BEHAVIOUR AND STRENGTH OF REINFORCED AND PRESTRESSED CONCRETE FLOOR SLAB-COLUMN CONNECTIONS

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ABSTRACT

This paper reports six tests of simply supported floor slabs 6 ft (1.8 m) square 6 in (15 cm) thick post-tensioned in the two orthogonal directions. The test slabs were intended to represent the region around intermediate column supports of prestressed flat plate structures. Prestressing was applied eccentrically in both directions by tendons of circular profiles using DYWIDAG deformed prestressed single-thread bar tendons of 15.1 mm diameter tendons, placed through 21 mm diameter corrugated metal conduits. Three slabs had bonded tendons by grouting their conduits and two were left unbonded. The sixth slab was ordinarily reinforced in the two directions by the same DYWIDAG straight bare bars at the same spacing layout of prestressed tendons and with the same end anchorage conditions at the slab simply supported edges. In one of the prestressed bonded slabs the tendons were symmetrically concentrated in both directions under the column region as shown in Fig.(1). The concrete compressive strength ranges between 5250 psi (367.5 k/cm^2) and 6360 psi (445.2 kg/cm^2). The slabs were tested to failure under centrally applied concentrated load through a square column stub, monolithicaly cast with the slab. Final failure of all the slabs were by punching of the column stub through the slab. The validity of the well recogized methods of analysis were compared and checked against the test results. Based on the tested slab results, other test results and the analytical studies, design recommendations were drawn.

INTRODUCTION

While sizable body of knowledge has been developed dealing with the flexural strength of prestressed concrete flat plates or slab bridges ,supported on columns or individual bearings, shear strength at thier connections with supports has received very little attentions in the past. Also current codes and standards give little or no guidance on the assessment of ultimate shear strength in such circumstances [1,2,3]. The connections between the floor or bridge slab and the columns in prestressed flat plate structures and prestressed slab bridges were generally the most critical part, because it is a region where large moments and shear forces are concentrated.

Research into the behaviour and strength of prestressed slabs has been mainly on unbonded slabs with corners free to lift [4]. The tests reported here were designed to provide experimental data on the shearing resistance of solid slabs orthogonaly prestressed in two directions by bonded and unbonded tendons with varying amount of prestress and different concentration of the unbonded tendons in the column vicinity.

Recommendations on punching capacity and design methods for the prestressed bridge and/or floor slab-column connections were concluded.

TEST PROGRAMME

Six test specimens were designed to simulate the part of a flat plate, in the actual structure, bounded by the lines of contraflexure in the column vicinity in a multi-panel floor system. Therefore, the specimens were supposed to represent the region of negative bending moment around an interior column . Figure(1) and Table(1) give the dimension of the test specimens , while Fig.(1) shows also the general arrangements of the post-tensioned prestress cables layouts.

The load on the connection which in practice consists of a uniformly distributed load on the deck slab was simulated

Table 1. Description of test specimens, notations and test results.

Specimen No.	Type of reinf.	f_c psi	f_{sp} psi	F_e kip	f_{pc} psi	f_{ps} psi	$S_{max.}$ in.	V_{test} kips
R	R	5249	544	-	-	-	0.658	85.40
P_{u3}	PU	6090	561	16.2	300	71299	0.806	113.98
P_{u5}	PU	6257	594	27.0	500	109076	0.859	125.88
P_{cu3}	PU	6120	591	16.2	300	71314	0.815	100.05
P_{b3}	Pb	6157	586	16.2	300	143083	1.155	117.63
P_{b5}	Pb	6356	624	27.0	500	143357	0.976	124.35

1-For conversion: 1 in.=25.4 mm , 1 kip=1000 lb=453.6 kg ,0.07 kg/cm^2 =1 psi
2-For all specimens: d_1=4.875 in. , d_2=4.056 in. , $d=d_1+d_2$=4.47 in.
(at center of slab), r=column's side width=10 in. , r/d=2.24 except for specimen R where d=4.5 in.[fig.(3)]
$\rho_p=A_{ps}/sd$=.69% , f_{py}=134400psi(9408 kg/cm^2)at 0.2% offset ,and f_{pu}=158200 psi(11074 kg/cm^2)

ii- Notations:

$A_{ps\phi}$= area of one prestressed tendon , A_s=area of ordinary reinforcement
d = average effective depth of prestressed steel, at center of slab
f'_c = compressive cylinder strength of concrete
f_{cp}= split cylinder strength of concrete
F_e = effective prestress force per cable
f_{pc} : average effective concrete prestress after losses $=F_e/st$
s = cable spacing and t= slab thickness
f_{py} = specified yield strength of prestress tendons
f_{pu} = specified tensile strength of prestress tendons
ρ_p = ratio of prestressed reinforcement or steel reinforcement in this paper
f_{ps} = stress in prestress at nominal strength in accordance to ACI 1989 Building Code requirements Eqs.18.3 and 18.4.
s_{max}= maximum deflection obtained before failure of specimen

Type of reinforcement notations : R Ordinary Reinforcement, P_u Prestressed with un bonded tendons , P_b prestressed with bonded tendons .

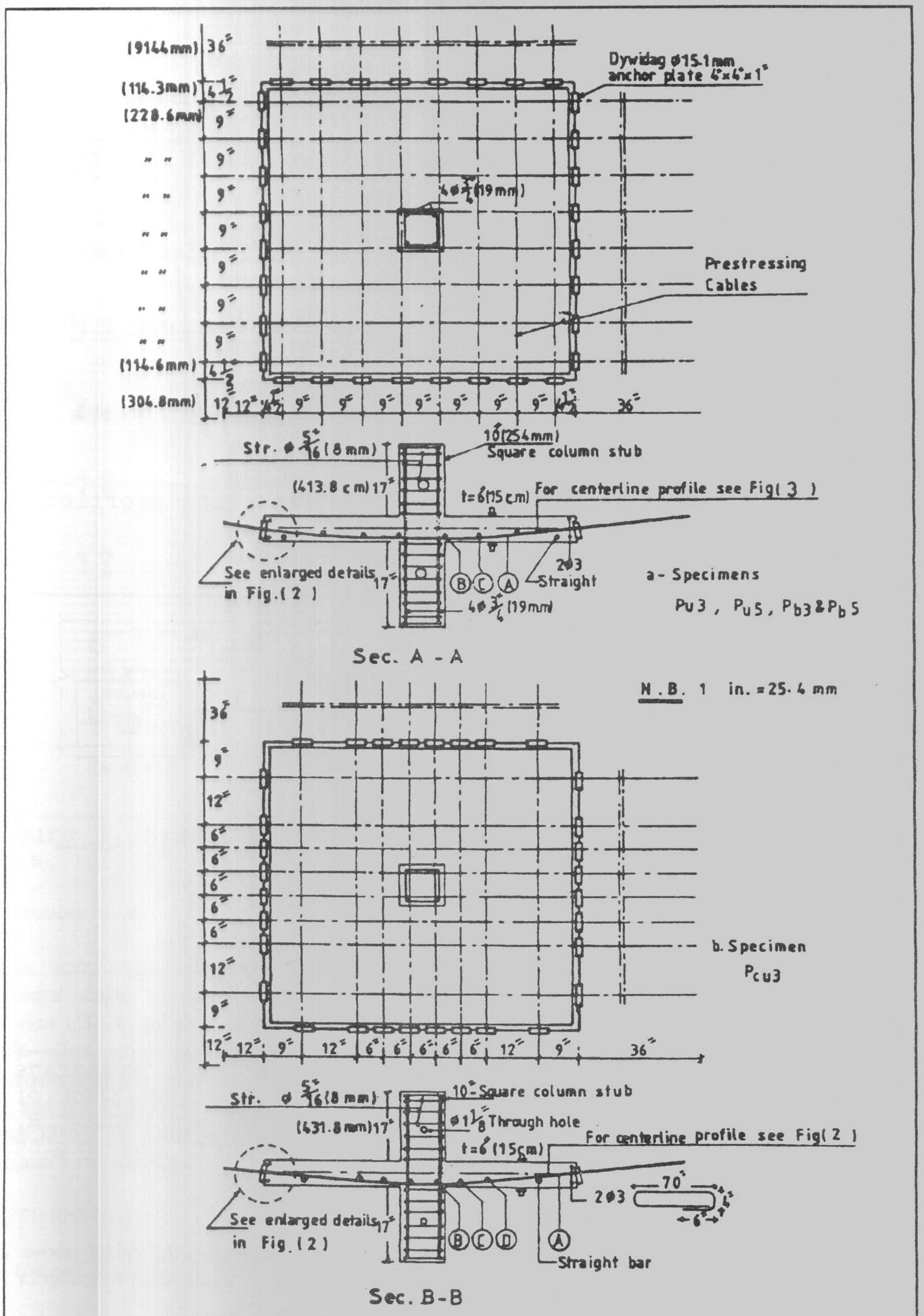


Figure 1. Test specimens dimensions and cables layout.

by supporting the slab along its four sides with the corners prevented from lifting up. The test specimens were tested under axial load through the column stub, monotonically loaded in increments till failure .

Three of the specimens were provided with unbonded DYWIDAG single thread bar tendons placed in corrugated metal ducts at 33% or 60% of the breaking strength according to the amount of the average concrete prestress, f_{pc}, required. Two of these specimens had varied effective mean prestressing stress after losses, f_{pc}. Specimens P_{u3} had f_{pc}=300 psi (21 kg/cm^2) and P_{u5} had f_{pc}=500 psi (35 kg/cm^2). The third Specimen P_{cu3} had over all mean effective concrete prestress stress after losses, f_{pc}=300 psi (21 kg/cm^2), but it had 75 % of the total tendons, in each direction, concentrated under the column stub, within 42 % of the total slab width under the column stub , see Fig.(1).

Two other specimens P_{b3} with f_{pc}=300 psi (21 kg/cm^2), and Pb5 with f_{pc}=500 psi (35 kg/cm^2) had bonded tendons, while the sixth test Specimen R is reinforced with unstressed DYWIDAG straight bare bars of the same diameter and general arrangement layout as that of P_{u3} , P_{u5} ,P_{b3} and P_{b5} and with similer end anchorage, Fig.(4-a). Included also in the test programme, as indirect parameter, is the concrete strength. The concrete compressive strength ranges between 5250 psi (367.5 kg/cm^2) and 6360 psi (445.2 kg/cm^2).

The tendons profiles for P_{u3} , P_{u5} , P_{cu3}, P_{b3} and P_{b5} were circular arcs with variable eccentricities, see Fig.(3). The lower points of the cables had maximum eccentricity, emax, from the middle plane of the slab which is the chosen level for the end anchorage of the cables =1.875 in (47.6 mm).

The test specimens were grouped into four groups as shown in Table(2) to clarify the effect, on the behaviour and shear strength, of the major studied parameters; mainly:

1- The effect of prestressing against that of ordinary reinforcement on the behaviour and strength. This parameter was studied in group I.
2- The amount of prestressing average effective stress, f_{pc} ,on the behaviour and strength. This parameter was studies for both unbonded and bonded prestressed connection as presented by group II and III respectively .

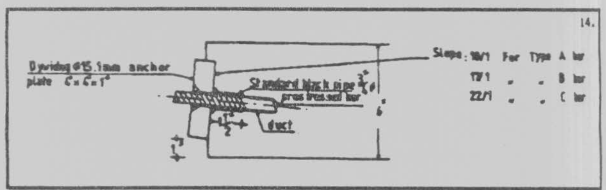


Figure 2. Detail At An Anchor Plate.

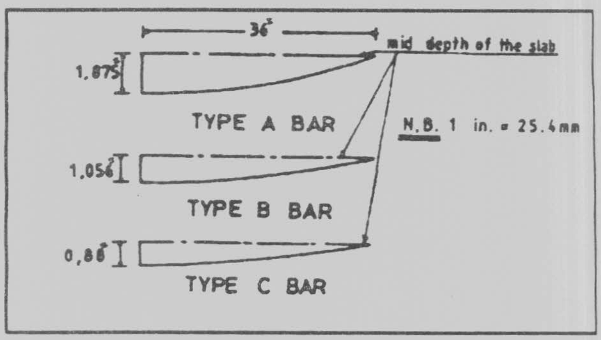


Figure 3. Centerline profiles of prestressed bars.

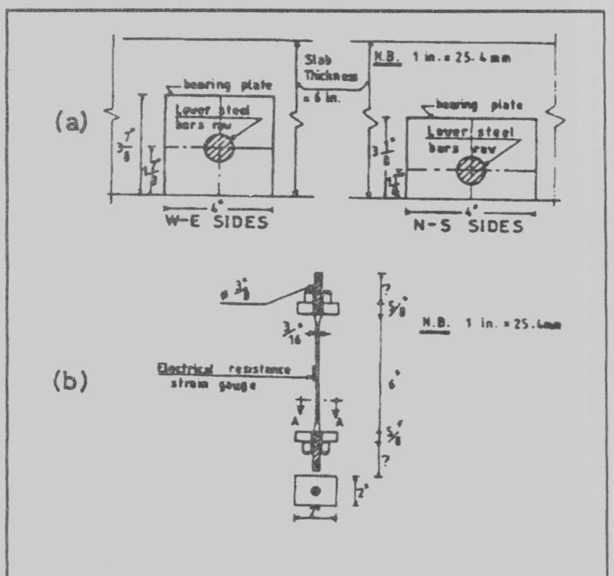


Figure 4 (a). End anchorage bearing plates and position of reinforcement for connection R, (b) Crack detector measurring device.

3- The effect of concentration of cables above the column head zone, this was studied by group IV.

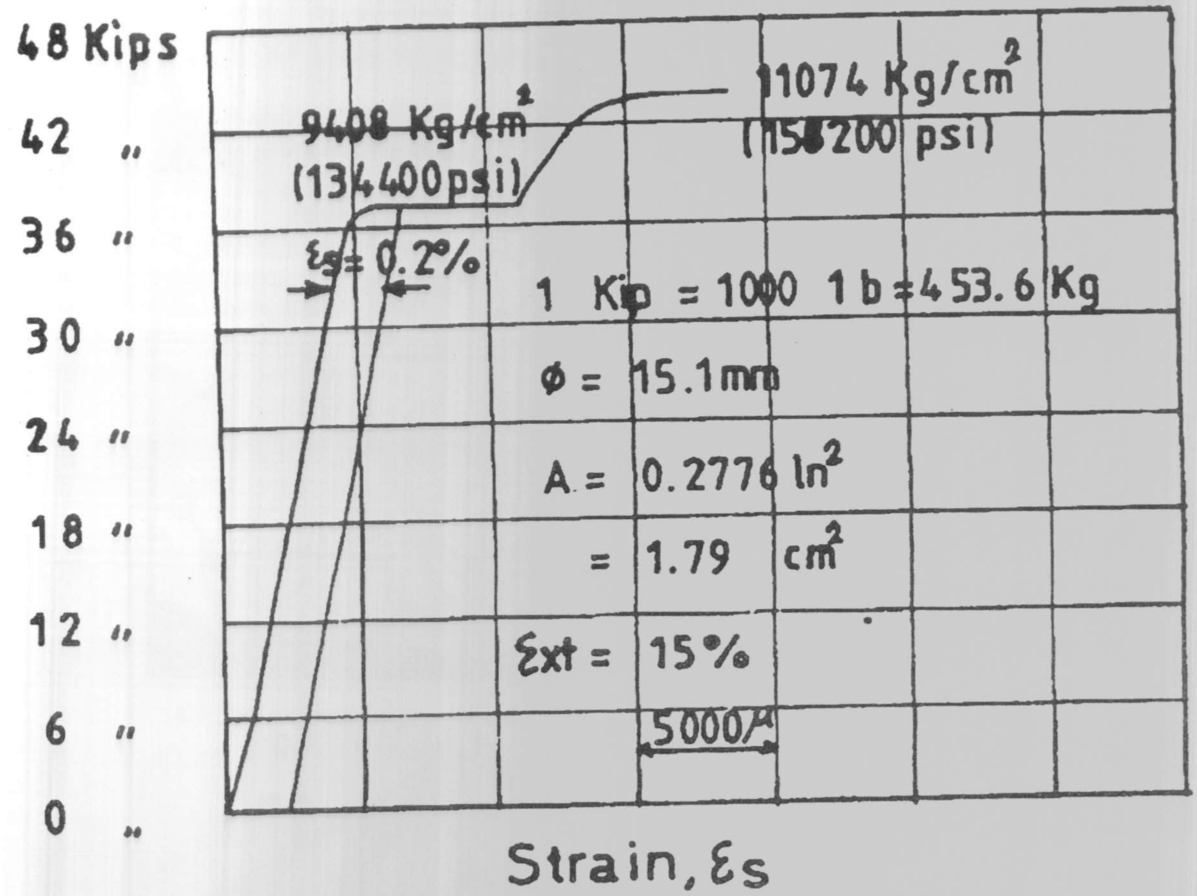


Figure 5. Stress strain relationships for dywidag prestressing steel bars.

Prestressing steel

The stress-strain relationships for the DYWIDAG threaded single bar prestressing ducts are shown in Figuure (5). As its yield point is not well defined, the 0.2% offset proof stress was chosen to represent the prestress tendon's yield stress. The specified yield strength, f_{py}, and the specified tensile strength of the tendons, f_{pu}, were 134400 psi (9408 kg/mm^2) and 158200 psi (11074 kg/mm^2) respectively.

TEST RESULTS

1- General behaviour of test slabs and mode of failure.

Visible flexural cracks first appeared on the tension side of slabs at between 47% and 52% of their ultimate loads for unbonded test specimens and between 42% and 48% for bonded test specimens. The visible flexural crack for test specimen, P_{cu3}, with tendons concentrated under the central column stub, appeared at 40% of the ultimate load and that for the connection R which had Ordinary Reinforced concrete floor slab appeared at 35% of its ultimate load.

The cracks generally started around the floor slab-column interface lines and at lines radiated from these intersection lines towards floor slab edges close to corners. With the increase in loading the cracks gradually progressed further towards the slab edges and those emanating at column corners intercept the edges of the slab at short distances from their corners. There were no other cracks parallel to floor slab edges or circomferencial around the periphery of the loading stubs or any where in

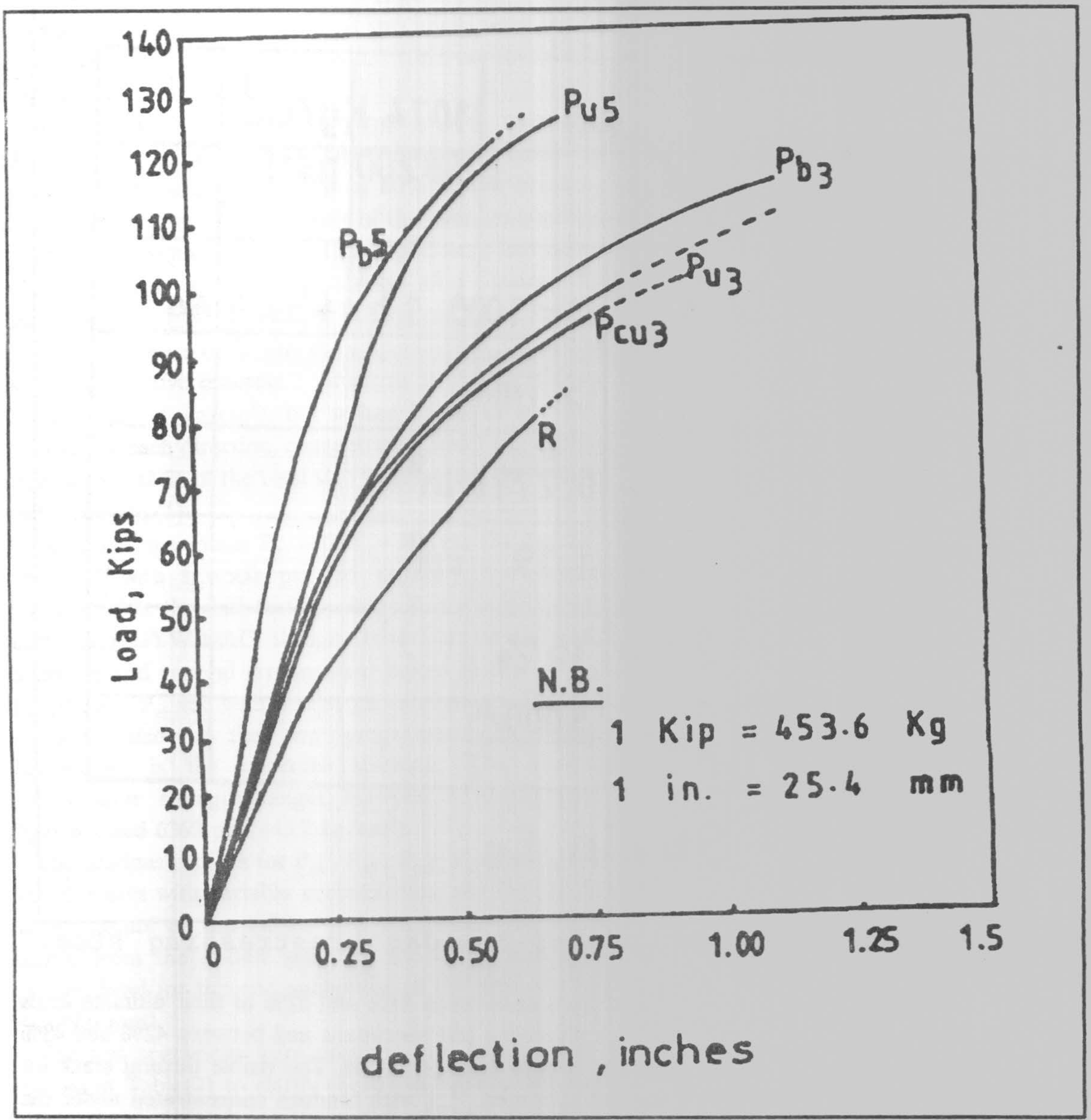


Figure 6. Load-central column stub deflection relationships.

the prestressed floor slabs, see Fig.(7-a) through (7-b). In the connection R with Ordinary Reinforced concrete floor slab, beside the cracks formed on the tension side along the column perimeter and the radial cracks mentioned before, cercomferencial cracks appeared at 47% and 50% of its ultimate load. In general, the intensity of cracks were great in case of the connection having an Ordinary Reinforced floor slab, Specimen R Fig.(7-h), compared to the connections having Unbonded or Bonded prestressed floor slabs.

Although most of the above major cracks followed the general expected yield-line crack pattern, the full development of yield-line mechanisms did not fully develope before failure of any of the connections took place. The failure of all the connections were reached when the column stub punched through the floor slabs, the punching displacement (drop out) was recorded to be in the range of 50 mm or a bit more. This displacement can be noticed from the marked measured displacement on the plain scale fixed to the column stub at the time of preparation of Fig.(7-e) for demonstration. Figure(7-e) shows also the clear cut of the punched periphery at the

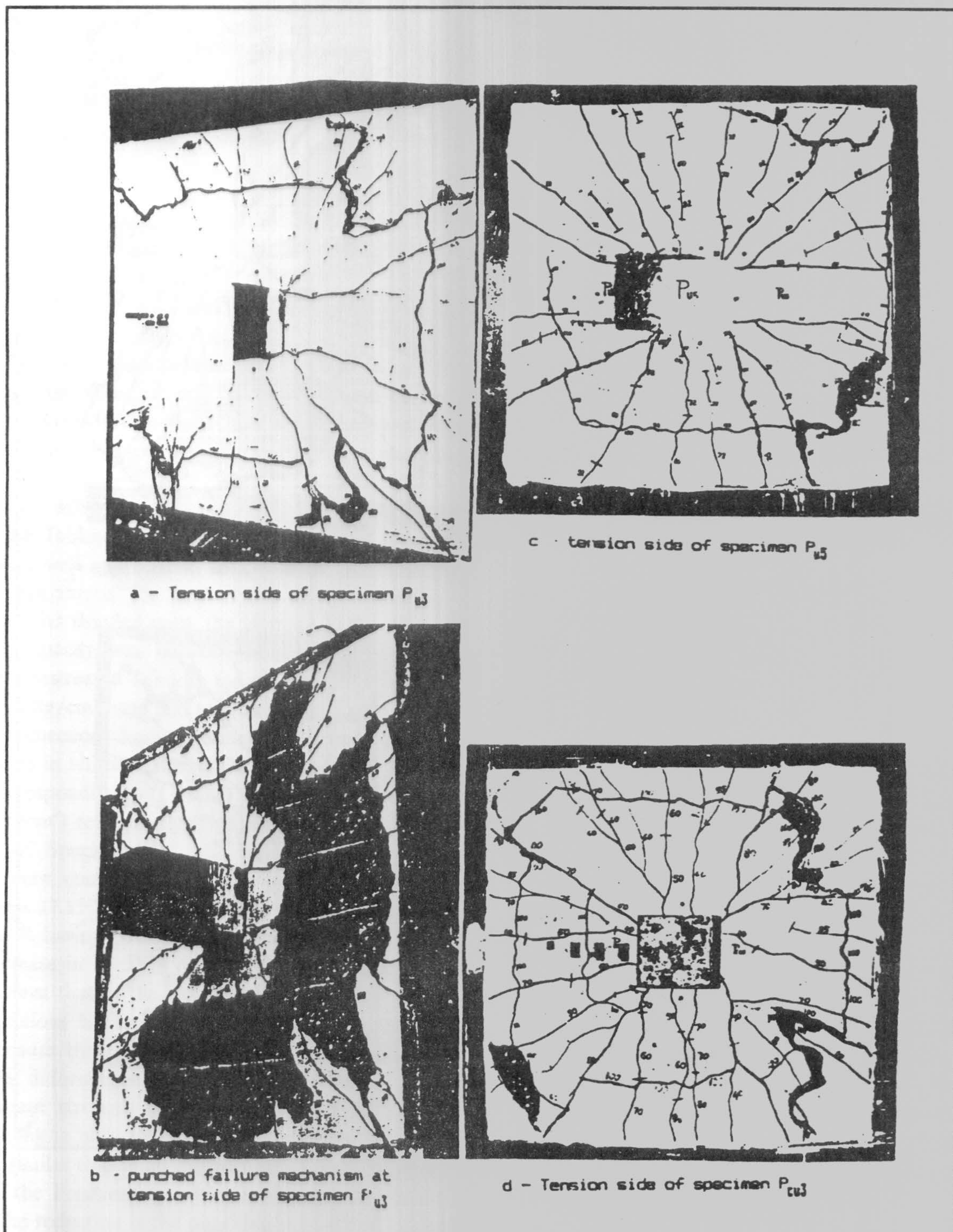
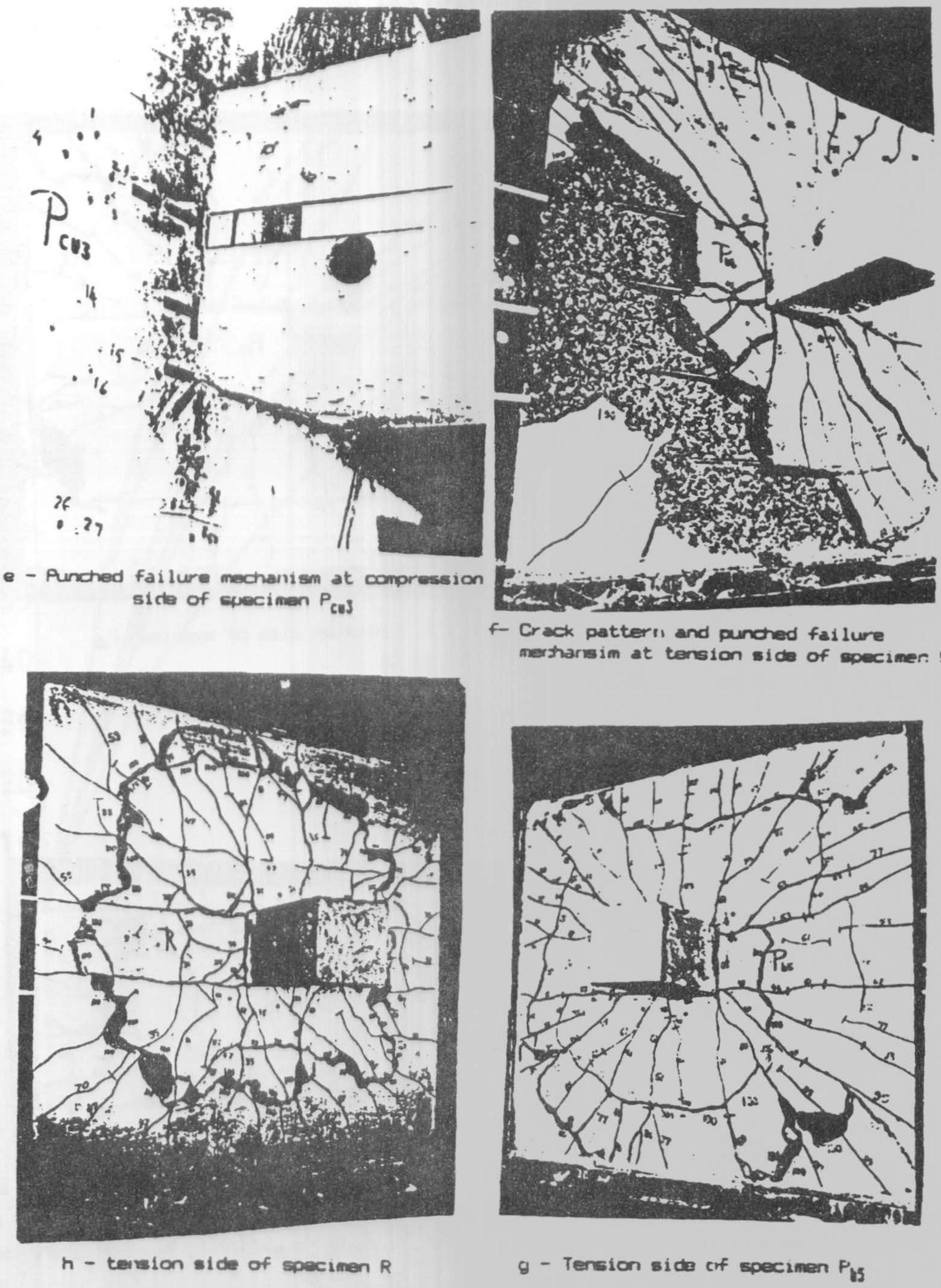


Figure 7. General crack pattern and failure mechanism surfaces after failure of test specimens.



e - Punched failure mechanism at compression side of specimen P_{cu3}

f- Crack pattern and punched failure merhansim at tension side of specimen P[illegible]

h - tension side of specimen R

g - Tension side of specimen P_{b5}

Figure 7. Continues

the immediate floor slab-column intersection lines which was common for all the test specimens included in this investigation. The punched through truncated pyramid for Specimens P_{u3} and P_{b3} are shown in Figs.(7-b) and (7-f) respectively. The failure surfaces were inclined at an angle about 30 to 40 of the floor surface.

Starting of Inclined diagonal cracks was recorded by the crack detectors Fig.(4-b). The results recorded for all the test specimens were plotted in Fig.(11). The figure shows that the start of Inclined cracks in the specimen's slabs were at between 65% and 80% of their ultimate load. The final failure load were by punching which was attained at much higher loads due to the effect of either the Ordinary or Prestressed reinforcement after the formation of the web(inclined)-shear failure mechanism.

2- Failure Loads:

All the test specimens failed in shear, Fig. (7), by punching of a plug of concrete in the form of a truncated pyramid. For the unbonded and bonded floor slab-column connections, it has been observed that cracking is restricted to the column perimeter and a few radial lines emanating from there on and from the column corners. This pattern takes flexure-shear cracking unlikely and web-shear cracking the most probable cause of a punching failure.

The factors affecting the punching resistance were very distinct, see Tables(2) and (3). The influence of prestress can be very well appreciated directly from the results of the test specimens for both types of prestressing , Unbonded and Bonded ones. the increase in the ultimate punching capacity was between 33.4% and 47.4% for Unboded Prestressed Tendons for average prestress,f_{pc} of 300 psi (21 kg/cm^2) and 500 psi (35 kg/cm^2) respectively. For the connections having Bonded Prestressed Tendons the increase in ultimate strength was between 37.8% and 45.6% corresponding to f_{pc} of 300 psi (21 kg/cm^2) and 500 psi (35 kg/cm^2) respectively. The increase in the ultimate strength of Specimen P_{cu3}, in which the prestressing tendons were concentrated in the column vicinity, see Fig.(1), was 17.15% compared to the ultimate strength of Specimen R having Ordinarily Reinforced Floor Slab.

The increase in strength due to the use of the bonded cables against that of the unbonded ones for the case of the connections having small average prestress was very small, because the specimens failed by punching before much total deformation occurred in either cases.

The ultimate strength of the connection P_{cu3} is much smaller than the ultimate strength of Specimen P_{u3}, which is exactly similar to it in all respects except for the general layout of the Prestressing cables in the column vicinity, Fig.(1). The reduction in the punching strength of P_{cu3} was due to the location of the Prestressed Tendons close to the ciritical shear plane, thus reducing the shear strength in a manner similar to a poorly located ducts[8].

3. Deflections:

Typical load deflection curves for all the test specimens are shown in Fig. (6). They did not show distinctive plateaus, especially for those specimens provided with high average concrete prestress value, f_{pc}, whether produced by unboded or bonded tendons. The maximum deflection attained in each slab was influenced by the amount of the average concrete prestress, f_{pc}, and the type of prestressing cables, unbonded or bonded ones, see Table(1).

The stiffness of the floor slab-column connections were shown to increase very rapidly as the average f_{pc} value increases. Small difference was found to occur for bonded connection compared to those provided with unbonded tendons. None prestressed floor showed the smallest stiffness as compared to prestressed ones even at a small level of stresses at all stages of loading. The stiffness of specimens prestressed with bonded cables is slightly greater than those with unboded cables.

Although the floor slab-column connections failed in shear at the column region. The prestressing of the connections at the column region did increase thier shear strength and stiffness, see Fig.(6). There was no increase in the ultimate capacity nor in the stiffness of the Specimen P_{cu3} compared to the Specimen P_{u3}, which have the same average prestress, f_{pc}, across the slab width.

4- Steel strain :

The load-increase in strain curves of the prestress steel,Fig.(8), usually followed the general shape of the load deflection curves of the corresponding slabs. They also did not display distinctive yield plateaus. The start of yielding of the prestress steel is clearly demostrated for Spcimens P_{u3} , P_{u5}, P_{cu3} , and P_{b3}. The strain gauges placed on the prestressed bars of P_{b5} were damaged during the grouting process and therefore they were not presented. The start of yielding of the prestressed steel before failure was markedly detected, following the nature of the load-deflection relationship for P_{b5} , Fig.(6) and Fig.(11).

The load-strain relationship of Connection R,the reinforced concrete one, Fig.(8), similar to its load-deflection relationship, Fig.(6), did not indicate the start of yield in this specimen. Figures(8) and (9) demonstrate the start of yield of the prestress steel close to the column region at higher loads, in all the others. Yielding was very prominent before complete failure of the connection by

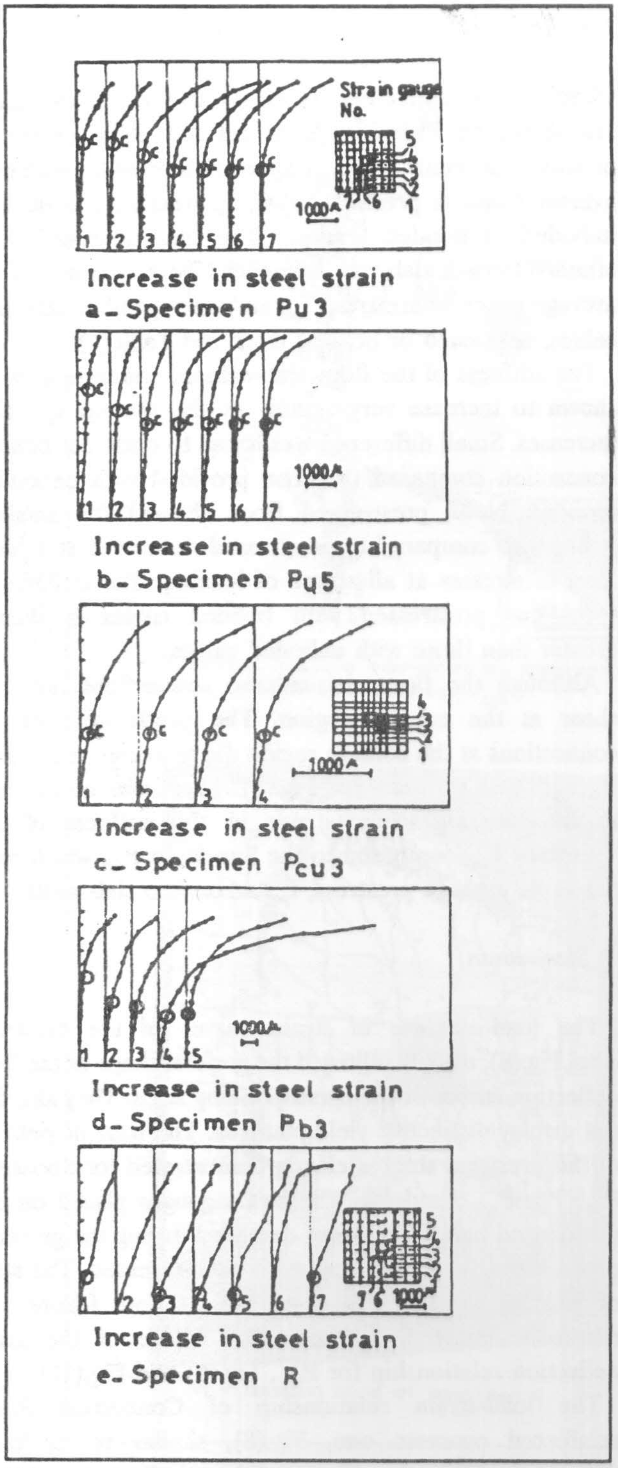


Figure 8. Load-increase in steel strain curves for prestressing steel.

punching of the column-stub through the floor slab, see Figs.(7) and (8).

Figure(9) demonstrates that the prestress forces in the tendons increased appreciably at higher loads before failure, especially those close to columns.

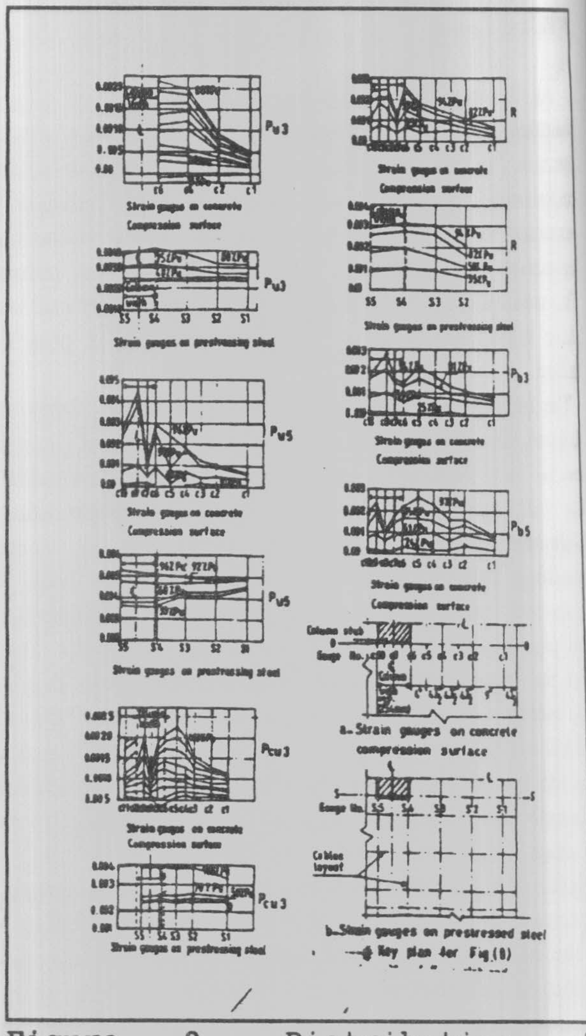


Figure 9. Distribution of concrete strain on compresion side of floor slab and distribution of prestressing steel bars strain at different load levels.

5- Concrete Strains:

The concrete strain, on the compression side across aline O-O Figure (9-a), as measured by Electric Strain Gauges in the immediate vicinity of columns and by mechanical demec gauges else where along the line, were plotted in Figure (9) at different stages of loading. This figure shows concentration of strains at about distance d from the column corner then followed by a decrease in strains as the distance gets away from it. Also the concrete strain as measured along centerline of the specimen on the concrete compression side between the column face and the floor slab edges were plotted in Fig.(10) for most of the specimens at different stages of loading. It also shows a gradual decrease of the strain as the distance gets away from column face. The maximum values of concrete strain measured close to the floor slab-column interfaces are of the order reached mostly at shear failure.

ANALYTICAL STUDY

The strength of the test specimens were calculated using the well recognized empirical equations developed for reinforced concrete floor slab-column connections [9] and those equations that were developed for prestressed floor slab ones([5,6,7]).

1- Moe's Equation:

This equation was developed by Moe[9]. It forms the basis for the ACI 1989 Building Code Requirement[5] as it was based on an extensive analytical and experimental study by Moe.

$$V_u = \frac{15(1-0.075r/d)}{1+\dfrac{5.25bd\sqrt{f_c'}}{V_{flex}}} \qquad (1)$$

where, V_u = ultimate shear force.

r = side length , and other terms are defined in Table(2).

For the case of a simply supported square slab of side length a and loaded through a square column with the corner free to lift, the ultimate flexural strength is given by the yield-line theory is:

$$V_{flex} = 8\, m_u \left[\left\{ \frac{1}{1-(r/a)} \right\} - (3-2\sqrt{2}) \right] \qquad (2)$$

where m_u = ultimate moment capacity per unit length of an isotropic floor slab

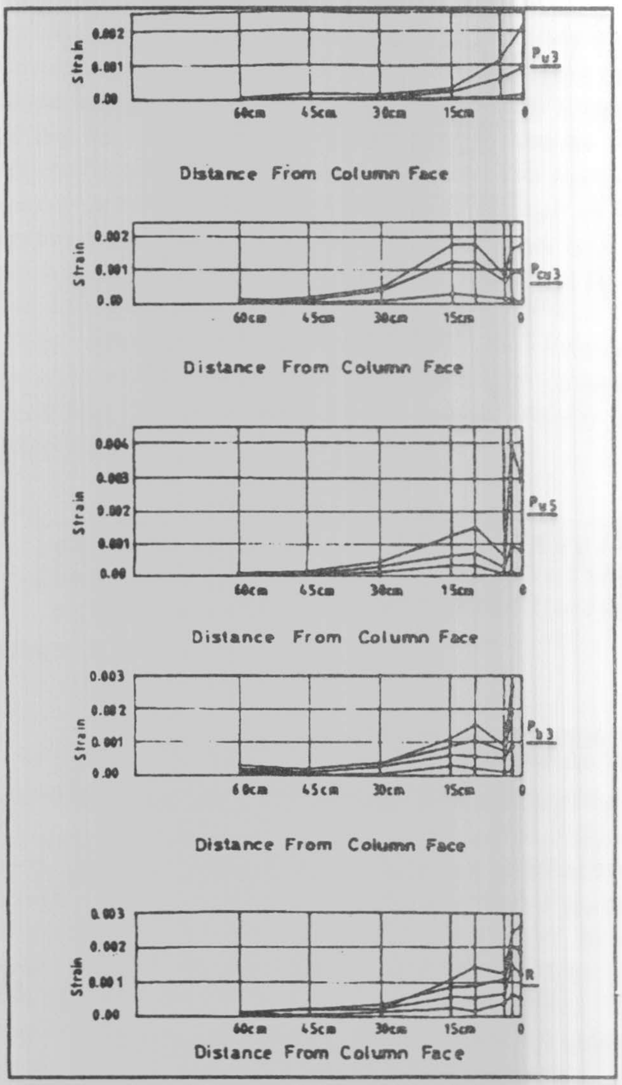


Figure 10. Distribution of concrete strain along centerline of test specimen.

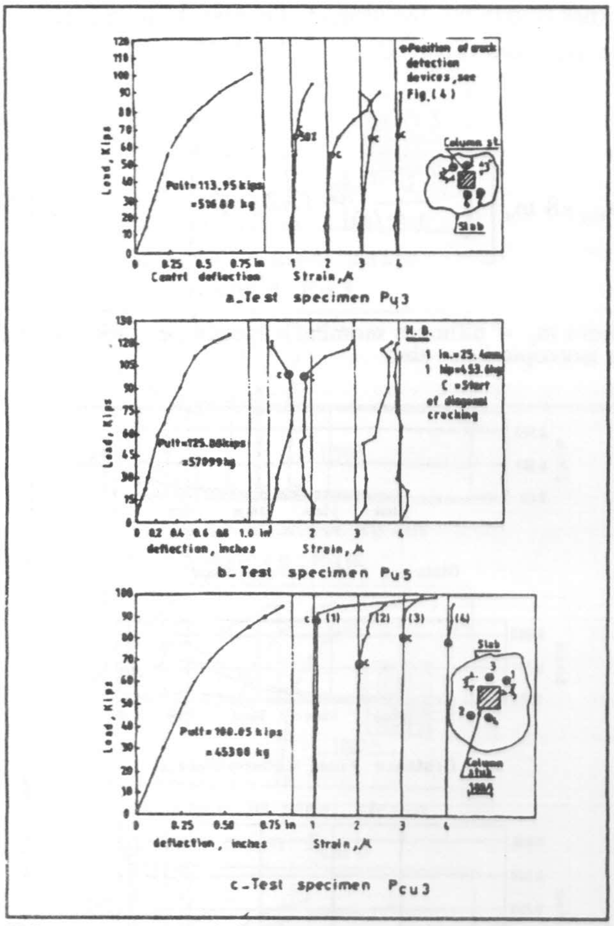
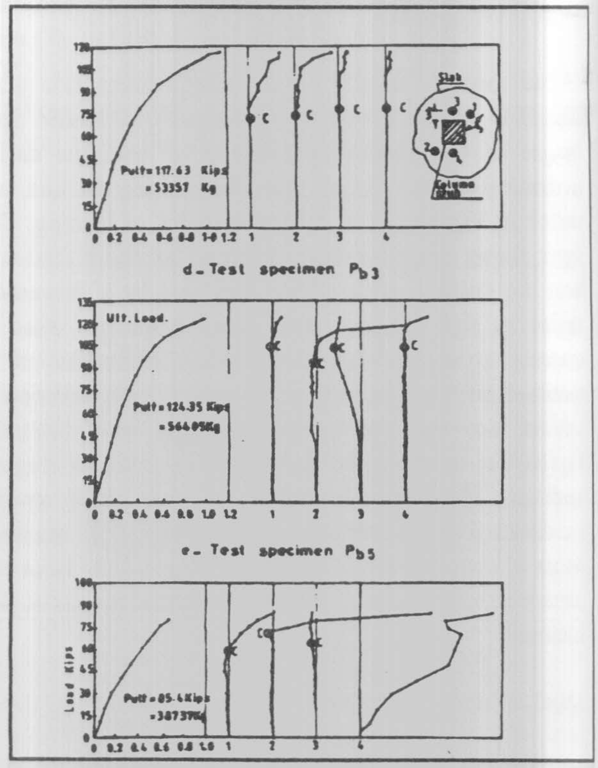


Figure 11. Load-central deflection and load crack detectorrs strain relationships.



f-Test specimen R

Figure 11. Continues.

2- Scordelis, Lin, and May equation:

The equation were developed by Scordelis et.al.[6] for prestressed floor slab-column connection. This equation was based on the results they obtained from tests of 15 normal weight prestressed floor slabs.

$$V_u = (0.175 - 0.0000242\, f'_c + 0.00002\, F_e /s)\, b\, d\, f_c' \quad [3]$$

Where F_e = effective prestress force per cable,lb.
s = cable spacing, in.

3- Grow and Vanderbilt equation:

This equation was introduced by Grow and Vanderbilt[7] as a result of their experimental study on tests to failure of the post-tensioned specimens constructed using an expanded shale aggregate.

$$V_u = (360 + 0.30\, f_{pc})\, bd \quad [4]$$

4- Building Code method's equation[5]:

i- Two-way prestressed slabs.

$$V_c = (\beta_p \sqrt{f_c'} + 0.3\, f_{pc})\, b_o\, d + V_p \quad [5]$$

Where,

b_o = perimeter of pseudocritical section located at a distance d/2 from column faces = 4(r + d).

V_c = nominal shear strength provided by concrete.

V_p = vertical component of effective prestress forces crossing the critical section.

β_p is the smallest of 3.5 or (α_s d/b + 1.5), α_s is 40 for interior column, 30 for edge column and 20 for corner columns.

Design values for f_c' and f_{pc} are restricted due to limited test data available for higher values as follows:

(a) f_c' in equation (5) shall not be taken greater than 5000 psi (350 kg/ cm^2), and

(b) f_{pc} in each direction shall not be taken less than 125 psi (8.75 kg/cm^2), nor be taken greater than 500 psi (35 kg/cm^2).

ii- *For non prestressed slabs:*

V_c shall be the smallest of:

$$V_c = \left(2 + \frac{4}{\beta_c}\right) \sqrt{f_c'}\, b_o\, d \qquad [6\text{-}a]$$

Where β_c is the ratio of long to short side of the column.

$$V_c = \left(\frac{\alpha_s\, d}{b_o} + 2\right) \sqrt{f_c'}\, b_o\, d \qquad [6\text{-}b]$$

wheree α_s is 40 for interior columns, 30 for edge columns, 20 for corner columns, and

$$V_c = 4 \sqrt{f_c'}\, b_o\, d \qquad [6\text{-}c]$$

COMPARISON BETWEEN ANALYTICAL AND TEST RESULTS

The results obtained from the analytical studies were tabulated in Table(2) and compared with test results. As may be seen in the table the test strength of the prestressed floor slabs were reasonably well predicted by Eq.(1), which was developed for Ordinary Reinforced concrete floor slabs and footing. But its results was not the same for Specimen R without prestress floor slab. This may be due to the use of the very high strength prestress steel, which was not taken as a parameter in the data used for the development of Moe's equation. This produced high flexural shear capacity, V_{flex}, thus reduced the predicted shear strength of Specimen R, as computed by Eq.(1).

Equations (3) and (4) were developed for prestressed floor slabs made of normal or light weight aggregate. they both gave very conservative results for both unbonded or bonded prestressed specimens, especially those bonded ones. Both equations could not accommodate for the effect of the bonded prestress tendons on the strength and behaviour of the studied Specimens. They also will not be able to accommodate for the presence of additional Ordinary reinforcement that may be necessary in some cases to improve the behaviour of bridge and floor slabs at their connections with the supporting columns. Equation(4) does not also take into account the effect of the compressive strength of concrete on shear strength of the connections.

The ACI 1989 Building Code Eq.(5) gave good correlation with the test results, although still some what conservative. It does not provide for the effect of the type or the amount of the Ordinary steel on the shear strength of the floor slab connections, nor does it consider the effect of bonded tendons on their behaviour. This equation seems to be able to predict the shear strength of an Ordinary Reinforced floor slab , when no upper limit is put for the value of β_p and compute its value from β_p = (α_s d/b + 1.5).

The shear strength computed from the following equation which is suggested to be used for Ordinary reinforced floor slabs was incorporated in Table(3) for comparison.

$$V_c = \beta_r \sqrt{f_c'}\, b_o\, d \qquad [7]$$

Where $\beta_r = \alpha_s\, d/b + 1.5$

As shown in Table(2) the value V_{flex} calculated for the unbonded tendon's connections, by using the expression for f_{ps} as given by Eq.(18.4) of the ACI 1989 Building Code which has been mainly proposed for beams, were greatly below the test stress. This indicated that this expression, for beams, probably developed for much longer cables , were not applicable to short cables used in this test specimens. A study of the test results, along with approximate computation for cable elongation due to fiber strain, cracking and deflection of floor slab, showed that the f_{py} value may favorably approximate the value of f_{ps} for such test specimens. The calculation of V_{flex} using this specified value for f_{ps}, resulted in good correlation with the test results for both V_{flex} and the correspondingly shear strength computed by Eq.(1) as shown in Table(3).

Table 2. Comparison of V_{test} and V_{calc}.

Group No.	Specimen No.	Type of reinf.	V_{test} kips	Y.L.Theory		Moe		Scordelis, Lime & May		Grow and Vanderbilt		ACI 1989 Code	
				$V_{calc.}$ kips	$\frac{V_{test}}{V_{calc}}$	V_{calc} kips	$\frac{V_{test}}{V_{calc}}$	V_{calc} kips	$\frac{V_{test}}{V_{calc}}$	Vcalc kips	$\frac{V_{test}}{V_{calc}}$	V_{calc} kips	$\frac{V_{test}}{V_{calc}}$
S_I	R	R	85.4	131.44	0.65	106.57	0.80	-	-	-	-	51.82	1.64
	P_{u3}	P_u	113.98	74.14	1.54	87.61	1.30	69.28	1.65	80.46	1.42	87.32	1.31
	P_{b3}	P_b	117.63	141.46	0.83	115.17	1.02	68.25	1.72	80.46	1.46	87.32	1.35
	P_{u8}	P_u	125.88	110.64	1.13	105.65	1.19	93.51	1.35	91.19	1.37	102.85	1.22
	P_{b8}	P_b	124.35	142.16	0.87	116.58	1.07	92.26	1.35	91.19	1.36	102.85	1.21
S_{II}	P_{u3}	P_u	113.98	74.14	1.54	87.61	1.30	69.28	1.65	80.46	1.42	87.32	1.31
	P_{u8}	P_u	125.88	110.64	1.13	105.65	1.19	93.51	1.35	91.19	1.38	102.85	1.22
S_{III}	P_{b3}	P_b	117.63	141.46	0.83	115.17	1.02	68.25	1.78	80.46	1.46	87.32	1.35
	P_{b8}	P_b	124.35	142.16	0.87	116.58	1.07	92.26	1.35	91.19	1.46	102.85	1.21
S_{IV}	P_{u3}	P_u	113.98	74.14	1.54	87.61	1.30	69.28	1.65	80.46	1.41	87.32	1.31
	P_{cu3}	P_u	100.05	74.14	1.35	87.61	1.30	68.82	1.45	80.46	1.24	87.32	1.15

1 kip = 1000 lb =453.6 kg

Notations:

V_{test} = measured ultimate shear force.

V_{flex} = shear force corresponding to flexural failure, obtained by Yield Line Theory for square slab with the corners free to lift.

V_{calc} = calculated ultimate shear force.

CONCOLUSIONS

The purpose of this investigation was to determine the effect on the behaviour, the actual failure mechanism and the shear strength for the connections of slab bridges and floor slabs to their supports of:

1- The varying average effective concrete presress, f_{pc}.
2- The type of floor slabs, which may be Ordinary or Prestressed.
3- The type of presressing tendons, unbonded or bonded ones.
4- The effect of concentration of tendons at the supporting column vicinity.
5- The effect of the compressive strength of concrete.
6- Studying the applicability and comparing between the most recognized analytical methods.

Studying also the availability of those methods to cater for the important parameters that thought to have the greatest influence on the behavior and shear strength. It also aimed at obtaining useful data for design purposes of slab-columns in slab bridges and flat plate structures. The results of this investigation showed that:

Table 3. Comparison of V_{test} and V_{calc} after modification.

Group No.	Specimen No.	Type of reinf.	V_{test} kips	Y.L.Theory		Moe		Scordelis ,Lino & May		Grow and Vanderbilt		ACI 1989 Code	
				$V_{calc.}$ kips	$\frac{V_{test}}{V_{calc}}$	V_{calc} kips	$\frac{V_{test}}{V_{calc}}$	V_{calc} kips	$\frac{V_{test}}{V_{calc}}$	Vcalc kips	$\frac{V_{test}}{V_{calc}}$	V_{calc} kips	$\frac{V_{test}}{V_{calc}}$
S_I	R	R	85.4	131.44	0.65	106.57	0.80	-	-	-	-	70.86	1.21
	P_{u3}	P_u	113.98	133.57	0.85	112.49	1.01	69.28	1.65	80.46	1.42	87.32	1.31
	P_{b3}	P_b	117.63	141.46	0.83	115.17	1.02	68.25	1.72	80.46	1.46	87.32	1.35
	P_{u8}	P_u	125.88	133.93	0.93	113.80	1.11	93.51	1.35	91.19	1.37	102.85	1.22
	P_{b8}	P_b	124.35	142.16	0.87	116.58	1.07	92.26	1.35	91.19	1.36	102.85	1.21
S_{II}	P_{u3}	P_u	113.98	133.57	0.85	112.49	1.01	69.28	1.65	80.46	1.42	87.32	1.31
	P_{u8}	P_u	125.88	133.93	0.93	113.82	1.11	93.51	1.35	91.19	1.38	102.85	1.22
S_{III}	P_{b3}	P_b	117.63	141.46	0.83	115.17	1.02	68.25	1.78	80.46	1.46	87.32	1.35
	P_{b8}	P_b	124.35	142.16	0.87	116.58	1.07	92.26	1.35	91.19	1.46	102.85	1.21
S_{IV}	P_{u3}	P_u	113.98	133.67	0.85	112.49	1.01	69.28	1.65	80.46	1.41	87.32	1.31
	P_{cu3}	P_u	100.05	133.64	0.75	112.69	0.89	68.82	1.45	80.46	1.24	87.32	1.15

1- High shear strength was obtained by prestressing the floor slab, even when applying low average concrete prestress, f_{pc}. The increase in shear strength was 33.4% and 47.4% for f_{pc} between 300 psi (21 kg/cm^2) and 500 psi (35 kg/cm^2) respectively. Also shear strength increased with the increase of f_{pc}, with almost the same rate for unbonded and bonded tendons.

2- Small increase in shear strength was noticed when using bonded prestressed tendons for slabs instead of unbonded ones, only when the applied average concrete prestress, f_{pc}, was small, compare between the results of Groups S_{II}andS_{III}, Tables(2) and (3).

3- There was no advantage of concentrating prestressed tendons,in the column vicinity. This may be due to the possible reduction in the shear strength due to the location of the prestressed tendons close to the critical shear planes.

4- The effect of compressive strength of concrete on the shear strength capacity, although of major effect, was not covered in this investigation, because its variation within the test specimens was negligible.

5- Examination of the design methods for obtaining the shear strength of floor slab-column connections show that:

Equations (3),(4)and(5) developed for prestressed floor slabs made of either normal weight or light weight concrete do not accommodate for the presence of Ordinary Reinforced Steel in the floor slab.They also did not account for unbonded and bonded prestress tendons effect. Equation(4) did not take the effect of compressive strength of concrete on the shear strength into consideration.

6- Comparison of the test results with the calculated values of Eqs.(1), (3), (4) ,(5) and (6) show that:

Equation(1), although was developed for normally reinforced floor slabs and does not account for the average concrete prestress, f_{pc}, surprisingly gave good results for the present tests, table(2)and(3), and those tested by others[6,7]. The reason for obtaining such good results by Eq(1) may be, mainly, due to the great influence of the flexure shear strength, V_{flex} which is the major parameter for the shear strength computed by this equation.

The flexural shear strength of the connection, V_{flex} depending mainly on the type and amount of the prestressing tendons i.e. the area and the specified tensile strength of the prestressing tendons, f_{pu}.

7- Equations (3) and (4) were found to be extremely conservative and do not take all the parameters affecting the shear strength into account it also did not consider the actual mechanism of failure.

8- Equation (5) and the suggested Eq.(6) produced reasonably conservative results, but they are capable of predicting the behaviour of the interior bridge and floor slab-column connections. Albeit simple to apply, they do not include all the variables affecting the shear strength.

9- Although the suggested specified tensile strength, f_{pu}, based on data obtained in this study resulted in good correlation between the computed and the test results, as given in table(3), further study is required in view of the actual mechanism of floor slab structure further than the studied short span simply supported floor slab models.

10- The punching of the column through the slab at ultimate can be considered a secondary phenomenon following the destruction of the compression zone, due to combined flexure and shear, after the web-shear cracking was occurred.

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