Three post-tensioned flat plate specimens (9 ft square) with a single column stub in the center and reinforced with various amounts of bonded reinforcement were loaded to failure. Test data were obtained on shear capacity, flexural strength, and general behavior in the column connection area. The results were compared with the current ACI Building Code (318-71) design equations and previous flat plate test results. The supplementary bonded reinforcement enhanced overall behavior providing crack control and increasing shear and flexural strength.

This study represents only the first phase of a continuing research program on post-tensioned flat slabs. The results of the two later nine-panel model tests will be reported upon in the near future.
The behavior of post-tensioned flat plates in the critical column area is of great concern to designers. Unfortunately, specific design provisions for shear strength and flexural strength of unbonded prestressed slabs are not provided in the present ACI Code (ACI 318-71).1

In order to provide answers to practical design questions, an extensive post-tensioned flat plate test program is currently being conducted at The University of Texas at Austin. The primary objective of this test series is to determine the overall behavior of a representative flat plate structure over the total range of loading. Data obtained can then be used to determine the reliability of analysis and design techniques presently being used in practice. The entire series will consist of three isolated column-slab model tests reported herein and two nine-panel scale models (one-third and one-half) designed to simulate practical bay arrangements, tendon layouts, and loading conditions.

The primary objective of the three isolated column model tests was to gain a better understanding of the behavior and strength in the critical column area and to investigate the influence of various amounts of supplementary bonded reinforcement placed in the column vicinity. Specific objectives of the program may be summarized as follows:

1. To observe the physical behavior of each test specimen, including deflections, crack patterns, crack widths, and mode of failure.
2. To study the effects on performance and contribution to strength of supplementary bonded reinforcement placed in the critical area around the column.
3. To examine the tendon stress at ultimate and check the reliability of the equations provided by ACI 318-71.
4. To compare the flexural strength with the corresponding values obtained by using ACI 318-71.
5. To compare shear strength with the values predicted by ACI 318-71 and some empirical formulas tested by other researchers.
DESCRIPTION OF TEST PROGRAM AND SPECIMENS

The three specimens tested were isolated panels with column stubs representing the region around an interior column of a multiple panel slab (Fig. 1). The models were designed following the "Tentative Recommendations for Prestressed Concrete Flat Plates," as reported by ACI-ASCE Committee 423.2

All three one-third scale slab specimens (S-1, S-2, and S-3) were identical in dimension, tendon layout, and tendon profile. The only variable was the amount of supplementary bonded reinforcement located in the column region. Each specimen was of normal weight concrete, 9 ft square, 2 3/4 in. thick, with an 8 in. square column stub in the center of the panel (see Fig. 2).

All specimens were post-tensioned in both directions with 18 tendons each way, as shown in Fig. 3a and 3b. Tendons were ASTM A-416, Grade 250, 1/8 in. diameter, 7-wire stress-relieved strands, which for one-third scale correspond to two 1/2 in. diameter strands in the prototype. All strands were mastic coated and paper wrapped to pre-

![Fig. 1. Plan view of prototype slab and model slab.](image-url)
vent bonding to the concrete. The tendon spacing corresponds to a 70 percent distribution in the column strip and a 30 percent distribution in the middle strip, with two tendons passing through the column in each direction.

The design average concrete pre-stress force, \( P/A \), after losses was to be 325 psi, corresponding to a tendon force of 5.40 kips per strand. To minimize variation in stress due to frictional effects, alternate tendons were stressed from opposite ends.

Design concrete strength for all specimens was 4000 psi, using Type III cement and \( \frac{3}{8} \) in. maximum size aggregate. Actual \( f'c \) values are given in Table 1. The column stub for all specimens was reinforced with four No. 6 bars, one in each corner, with plain No. 2 ties spaced at 6 in.
Supplementary reinforcement

All supplementary, bonded reinforcement was No. 2 deformed bars with a cross-sectional area of approximately 0.05 sq in. and a yield strength of 55.8 ksi.

Specimen S-1 contained no supplementary bonded reinforcement and served as a control specimen.

Specimen S-2 contained the minimum amount of bonded reinforcement recommended in Section 2.10.1 of the ACI-ASCE 423 Recommendations. The report recommends that a minimum area of bonded reinforcement equal to 0.15 percent of the cross-sectional area of the column strip should be provided in the top of the slab in both directions over the column. To satisfy this requirement, Specimen S-2 contained five No. 2 deformed bars, 3 ft 6 in. long, spaced at 4 in. on center in each direction, as shown in Fig. 4.

In Specimen S-3, bonded reinforce-
ment amounting to 0.24 percent of the column strip area was provided corresponding to a 60 percent increase over the amount in Specimen S-2. Eight No. 2 deformed bars were used, with a length of 3 ft 6 in. and spaced at 2 2/3 in. on center in each direction, as shown in Fig. 5. All No. 2 bars in Specimens S-2 and S-3 were placed in between the unbonded tendons at the same depth as the tendons in both directions.

**TESTING**

**Loading**

Loading of the slab was accomplished by pulling downward with a whiffle-tree apparatus producing a line load 4 ft square, as shown in Fig. 2. This loading was chosen to provide a symmetrical load around the column simulating loading conditions at a typical interior column. The load line was located a sufficient distance away from the column so it would not directly affect the strength or cracking behavior in the column vicinity and so the column area would remain completely visible for observation during testing.

A separate whiffle-tree was used for each side of the load line providing four load points along each line. Each whiffle-tree was loaded with a 30-ton hydraulic ram, all of which were connected to a common manifold to produce equal loading along all four sides. The loading frame and whiffle-tree arrangement are shown in Fig. 6.

**Instrumentation**

Prestress forces in the tendons were measured by 12 load cells placed at the holding ends of the tendons, as shown in Fig. 7. Six load cells were provided in each direction and were monitored during stressing and loading.
Table 1. Summary of test data.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>S-1</th>
<th>S-2</th>
<th>S-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Strength at Time of Test $f'_c$ (psi)</td>
<td>4350</td>
<td>4193</td>
<td>4625</td>
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<tr>
<td>Supplementary Reinforcement</td>
<td>None</td>
<td>S-5 NW</td>
<td>S-7 EW</td>
</tr>
<tr>
<td>Design Load (Applied Load kips)</td>
<td>13.6</td>
<td>13.6</td>
<td>13.6</td>
</tr>
<tr>
<td>1st Cracking Load (kips)</td>
<td>16.3</td>
<td>16.3</td>
<td>19.3</td>
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<tr>
<td>Failure Load (kips)</td>
<td>25.3</td>
<td>27.3</td>
<td>30.3</td>
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</table>

<table>
<thead>
<tr>
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<th>S-1</th>
<th>S-2</th>
<th>S-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment Capacity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate Load Based on Moment Capacity (kips)</td>
<td>25.25</td>
<td>27.31</td>
<td>30.41</td>
</tr>
<tr>
<td>$P_u$(test)</td>
<td>24.29</td>
<td>27.06</td>
<td>27.49</td>
</tr>
<tr>
<td>$P_u$(calc) ACI Code</td>
<td>1.04</td>
<td>1.01</td>
<td>1.11</td>
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<table>
<thead>
<tr>
<th>Specimen</th>
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<th>S-3</th>
</tr>
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<tbody>
<tr>
<td>Shear Strength</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate Shear Stress $V_u$ psi</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>$V_u$ (test)</td>
<td>295</td>
<td>319</td>
<td>350</td>
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<tr>
<td>$V_u$ (calc) = $4\sqrt{f'_c}$ ACI Code</td>
<td>264</td>
<td>259</td>
<td>272</td>
</tr>
<tr>
<td>$V_u$(test) / $V_u$(calc)</td>
<td>1.12</td>
<td>1.23</td>
<td>1.29</td>
</tr>
<tr>
<td>$V_u$(test) / $V_u$(calc) x $\phi$</td>
<td>1.32</td>
<td>1.45</td>
<td>1.52</td>
</tr>
<tr>
<td>$V_{cw}$(calc) ACI Eq. 11-12</td>
<td>329</td>
<td>324</td>
<td>332</td>
</tr>
<tr>
<td>$V_u$(test) / $V_{cw}$(calc)</td>
<td>0.90</td>
<td>0.985</td>
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<tr>
<td>$V_u$(test) / $V_{cw}$(calc) x $\phi$</td>
<td>1.05</td>
<td>1.16</td>
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<td>$V_u$(LSM) Lin-Scordelis-May Eq.</td>
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<td>316</td>
</tr>
<tr>
<td>$V_u$(test) / $V_u$(LSM)</td>
<td>0.98</td>
<td>1.05</td>
<td>1.11</td>
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<tr>
<td>$V_u$(GV) Grow-Vanderbilt Eq.</td>
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<td>360</td>
<td>360</td>
</tr>
<tr>
<td>$V_u$(test) / $V_u$(GV)</td>
<td>0.82</td>
<td>0.89</td>
<td>0.98</td>
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</tbody>
</table>

Strains in the bonded reinforcement were measured by SR-4 foil strain gages with a gage length of $\frac{1}{4}$ in. The gages were located for Specimens S-2 and S-3 as shown in Figs. 4 and 5.

Concrete strains near the column were measured with 0.80 in. paper backed electrical resistance type strain gages applied directly to the concrete surface. The locations of these gages was the same for all three specimens and are shown in Fig. 8.

Deflections of the specimens under load were measured at 18 points, 9 points in each direction, by dial gages with an accuracy of 0.001 in. The loca-
tions of the dial gages are shown in Fig. 8, and the support frame for the dial gages is shown in Fig. 9. The supporting frame for the dial gages was totally independent of the test specimens and the loading system, thus providing a fixed reference line for deflection data.

In order to monitor the loading of the specimens, a 100-kip load cell was placed under the column stub. Load cells were also located under two of the loading rams and at two load points on the top of the specimen. When load was applied by the hydraulic rams, all load cells were monitored to check the efficiency of the loading system in distributing the desired line loading.

Test procedure

The applied load was primarily regulated by reading the 100-kip load cell under the column with a calibrated pressure gage on the pump used as a check. Load was applied in 2000-lb increments up to the first cracking load for all three specimens and then unloaded in 4000-lb increments. Specimen S-1 was then loaded in 4000-lb increments to failure and Specimens S-2 and S-3 were loaded in increments of 2000 and 1000 lb to failure.

At every loading stage all instrumentation data were recorded. All cracks were marked and when cracks began to enlarge significantly, crack widths at several locations were measured with a small hand microscope. No additional load was applied after failure had occurred.

RESULTS

General

All three specimens behaved almost identically in the elastic range up to first cracking loads recovering practically all deflection when unloaded, as shown in the deflection curves in Fig. 10. As the specimens were reloaded
past the first cracking load, the extra stiffness and ductility provided by the supplementary bonded reinforcement in Specimens S-2 and S-3 began to be noticeable when compared with Specimen S-1.

The cracking patterns and failure shear perimeters are shown in Figs. 11 and 12. All specimens failed in a combination of flexure and shear with a final failure mode of "punching shear." In all tests, the shear perimeter that appeared at failure outlined the base of a truncated pyramid. The surface of failure ex-
tended conically from the perimeter of the column at the bottom surface of the slab to the perimeter on the top surface of the slab, shown as a dotted line in the above-referenced figures.

After failure, all three specimens remained supported by the four tendons passing through the column and aided by the bonded reinforcement in Specimens S-2 and S-3. Each specimen remained intact after failure and did not completely collapse.

The first cracking loads and failure loads are given in Table 1, as compared to the design service load of 13.6 kips. All specimens exhibited similar cracking
behavior throughout the test. First cracks occurred at the column corners and at the column-slab interface.

In Specimen S-1, very large flexural cracks opened suddenly under an applied load of 21 kips, propagating across the entire width of the slab in the north-south direction and greatly reducing the stiffness of the slab.

Specimens S-2 and S-3 exhibited similar behavior, except that when the large flexural cracks opened at 20 and 21 kips, respectively, the decrease in stiffness was not as pronounced. Also, the overall crack distribution was much improved in Specimens S-2 and S-3.

Ductility improved with the increase of bonded reinforcement as both Specimens S-2 and S-3 maintained applied load even when severely cracked.

Specimen S-1 did not behave well after severe cracking, and was unable to sustain the applied load without severe deformation, and failed soon afterward. Crack width measurements showed that for a given loading, crack widths for Specimen S-1 were higher than for either Specimens S-2 or S-3.

**Tendon stress**

Load cell measurements located on the tendons indicated only very small

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*Fig. 11a. Cracking pattern for Specimen S-1.*
tendon stress increases near the failure load for all three specimens. Throughout the elastic range and past the first cracking load there was no tendon stress increase observed at the holding end in any tendon for all three specimens. This trend continued until severe flexural cracking and deformation occurred at the higher loads near failure.

Measurements for Specimen S-1 were last taken at 18 kips applied load and indicated no stress increases. In Specimen S-2 the tendon stress increase at an applied load of 23 kips (last measured load increment) for one interior tendon was 13.82 ksi with the greatest increases occurring in the east-west tendons due to the smaller effective depth.

Specimen S-3 exhibited similar behavior with little increase in tendon stress until large flexural cracks and deformation had occurred near the failure load.

Stress increases occurred in all tendons with maximum increases of 13.82 and 14.16 ksi in the two east-west column tendons. Only small increases were observed in all other tendons.

Concrete strains

Concrete strain measurements on the top and bottom slab surfaces near the
column indicated elastic behavior of all three specimens up to first cracking. When the higher loads were reached, erratic measurements were observed from the top gages, indicating cracking near the gages.

Marked strain increases were observed from the compression gages on the bottom surface as the applied load passed the cracking load and flexural cracking became prevalent. Compressive strains were quite high for Specimens S-2 and S-3, indicating compressive stresses near $f'_{c}$. However, no crushing of the concrete was observed prior to failure.

**Supplementary bonded reinforcement**

Strains in the supplementary bonded reinforcement in Specimens S-2 and S-3 showed that the behavior for both specimens was virtually identical. All load-strain curves for the bars exhibit a linear elastic region up to first cracking with almost complete recovery on unloading (see Fig. 13).

In both specimens the bars passing through the columns yielded first with all bars reaching yield prior to failure and exhibiting some strain hardening. In the north-south direction only the column bars reached yield prior to failure. The east-west bars would be expected to yield first since they were located under the north-south bars and thus had a smaller effective depth.

**DISCUSSION**

**General**

Deflection characteristics for all three specimens were virtually identical for the range of loading up to 18 kips ap-
plied load. The deflection readings have little quantitative significance for continuous slabs, since no edge restraint was present that could provide the stiffness due to continuity.

However, the deflections do show the elastic recovery of the specimens in the early load stages and exhibit the reduced stiffness due to cracking at higher applied loads. They also show the increase in ductility and stiffness due to the supplementary bonded reinforcement in Specimens S-2 and S-3 over the behavior of Specimen S-1 at the higher applied loads near failure.

Although the first cracking load was taken at the first appearance of hairline cracks occurring at the slab-column interface and at the column corners, these cracks have little practical significance, since they were localized at points of peak moment and were due largely to stress concentrations. Significant cracking did not occur in any specimen until applied loads of 16 to 18 kips were reached.

Crack distribution and the number of cracks observed varied considerably between each specimen. Up until failure, relatively few cracks appeared in Specimen S-1. As the load was increased, the first cracks simply propagated and widened instead of new cracks forming. This is typical of structures reinforced with unbonded tendons only, as the tendons tend to slip causing large concrete strains to be accumulated at a few locations.

However, for Specimens S-2 and S-3, the crack distribution was good with a greater number of cracks observed and smaller crack widths due to the effects of the supplementary reinforcement (see crack patterns in Fig. 11a, 11b, and 11c).

**Tendon stress**

Maximum stress increase was observed in the east-west tendons passing through the column for all three specimens. This was due primarily because the east-west tendons had a smaller effective depth (see Fig. 3b) than the north-south tendons. The maximum increases occurred in Specimen S-3 with Specimen S-2 tendon stresses very near the same. Specimen S-1 indicated little or no increase prior to failure.

The maximum stress increases observed at the holding end were only 59 percent of that predicted by the ACI Eq. (18-4). However, this type test specimen, due to its lack of continuity and actual plate behavior, may not yield quantitatively significant results.

**Bonded reinforcement**

The presence of bonded reinforcement in both Specimens S-2 and S-3 greatly improved their performance under load. The crack width reductions and greatly increased number of cracks observed for Specimens S-2 and S-3 were significant improvements over observed behavior of Specimen S-1. Strain measurements on the reinforcing bars proved their effectiveness, as all the east-west bars in both Specimens S-2 and S-3 reached the yield point.

As the amount of bonded reinforcement was increased, the ductility and load-carrying capacity of the specimen was increased. The bonded reinforcement also helped to hold the specimen intact after failure.

**Flexural strength**

ACI 318-71 specifies in Section 18.8.3 that the total amount of prestressed and nonprestressed reinforcement shall be adequate to develop a design load in flexure at least 1.2 times the cracking load as calculated on the basis of a modulus of rupture, \( f_r = 7.5 \sqrt{f_{c'}} \).

Since this requirement refers to a cracking moment based on beam action, there is some question as to its validity when applied to two-way prestressed flat plates. For the design loads used, 1.2 times the cracking load
moment is substantially in excess of the design ultimate moment.

Cracking loads from the test, \( P_{cr(test)} \), were all found to be below the cracking loads based on ACI 318-71, \( P_{cr(calc)} \), with \( P_{cr(test)}/P_{cr(calc)} \) ratios ranging from 0.82 for Specimen S-1 to 0.95 for Specimen S-3. This discrepancy is due to the fact that first cracking was taken at the appearance of small hairline cracks at the column corners, as described earlier. The applied loads, when significant cracking occurred, would have yielded ratios exceeding unity.

Table 1 shows a comparison of actual ultimate loads, \( P_u(test) \), and the ultimate loads, \( P_u(calc) \), based on the predicted ultimate moment capacity according to ACI 318-71. The predicted ultimate moment capacity was calculated assuming that the tendon stress would increase according to Eq. (18-4) in the Code.

According to the values shown in Table 1, \( P_u(test) \) was higher than \( P_u(calc) \) for all three specimens and they all should have failed in flexure. The large flexural cracks and deformation of each specimen near the failure load indicate that flexural failure was imminent and probably contributed to the punching shear failure.

Since the predicted values assumed higher tendon stress increases at ultimate than were actually observed, the \( P_u(test)/P_u(calc) \) ratios should actually be even larger if based on the actual tendon stress at failure.

Although the tendons did not contribute the full tensile force predicted by ACI 318-71, the extra moment capacity can be attributed to the extra flexural stiffness provided by the two-way deformation of the slab. Also, in Specimens S-2 and S-3, the strain readings for the bonded reinforcement indicated that most bars, especially those passing through the column, had yielded and had begun to strain harden, thus contributing a higher tensile force than the yield strength used for the predicted ultimate moment values.

**Shear strength**

The shear strength of post-tensioned flat plates is governed by two-way action with potential diagonal cracking along the surface of a truncated pyramid around the column, as defined in Section 11.10.2 of ACI 318-71. All three specimens failed in a combination of flexure and shear, with the actual failure mode occurring as punching shear.

Shear strength design methods and
requirements for prestressed flat plates are not defined in ACI 318-71 or local building codes. Section 11.10 of the Code allows an ultimate shear stress of $4 \sqrt{f_c}$ used in conjunction with the $\phi$ factor for shear of 0.85.

This allowable shear stress is to be used for two-way slabs at a critical area located a distance $d/2$ from the face of the support. However, no differentiation is made between prestressed and normal reinforced concrete construction.

Another ACI 318-71 equation has been recommended by ACI-ASCE Committee 423. The same critical section is used and the nominal ultimate shear stress is calculated by Eq. (11-12) of ACI 318-71:

$$ v_{cu} = 3.5 \sqrt{f_c} + 0.3f_{pc} + \frac{V_p}{b_w d} $$

The last term contributes only a very small portion to the shear strength and is usually conservatively neglected. Although this equation takes into consideration the prestress force, it has the disadvantage of being based on beam action instead of two-way plate behavior.

Other empirical data and shear strength equations have been published by Lin, Scordelis, and May, Grow and Vanderbilt, and Gerber and Burns. All of these test programs involved the testing of specimens very similar to Specimens S-1, S-2, and S-3 reported herein and therefore significant comparisons can be made between them. The shear strength of Specimens S-1, S-2, and S-3 are compared with the two ACI 318-71 predicted values and the empirical values in Table 1.

The $v_{u(test)}/v_{u(calc)}$ ratios are greater than unity and therefore conservative using the ACI 318-71 provisions for
two-way slabs. However, ACI Code Eq. (11-12) provides ratios for Specimens S-1 and S-2 which are less than 1, while the Specimen S-3 ratio was 1.05, showing very close correlation to this equation. When the \( \phi \) factor of 0.85 is introduced, both ACI equations yield conservative ratios above unity.

Comparison with the empirical equations based on prestressed slab behavior indicates unconservative ratios for \( V_u(test)/V_u(calc) \). The Lin-Scordelis-May equation, \( V_u(test)/V_u(LBM) \), predicts the behavior of Specimens S-1 and S-2 quite closely. As the amount of bonded reinforcement increases the ratios increase with the most conservative ratio for Specimen S-3. All ratios using the Crow-Vanderbilt equation, \( V_u(test)/V_u(GV) \), are less than 1.0 and unconservative, but do indicate the same upward trend as the amount of bonded reinforcement is increased.

The low \( V_u(test)/V_u(calc) \) ratios can be attributed to the severe flexural cracking present at loads near failure. After the applied load reached the theoretical ultimate moment capacity of the specimens, severe flexural distress was observed in the form of widespread cracking in the critical area around the column. This cracking reduced the shear strength of the specimens in this region, resulting in early shear failure.

The higher ratios exhibited by Specimens S-2 and S-3 can be attributed to the presence of the bonded reinforcement which increased the moment capacity and was effective in distributing cracks and controlling crack width in the column area.

**CONCLUSIONS**

From the results of this test program the following conclusions can be drawn:

1. In the elastic range the behavior observed for all three specimens was almost identical. All specimens showed elastic response up to the first cracking load, unloading, and reloading to the point of first cracking.

2. The first cracking load has little practical significance since first cracking is localized at points of peak moment and is due largely to stress concentrations. The slab is capable of sustaining much larger loads before severe cracking occurs.

3. The bonded reinforcement improved the flexural behavior, improved the crack distribution, reduced the maximum crack widths, and increased the load carrying capacity of Specimens S-2 and S-3. As the amount of bonded reinforcement increased, the behavior of the specimens improved.

4. The stress in the unbonded tendons remained virtually constant until severe flexural cracking and deformation had occurred near the failure load. The ACI 318-71 equation for estimating tendon stress at ultimate, Eq. (18-4), proved unconservative for all three specimens.

5. All three specimens failed by punching shear, which followed the development of severe flexural cracks.

6. The observed flexural strength was greater than the strength predicted by ACI 318-71 for all three specimens. The flexural capacity was improved by bending in two-way action, while the ACI Code equation is based on beam theory. The bonded reinforcement increased the capacity of Specimen S-2 by 8 percent and Specimen S-3 by 20 percent compared to the companion Specimen S-1 with no bonded reinforcement.

7. Shear strengths from these test specimens (4.45 \( \sqrt{f'_c} \), 4.93 \( \sqrt{f'_c} \), and 5.16 \( \sqrt{f'_c} \), for Specimens S-1, S-2, and S-3, respectively) were slightly above the values predicted by Eq. (11-12) based on ACI-ASCE Committee 423.
recommendations. The nominal permissible shear strength based on two-way action according to Section 11.10 of ACI 318-71 was conservative for all three specimens. The shear strength from these test specimens gave results similar to those results predicted from the empirical equation by Lin-Scordelis-May. The equation developed by Grow and Vanderbilt was unconservative for all three specimens although the shear strength of Specimen S-3 was very closely estimated.

ACKNOWLEDGMENT

This study is the first phase of a continuing study on post-tensioned flat slabs (two later nine-panel models were tested) at The University of Texas at Austin sponsored by the Post-Tensioning Division of the Prestressed Concrete Institute and the Reinforced Concrete Research Council.

Theses by Stephen W. Smith and Chaiwat Changwatchai in partial fulfillment of their MS degrees served as the basis for this paper. The work was supervised by Dr. Ned Burns and an oral presentation by Dr. Burns to the PCI at its Annual Convention in Atlanta, Georgia, October 11, 1972, covered preliminary results from this study.

REFERENCES

1. ACI Committee 318, “Building Code Requirements for Reinforced Concrete (ACI 318-71),” American Concrete Institute, Detroit, 1971, 78 pp.


Discussion of this paper is invited. Please forward your discussion to PCI Headquarters by October 1, 1974, to permit publication in the November-December 1974 PCI JOURNAL.