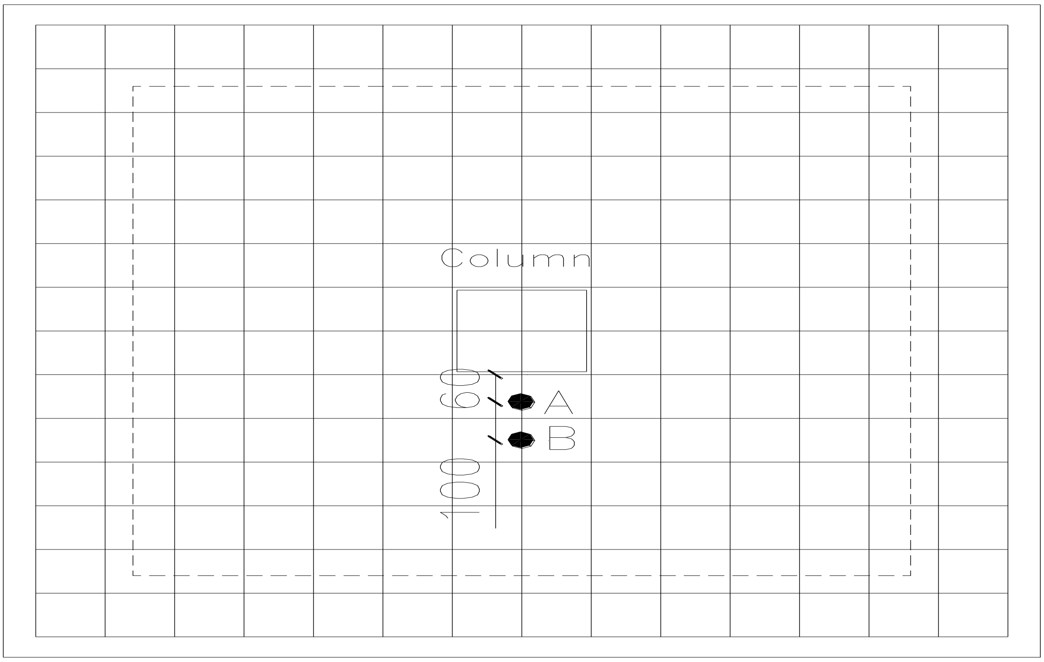

Fig.2. Strain gauge location

6. Test program

The parameters, which were studied experimentally in this research, Number of cables in specimens Prestressing force in slab. The details of test specimens are listed in table (1) and divided, as mentioned before to two groups. Group No 1, It consists of three prestressed flat slabs (P.F.S1, P.F.S2, P.F.S9) to study the effect of number of cables on behavior of slab , these specimens contain (3 , 4 , 5) cables respectively in two direction. The total prestressing force is constant for these entire specimens equal 3.82 kN in each direction. Group No 2, It consists of two prestressed flat slabs (P.F.S2, P.F.S6) to study the effect of prestressing force value on behavior of slab , it consist of four cables in each direction .The prestressing force for slab (P.F.S2) and (P.F.S6) are 3.82 kN, 5.00 kN respectively in each direction.

Table 1: Description of Test Specimens

Group No	Parameter Effect	Slab No	Number of cables	Presstrising Force kN	Top and bottom steel
1	Number of cables	P.F.S1	3	3.82	14 ϕ 6
		P.F.S2	4	3.82	14 ϕ 6
		P.F.S9	5	3.82	14 ϕ 6
2	Presstresing force	P.F.S2	4	3.82	14 ϕ 6
		P.F.S6	4	5.00	14 ϕ 6

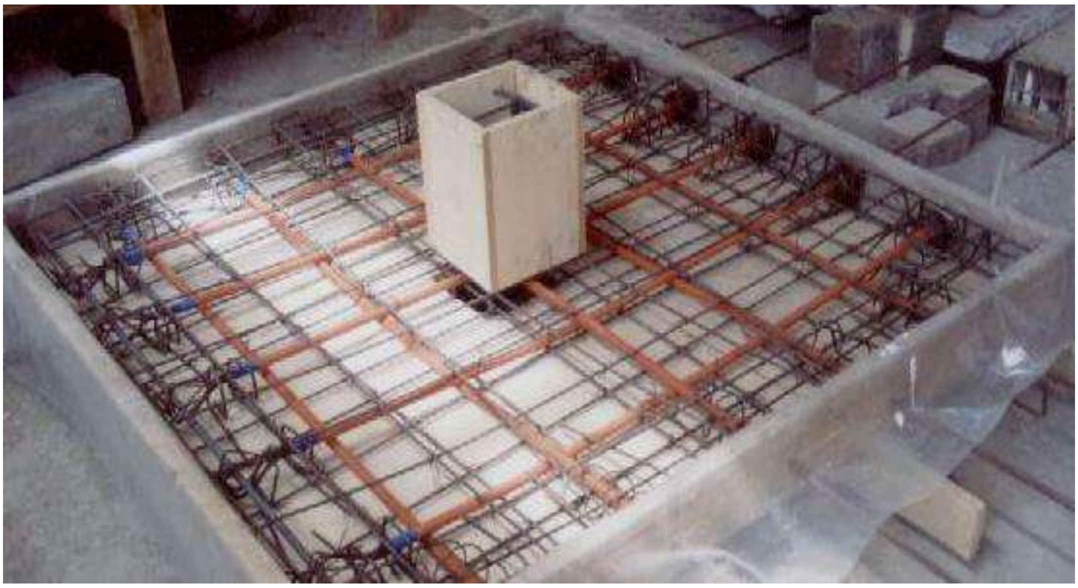

Fig.3: Over view of preparation for slabs

Fig.4. Details of reinforcement for all prestressed slabs

7. Test results and analysis

All specimens exhibited similar cracking behaviour throughout the test, and failed in a combination of flexure and shear with a final failure mode of "Punching shear ". After failure all specimens remained supported by the tendons, passing through the column and aided by the bonded reinforcement. Each specimen remained intact after failure, and did not completely collapse. Cracking and failure loads are tabulated in Table 2. The measured deflections at column face for all tested slabs are plotted in Fig. 5. The measured steel strains and strand strains are plotted in Figs. 6.

7.1 Strains

Steel Strain The profiles of the measured strains with loading are plotted in Figs .6. (a, c, e, g). Fig .6. (a) Shows the reinforcement strains for P.F.S.1. The first yield strain was recorded by S.G.B at load 120 kN and S.G.A did not reach the yield strain. From Fig 6.(a) it seems that the strains are generally low in the bars.

Fig.6. (c) shows the reinforcement strains for P.F.S.2, strain gauge S.G.A and S.G.B did not reach yield strain. Even at failure S.G.A and S.G.B recorded small strains compared to the yield. Fig.6. (e) shows the reinforcement strains for P.F.S.6. The first yield strain was recorded by S.G.A at load 80 kN followed by S.G.B approximately at 100 kN. Even at crack load S.G.A and S.G.B recorded very small strains compared to the yield strain and then the strain increase. It seems from Fig 6. (e) that the strains were generally high at failure.

Fig .6.(g) shows the reinforcement strains for P.F.S.9. An electrical problem occurred for strain gauge S.G.B during test, and the strain gauge S.G.A did not reach the yield strain, and record very small strains during test.

Tendon Strains, Strains in the unbonded tendon was measured, at location as shown before in Fig.2, the profiles of the measured strains with loading are plotted in Fig 7. (b, d, f, h)

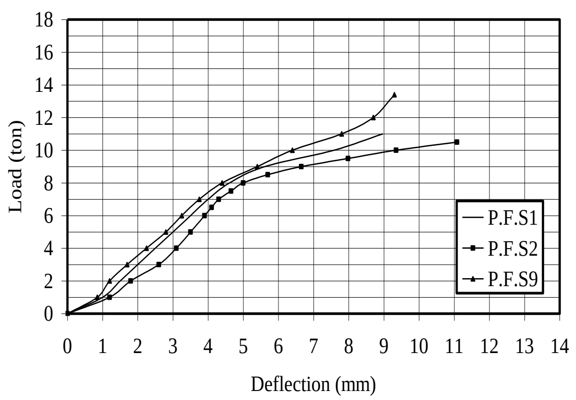

Fig.5. (a): Effect of Cables Number.

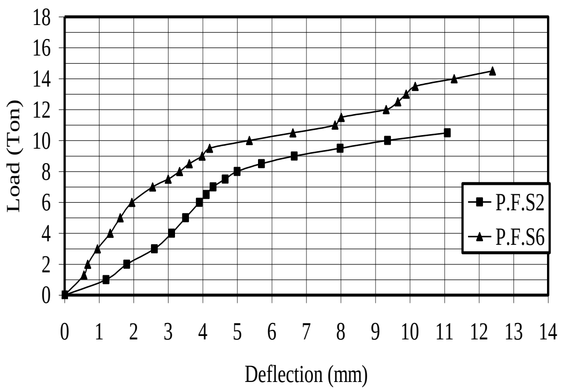

Fig.5. (b): Effect of Prestressing Force

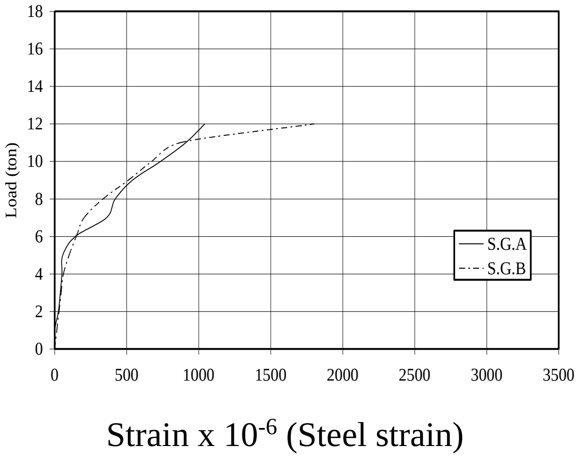

Fig.6. (a): Load strain curve for P.F.S.1

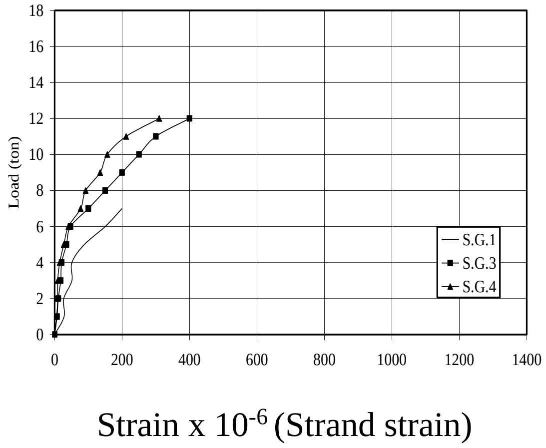

Fig.6. (b): Load strain curve for P.F.S.1

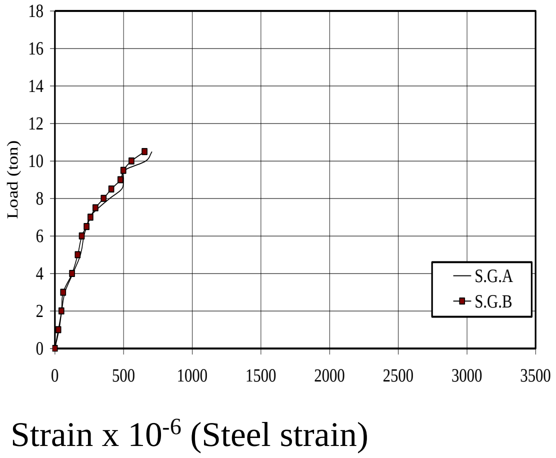

Fig.6. (c): Load strain curve for P.F.S.2

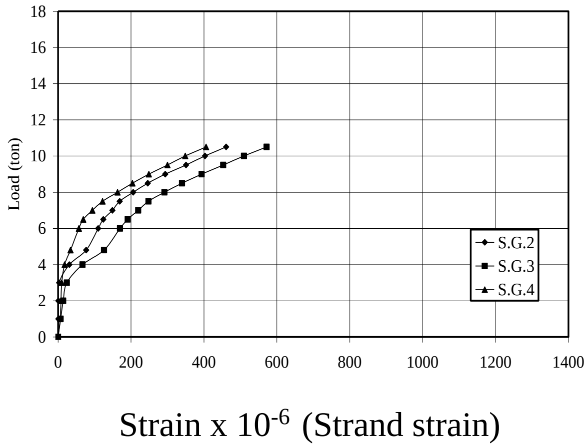

Fig.6. (d): Load strain curve for P.F.S.2

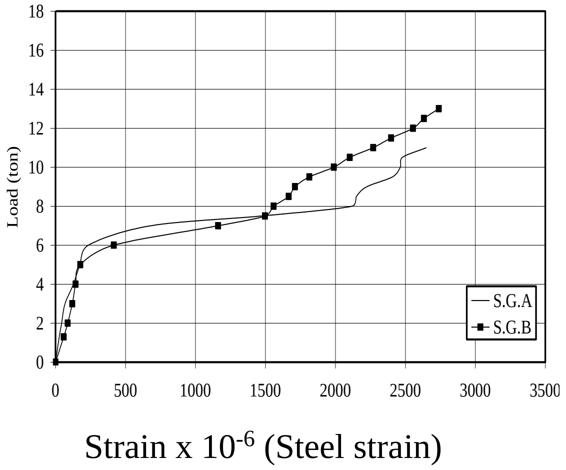

Fig.6. (e): Load strain curve for P.F.S.6 slab

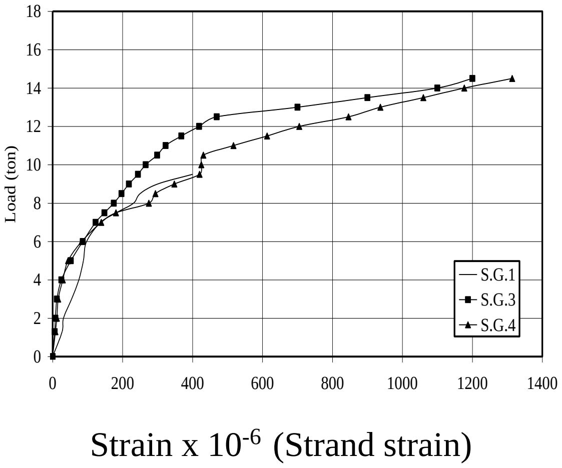

Fig.6. (f): Load strain curve for P.F.S.6

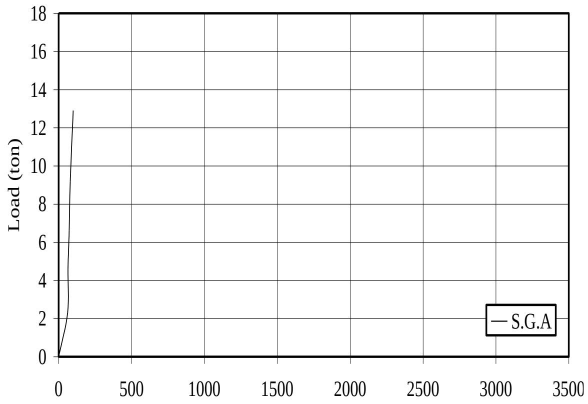

Fig.6. (g): Load strain curve for P.F.S.9

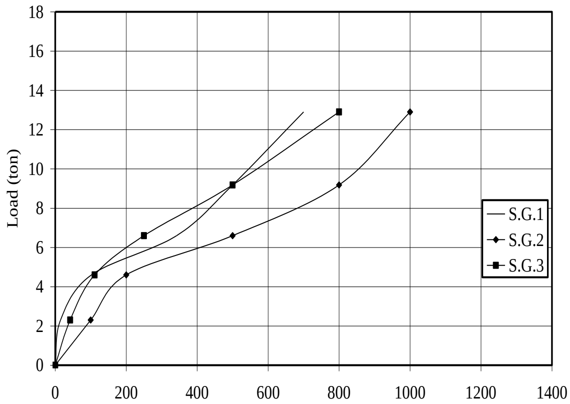

Fig.6. (h): Load strain curve for P.F.S.9

Table 2: Results of slab tests.

Group No	Slab No	P_{cr} At first crack (kN)	P_f Failure load (kN)	Δ_{cr} (mm)	Δ_{so} (mm)	Δ_s (mm)	Δ_{max} (mm)	Rigidity (P_{cr}/Δ_{cr})
1	P.F.S1	56.00	124.0	3.35	5.20	5.20	8.97	1.67
	P.F.S2	51.00	110.0	3.60	5.70	4.70	11.08	1.42
	P.F.S9	56.00	134.0	3.50	4.75	5.70	9.30	1.60
2	P.F.S2	51.00	110.0	3.60	5.70	4.70	11.08	1.42
	P.F.S6	60.00	146.0	1.95	3.70	5.90	12.39	3.07

7.2 Comparison between Experimental Results and Different Codes.

Table 3.a: Comparison between Failure and Predicted Shear Force of slabs of group (1)

Slab No	$V_{(test)}$ kN	ACI 318 Test/Code	BS 8110 Test/Code	Gardner Test/Code
P.F.S.1	124	1.26	1.18	1.21
P.F.S.2	110	1.07	1.02	1.01
P.F.S.9	134	1.29	1.23	1.26
Average		1.21	1.14	1.16

Table 3.b: Comparison between Failure and Predicted Shear Force of slabs of group (2)

Slab No	$V_{(test)}$ kN	ACI 318 Test/Code	BS 8110 Test/Code	Gardner Test/Code
P.F.S.2	110	1.07	1.02	1.01
P.F.S.6	146	1.47	1.41	1.33
Average		1.27	1.22	1.17

7.2.1 Comparison with respect to the Number of Cables.

The American Code ACI 318-02

For P.F.S.1, P.F.S.2 and P.F.S.9 (represents the effect of the number of cables), the ratios (V_{test}/V_{code}) are 1.26, 1.07 and 1.29 respectively. It seems like the code is conservative for slabs P.F.S.1 and P.F.S.9. Table 3.a shows the relationship between the number of cables and the ratios (V_{test}/V_{code}). Such conservatism is clearly evident.

For P.F.S.2 ratios (V_{test}/V_{code}) it is less conservative because no tendon pass through the column, the ACI provision state that it must be two tendons passing through the column in two directions.

The British standard (BS 8110-97)

For P.F.S.1, P.F.S.2 and P.F.S.9 (represents the effect of the number of cables), the ratios (V_{test}/V_{code}) are 1.18, 1.02 and 1.23 respectively. It seems like the code is conservative for slabs P.F.S.1 and P.F.S.9. Table 3.a shows the relationship between the number of cables and the ratios (V_{test}/V_{code}). Such conservatism is clearly evident.The degree of conservatism increase with increasing number of cables.

Gardner method

For P.F.S.1, P.F.S.2 and P.F.S.9 (represents the effect of the number of cables), the ratios (V_{test}/V_{code}) are 1.21, 1.01 and 1.26 respectively. It seems like the code is conservative for slabs P.F.S.1 and P.F.S.9.

7.2.2 Comparison with respect to the Prestressing Force.

The American Code ACI 318-02

For P.F.S.2 and P.F.S.6 (represents the effect of the Prestressing Force), the ratios (V_{test}/V_{code}) are 1.07 and 1.47 respectively. It seems like the code is conservative for slabs P.F.S.1 and P.F.S.6.

The degree of conservatism increase with increasing the total prestressing force, the punching shear provisions of ACI 318-02 for slabs with only precompression or precompression and bonded reinforcement, even ignoring the vertical component of the prestressing force are significantly less conservative than those for reinforced concrete slabs.

The British standard (BS 8110-97)

For P.F.S.2 and P.F.S.6 the ratios (V_{test}/V_{code}) are 1.02 and 1.41 respectively. It seems like the code is conservative for slabs P.F.S.6 Table 3.b shows the relationship between the number of cables and the ratios (V_{test}/V_{code}). Such conservatism is clearly evident, the degree of conservatism increase with increasing the prestressing force.

Gardner's method

For P.F.S.2 and P.F.S.6 the ratios (V_{test}/V_{code}) are 1.01 and 1.33 respectively. It seems like the code is conservative for slabs P.F.S.6. Table 3.b shows the relationship between the Total Prestressing Force and the ratios (V_{test}/V_{code}). Such conservatism is clearly evident. The degree of conservatism increase with increasing the prestressing force, in other words the provisions of the proposed method underestimate the effect of prestressing force.

8. CONCLUSION

Based on the analysis of experimental test results, the following conclusion can be drawn:

1- The flexural behaviour of post-tensioned concrete flat slabs in flexure is excellent with the slabs sustaining large loads before wide spread cracking takes place. Cracks are restricted to areas of high moment. However, the ultimate load capacity can be governed by punching shear failure.

2- Neglecting the vertical component of the prestressing force in calculations attributed to ACI 318-95, BS 8110-97 and the proposed method, will be conservative.

3- The punching shear provisions of ACI 318-02 for slabs with only precompression or precompression and bonded reinforcement, even ignoring the vertical component of the prestressing force are significantly less conservative than those for reinforced concrete slabs.

4- As many tendons in each direction as practical should pass through the column, with increasing number of tendons, an increases in the failure, crack load and improvement in the physical behavior of slabs.

5- Increasing the prestressing force lead to more confinement for slabs, and more enhancements in behavior and the punching shear capacity.

6- Increasing the prestressing force lead to an increase in the punching shear capacity of P.F.S.1 approximately by 32%.

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