ULTIMATE STRENGTH TESTS OF POST-TENSIONED FLAT PLATES

Ten post-tensioned flat plate specimens, 12 ft. (3.7 m) square, simulating both lift-slab and cast-in-place construction, were loaded to failure, with observations made of shear capacity, flexural capacity, and ability to carry load after initial concrete failure. Results indicate that current shear design practice is adequately conservative, providing factors of safety of the order of 3 to 4½. Supplemental reinforcing steel was found to help control cracking, increase ductility of the structure, and greatly improve transmission of loads to columns.

Loris L. Gerber
The Prescon Corp.
Corpus Christi, Texas

Ned H. Burns
The University of Texas
Austin, Texas

Following the collapse of the Four Seasons Apartment Building in the Anchorage, Alaska, earthquake of 1964, many questions arose regarding the adequacy of post-tensioned flat plates, particularly those erected by the lift-slab technique. As a result of these questions and a general lack of validated test information on post-tensioned construction of this type, the American BBR Research Association was formed in 1964 for the sole purpose of research and development in the field of post-tensioned concrete. Membership was divided into two groups:

Active Members

Western Concrete Structures Co., Inc.
  Gardena, California
The Prescon Corporation
  Corpus Christi, Texas
Prestressing Industries, Inc.
  San Antonio, Texas
American Stress Wire Corp.
  Englewood, Colorado

Contributing Members

Joseph T. Ryerson & Son, Inc.
  Chicago, Illinois
Transit Mixed Concrete Co.
  Los Angeles, California
Stressteel Corporation
  Wilkes-Barre, Pennsylvania

All of the active members, as well as contributing member Joseph T. Ryerson and Son, Inc., were active in the post-tensioning industry using the BBR buttonhead system of post-tensioning anchorage.

Tests were conducted by the Association in the latter part of 1965 in Gardena, California, to study the ultimate load capacity of post-tensioned flat plates, and the effect of supplemental non-stressed reinforcement, for both cast-in-place and lift-slab construction methods. The data were withheld from earlier publication because of litigation, however, not connected with these tests. The test activity was intended merely to develop information for selecting an optimum quantity and configuration of supplemental reinforcing steel and the column-tendon relationship for use in a future full-scale test. Such a test, or a comparable scale model test, is planned for the near future. Meanwhile, the information now released in this report may offer valuable comparisons with computer designs or actual tests of flat plate post-tensioned floor systems.

INTRODUCTION

These tests were conducted to study the ultimate strength of post-tensioned flat plates. Both lift-slab and cast-in-place specimens were tested. Specifically, the program was designed to provide information in the following areas:

1. Shear capacity—Compare test results with values predicted by design practice. Determine relative effectiveness of different reinforcing steel configurations.
2. Flexural capacity—Same as for shear capacity.
3. Reserve capacity—Determine effectiveness of the various specimens in sustaining load after primary failure has occurred.
4. General—A qualitative analysis of the various reinforcing steel configurations and their effect on crack control, load capacity, and other factors.

DESCRIPTION OF SPECIMENS

All ten slab specimens tested were normal weight concrete flat plates 7 in. (18 cm) thick and 12 ft. (3.7 m) square. The specimens were supported on precast columns 12 in. (30 cm) square and were loaded with four 100-ton (91 t) hydraulic jacks. Four of the specimens simulated cast-in-place construction and six of the specimens simulated lift-slab construction. The specimens tested were designated C-1, C-2, C-3, C-4, L-1, L-2, L-3, L-4, L-7 and L-8. Letters C and L indicate cast-in-place or lift-slab specimens, respectively, and

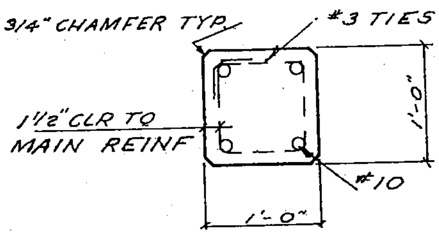

Fig. 1. Column detail

the number in each designation represents the reinforcement configuration. As far as practicable, the cast-in-place specimens and the lift-slab specimens having the same number in their designations had the same amounts and configurations of supplementary reinforcement.

All of the specimens were post-tensioned in both directions with six tendons, each composed of six wires having a diameter of ¼ in. (6 mm). The tendons were anchored to a force of 49.4 kips (168 ksi) (22t or 11,800 kgf/cm²) and the final force after losses was 42.0 kips (19 t) per tendon (143 ksi) (10,000 kgf/cm²). With the exception of specimen L-7 which had grouted tendons, all tendons were mastic coated and paper wrapped prior to being placed in the forms and hence were not bonded to the concrete. The only other variation in the post-tensioning was the horizontal spacing of the tendons. This varied because the first tendon from the column centerline ran over the column in the cast-in-place specimens and over the lifting collars in the lift-slab specimens. The post-tensioning force selected resulted in an average compression of 250 psi (17.6 kgf/cm²) in the concrete after an assumed value of 25 ksi (1760 kgf/cm²) losses in the prestressing tendons had taken place.

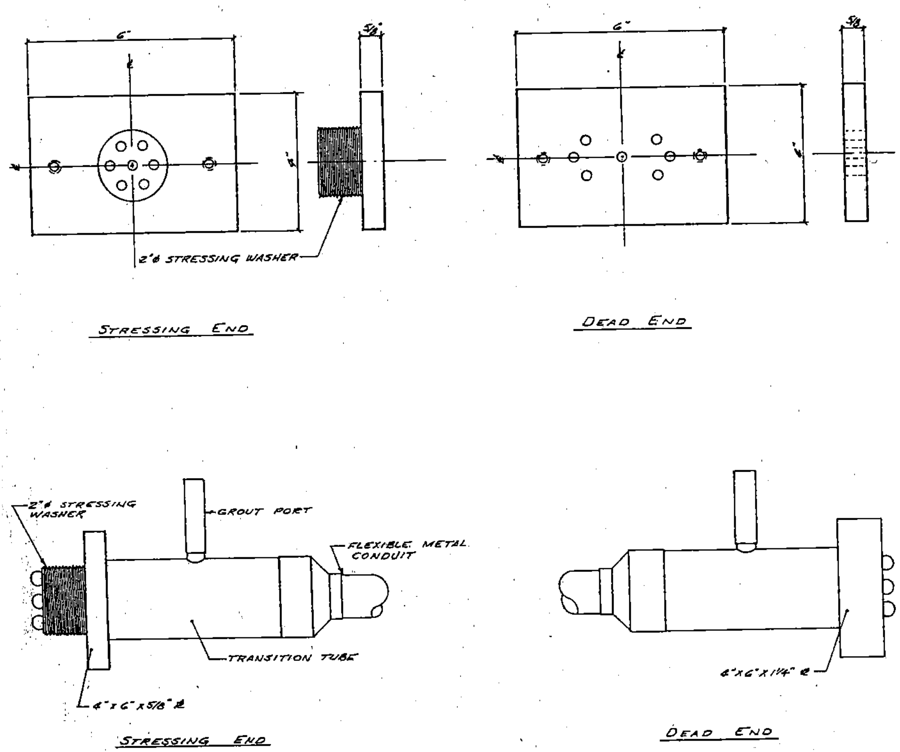

Fig. 2. Tendon anchorage details

Table 1. Reinforcing steel properties

Type or size of reinforcement	Yield point, psi (kgf/cm²)	Ultimate tensile strength, psi (kgf/cm²)	Elastic modulus, psi (kgf/cm²)
No. 3 (10 mm)	50,900 (3,600)	72,750 (5,100)	—
No. 5 (16 mm)	61,290 (4,300)	79,350 (5,000)	28,200,000 (1,980,000)
No. 9 (29 mm)	51,750 (3,600)	82,800 (5,800)	27,100,000 (1,900,000)
Deformed wire	84,350 (5,900)	98,950 (6,950)	29,800,000 (2,100,000)
Welded wire fabric	89,150 (6,300)	96,750 (6,800)	31,460,000 (2,200,000)

In all test specimens the column had the cross section detail shown in Fig. 1. Tendon anchorages for grouted and ungrouted specimens are shown in Fig. 2. Typical tendon layouts are shown in Figs. 3 and 4.

Supplementary reinforcement. Specimens C-1 and L-1 were constructed without any supplementary reinforcing steel. Specimens C-2 and L-2 had supplementary reinforcement composed of four No. 5 (16 mm) bars 8 ft. (2.4 m) long each way in the top of the slab immediately over the post-tensioning tendons and positioned close to the column centerlines. Specimens C-3 and L-3 had supplementary reinforcing steel composed of welded wire fabric and deformed wire (Fig. 5) in a quantity that resulted in an area of steel equivalent to four No. 5 (16 mm) bars in each direction (1.24 in.²) (8.0 cm²). The supplementary reinforcement in Specimens C-4 and L-4 was proportioned to comply with the proposed requirements of the Department of Building and Safety, City of Los Angeles (steel area equal to the total column design load in kips divided by 25 with 50 percent of the area being placed each way), and consisted of two No. 9 (29 mm) bars 11½ ft. (3.5 m) long each way placed in the bottom of the slab and positioned near the column centerline in such a manner that they passed through the lifting collar or over the column. Fig. 6 shows the details of the lift-slab column connection used in this study. Specimen L-7 had no supplementary reinforcing steel but the tendons were grouted. Specimen L-8 had four No. 5 (16 mm) bars 8 ft. (2.4 m) long in the top of the slab similar to that in Specimen L-2 as well as a special shear reinforcing composed of No. 3 W-shaped bars which were anchored at the bottom of the slab by looping them around short No. 4 (13 mm) bars (Fig. 7).

Materials. The prestressing wire conformed to the minimum requirements of ASTM A 421, Type BA. The wire had an actual diameter of 0.251 in. (approx. 6 mm), a yield stress at 1.0 percent extension of approximately 210,000 psi (14,800 kgf/cm^2), an average ultimate strength of approximately 250,000 psi (17,600 kgf/cm^2), and an elastic modulus of the order of 29,000,000 psi (2,000,000 kgf/cm^2). The reinforcing steel used in the various specimens had the properties given in Table 1.

The compressive strength of the concrete at various ages, the age and strength at time of testing, and other test data are listed in Table 2.

INSTRUMENTATION

Electrical resistance type strain gauges were placed on the supplementary reinforcing steel and on the post-tensioning tendons at the locations shown in Figs. 3 and 4. The strain gauges on the post-tensioning tendons were installed after the tendons had been stressed.

Deflections of the specimens were measured at 12 points located as shown in Fig. 8. Deflections were measured with a transit by reading targets, graduated in hundredths of a foot, which were suspended from the bottom of the specimens. Deflection measurements were estimated to the nearest 0.005 ft. (1.5 mm).

The hydraulic ram loads were measured by a Transducers type PCL-78 strain gauge load cell accurate to ±1 percent.

TESTING

Loading. In order to relate the test specimens to an actual building condition, the 12 ft. (3.7 m) square specimens were assumed to be representative of a portion of a slab over an interior column of a large slab having spans of 24 ft. (7.3 m) in each direction. The full size slab, after which the specimens were patterned, was assumed to carry the loads tabulated below.

The location of the ram supports was selected to produce a moment resulting in 190 psi (13.4 kgf/cm^2) tension in the concrete under the action of the full 94,000 lb. (42,500 kgf) column load. Actual loads went as high as 276,000 lb. (125,000 kgf) or 2.95 times the design load.

Test procedure. The load was applied to the slab by four hydraulic rams located as shown in Fig. 8. It was generally applied in increments of 20,000 lb. (9000 kgf)—5,000 lb. (2300 kgf) per ram—up to 100,000 lb. (45,000 kgf) and then released.

Slab dead load—7 in. thick (18 cm)	=	88 psf (430 kgf/m^2)
Superimposed dead load	=	25 psf (122 kgf/m^2)
Live load (unreduced)	=	50 psf (244 kgf/m^2)
		163 psf (795 kgf/m^2)
Total column load for 24 x 24 ft. (7.3 x 7.3 m) bay = 24 x 24 x 163	=	94,000 lb. (42,600 kgf)
Specimen's weight = 12 x 12 x 88	=	12,600 lb. (5,700 kgf)
Total load applied by jacks to simulate column load	=	81,400 lb. (37,000 kgf)

Table 2. Summary of test data

Line No.	Specimen Designations	C-1	C-2	C-3	C-4	L-1	L-2	L-3	L-4	L-7	L-8
1	Concrete age at time of test (days)	30	22	26	28	30	22	26	28	30	30
2	Concrete strength at time of test, f'_c (psi)	5300	4600	4915	4930	5300	4500	4610	4820	5000	5000
3	Supplementary reinforcing	None	8-#5	Mesh	4-#9	None	8-#5	Mesh	4-#9	None	Special
4	Loads at which cracking first detected (kips)	92.6	152.6	92.6	92.6	92.6	112.6	72.6	72.6	112.6	112.6
5	Ave. corner deflection for 100 k load[1]—Cycle 1 (in.)	0.179	0.226	0.150	0.180	0.145	0.240	0.226	0.196	0.166	0.151
6	Ave. corner deflection for 100 k load—Cycle 2 (in.)	0.225	0.210	0.150	0.166	0.193	0.225	0.226	0.196	0.161	0.145
7	Ave. corner deflection for 160 k load (in.)	—	0.465	0.405	0.600	0.358	0.435	0.420	0.390	0.388	0.285
8	Max. load for which deflections were measured (kips)	148	170	190	160	168	180	180	170	169	180
9	Ave. corner defl. for max. load (in.)	0.570	0.570	0.750	0.600	0.403	0.600	0.630	0.420	0.450	—
10	Loads at which primary failure occurred (kips)	177.2	192.6	202.6	182.6	197.2	200.2	210.2	192.6	200.6	276.6
11[2]	Primary failure load / $f'_c \times$ shear area	4.07	5.33	5.45	4.88	4.28	4.73	4.90	4.38	4.48	6.20
12	Principal tension at failure, f_t (psi)	390	431	458	403	358	364	388	348	364	543
13	Ratio—f_t/f'_c	0.074	0.094	0.093	0.082	0.068	0.081	0.084	0.072	0.073	0.109
14	Ratio—f_t/120 psi	3.26	3.60	3.82	3.36	2.98	3.04	3.24	2.90	3.04	4.52
15	Calculated ultimate shear (kips)	109.5	105.3	113.0	112.9	151.6	158.8	158.9	156.9	155.0	155.0
16	Ratio—primary failure to calculated ultimate, R_{ult}	1.62	1.82	1.79	1.62	1.30	1.26	1.33	1.23	1.29	1.79
17	Max. reserve load (kips)	113.2	192.6	149.0	128.6	130.2	130.6	140.6	212.6	152.6	241.6
18	Ratio—maximum reserve load to design load	1.20	2.05	1.58	1.37	1.38	1.39	1.50	2.26	1.62	2.57

1) Loads given are jacking loads only—specimen weights of 12.6 kips not included.
2) Sample calculations for these values are in the Appendix.

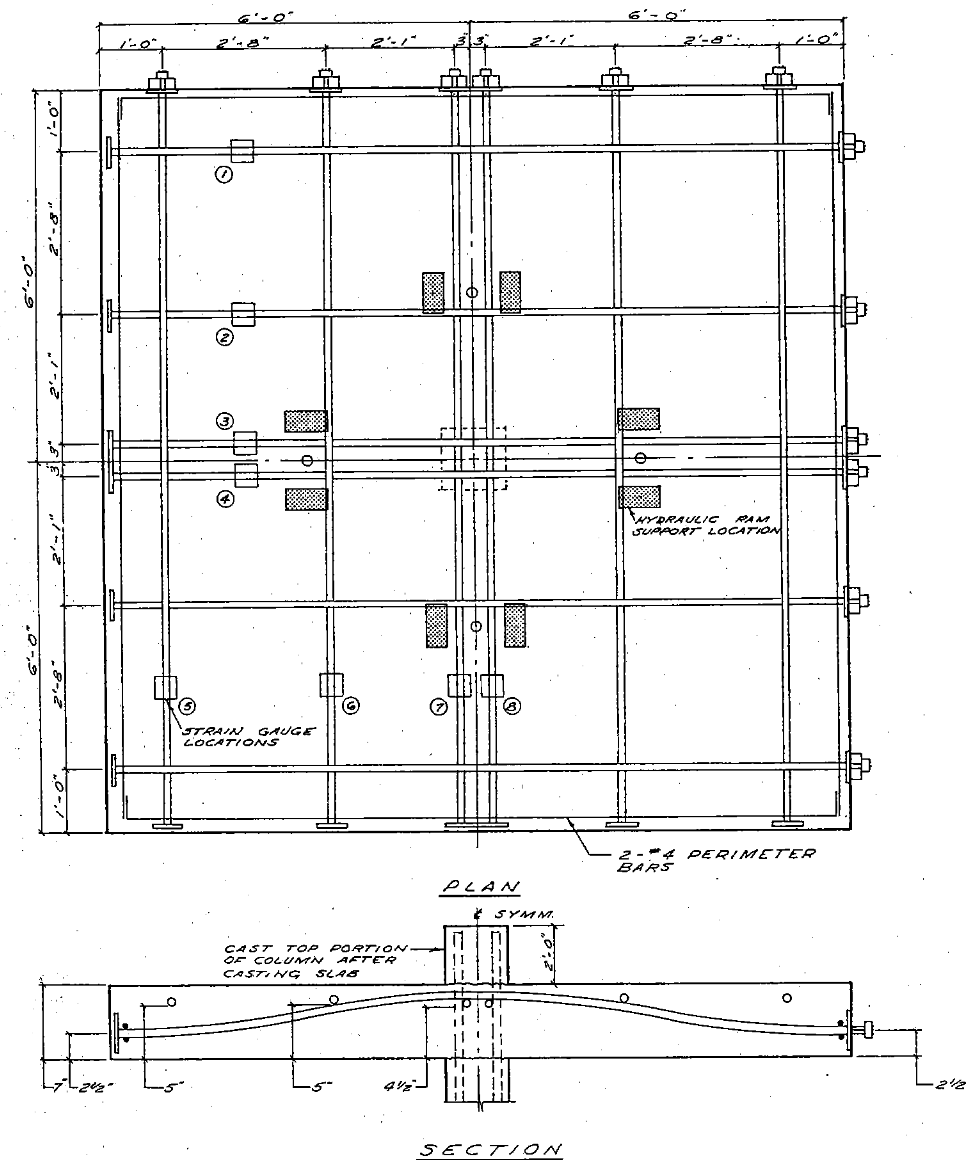

Fig. 3. Tendon layout for Specimen C-1 (without supplemental reinforcement)—typical of the cast-in-place slab specimens

Strain and deflection readings were taken at each increment of loading.

The load was then re-applied in increments of 20,000 lb. (9000 kgf) until a load of 160,000 lb. (73,000 kgf) was attained and thereafter in increments of 10,000 lb. (4500 kgf) until concrete failure occurred. The load at concrete failure is referred to as the primary failure load. After the primary failure occurred, the load was removed and re-applied until individual wires in the post-tensioning tendons commenced to break at which time the test was discontinued. The load at which wires commenced breaking is referred to as the maximum post-failure "reserve" load.

RESULTS

The loads resisted by the specimens at various conditions are recorded in Table 2, Lines 4 and 10. These should be compared with the total column design load of 94,000 lb. (42,600 kgf). The loads include 12.6 kips (5,700 kgf) of specimen weight. Load at which first cracking was detected was observed and recorded at the end of 20,000 lb. (9000 kgf) loading increments.

Primary failure in each specimen occurred as a combination of flexure and shear. For the cast-in-place specimens, the surface of failure extended conically from the perimeter of the column at the bottom surface of the slab to a diameter of approximately 6 ft. (1.8 m) at the top surface of the slab. The lift-slab specimens failed in like manner, with the failure surface commencing at the perimeter of the lifting collar at the bottom surface of the slab.

In each test, after the concrete had failed as described above, the slabs remained supported by the columns. This was due to post-tensioning tendons, as well as the supplementary reinforcing steel, that passed over the columns in the cast-in-place specimens and over the lifting collars in the lift-slab specimens, thereby preventing the slabs from falling after the primary concrete failure had occurred. As can be seen from the data in Table 2, all of the specimens were able to withstand post-failure "reserve" loads that were greater than the design load of 94,000 lb. (42,600 kgf).

None of the post-tensioning tendons failed at the primary load. Prestressing wire failure occurred at the "reserve" load as defined above. Wire failures at the reserve load were at the point of maximum curvature of the wire where the tendon was bending sharply as it passed over the lifting collar in the lift-slab specimens or over the column in the cast-in-place specimens. None of the wire failures occurred at the tendon anchorages and none of the anchorages was deformed in any way.

Strain measurements on the post-tensioning tendons revealed only a nominal stress increase in the tendons up to the primary failure load. At the design load of 94,000 lb. (42,600 kgf) the stress increase in the tendons ranged from about 1500 psi (106 kgf/cm^2) for the tendons nearest the center of the specimens. At primary failure loads, the stress increase in the tendons was approximately 4500 psi (317 kgf/cm^2) for the tendons near the edges and 9000 psi (633 kgf/cm^2) for the tendons nearest the column. In loading the specimens after primary failure, in order to determine the reserve capacity of the construction, the centermost tendons which passed over the column or lifting collar were strained into the plastic range, as would be expected, whereas the re-

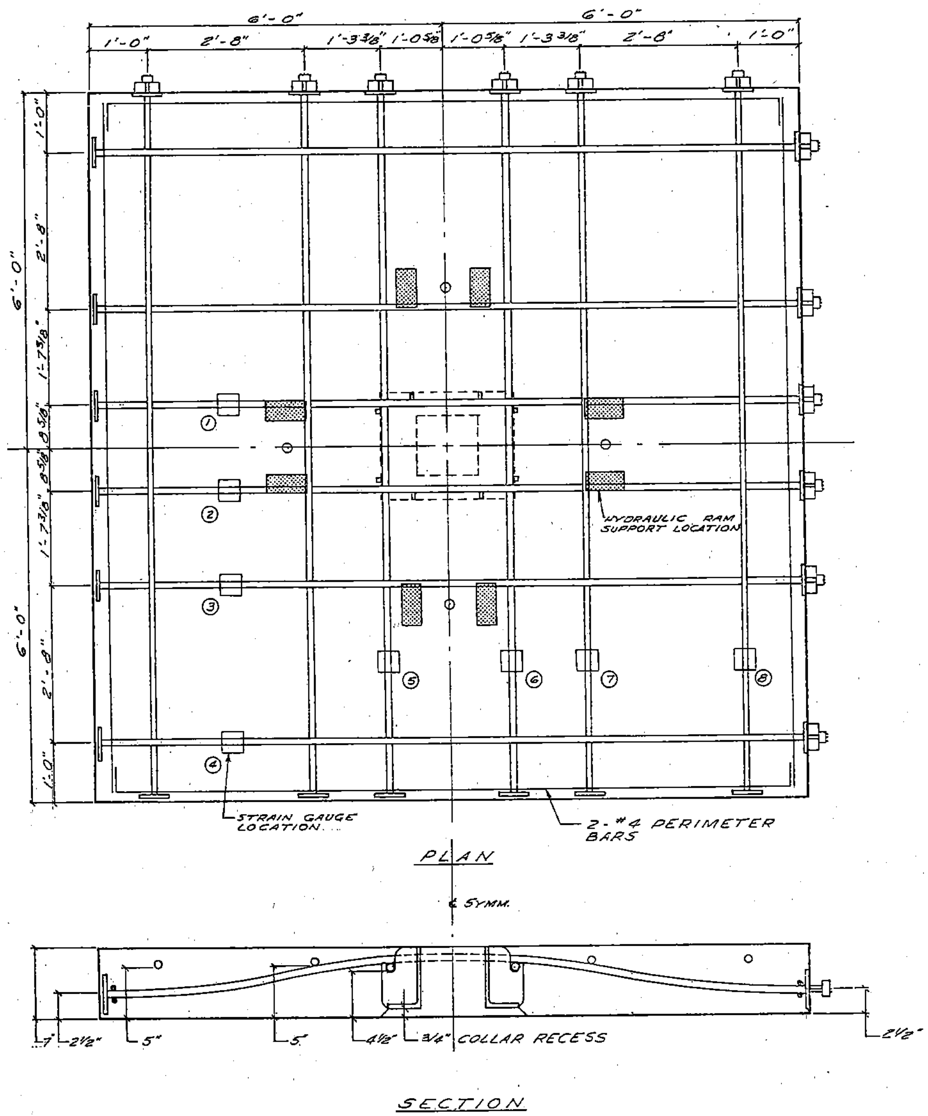

Fig. 4. Tendon layout for Specimen L-1 (without supplemental reinforcement)—typical of the lift-slab specimens

maining tendons revealed relatively nominal strain increases.

Strain measurements taken at the 94,000 lb. (42,600 kgf) design load on the supplementary reinforcing indicated stress increases in the order of 3500 to 9000 psi (246 to 633 kgf/cm^2) for bars placed in the top of the slab whereas at the primary failure load the stress increases were of the order of 20,000 psi (1400 kgf/cm^2) in lift-slab specimens and 30,000 to 40,000 psi (2109 to 2812 kgf/cm^2) in the cast-in-place specimens. In specimens C-4 and L-4, which had the supplementary reinforcing near the bottom of the slab, the increase in stress at the 94,000 lb. design load was nominal. At the primary failure load for specimen C-4, the increase in stress was in the order of 12,000 psi (840 kgf/cm^2) for the lower two bars and 20,000 (1400 kgf/cm^2) for the upper two bars. In specimen L-4 the upper two bars indicated a stress increase of 5000 psi (350 kgf/cm^2) while the lower two bars had an increase of about 900 psi (63 kgf/cm^2). This difference is attributed to the stiffening effect of, and reduced moment arm created by, the lifting collars.

A résumé of corner deflection readings is given in Table 2, Lines 5 through 9, for first and second cycles of loading at 100 kips (45,000 kgf), for 160 kips (73,000 kgf), and for the maximum jacking load at which deflection measurements were obtained.

DISCUSSION OF RESULTS

Shear capacity. While all specimens failed in a combination of flexure and shear, the shear capacity was reached before the flexural capacity in all cases.

Design methods and requirements for shear capacity in prestressed slabs are not well defined. Normal design procedure calls for limiting the punching shear under working loads to a value of between $0.04\, f'_c$ and $0.06\, f'_c$. In addition most designers will check the principal tension around the support (lifting collar,

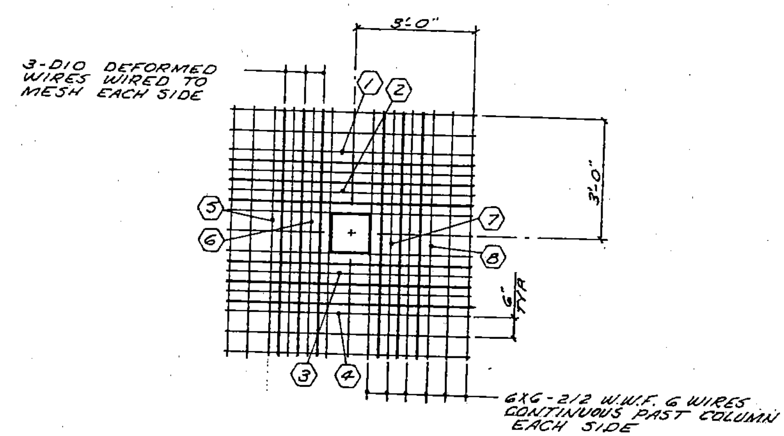

Fig. 5. Outline of supplemental mesh reinforcement used in Specimen C-3 (Specimen L-3 similar)

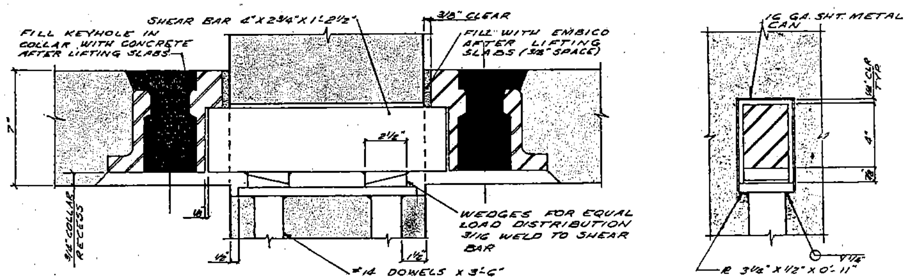

Fig. 6. Lift-slab to column connection

column or column capital) and limit this value to normal code limitations of $0.03\ f'_c$ or a maximum of 120 psi (8.4 kgf/cm²). Test values are listed in Table 2, Lines 10 through 14.

In addition to the above checks for working loads, building codes and ACI 318-63 require that the ultimate capacity of prestressed structures be checked. However, there are no ultimate shear design formulas in ACI 318-63 nor in local building codes for prestressed flat plates. Section 2610 of the ACI Code provides for beam design only. Consequently, the primary failure load must be compared to design formulas from other sources.

One source is a report on a series of tests run at the University of California at Berkeley in 1957 by T. Y. Lin, A. C. Scordelis and H. R. May. From these tests the authors developed the following empirical formula for the shear capacity of a prestressed flat plate:

$$\frac{P_u}{bd\,f'_c} = 0.175 - 0.0000242\ f'_c + 0.000020\ F_e/s$$

where P_u = ultimate shear capacity, kips
b = perimeter of lifting collar, column or column capital, in.
d = depth to post-tensioning tendons, in.
F_e/s = post-tensioning force, lb. per in.

The values in Table 2, Lines 15 and 16, for calculated ultimate shear are derived from this equation.

As seen from Table 2, Line 13, the minimum value of f_t/f'_c is 0.068. This represents a minimum factor of safety of 2.27 when compared to the limiting design value of $0.03\ f'_c$. The safety factor varies from 2.9 to 4.5, when computed on the basis of a limiting value of 120 psi (8.4 kgf/cm²). The average stress on the shear area ($d/2$ from face of column or edge of shear head) varied from $4.57\sqrt{f'_c}$ to $5.45\sqrt{f'_c}$ for specimens with conventional reinforcement (Table 2, Line 11). Specimen L-8 with added shear reinforcement carried $6.20\sqrt{f'_c}$. This is certainly an adequate factor of safety to allow one to conclude that current design procedure is sufficiently conservative (Table 2, Line 14).

The higher values of f_t/f'_c for the cast-in-place specimens as a group over the lift-slab specimens is thought to be due to two factors. The first consideration is in the determination of the shear perimeter. The perimeter for the lift-slab specimens is taken at the edge of the lifting collar. If the collar bends a small amount the failure plane is shifted back towards the column thereby decreasing the perimeter of the shear area. This reduced shear area would result in a higher value of f_t and consequently a higher value of f_t/f'_c. The other consideration is the fact that in the lift-slab speci-

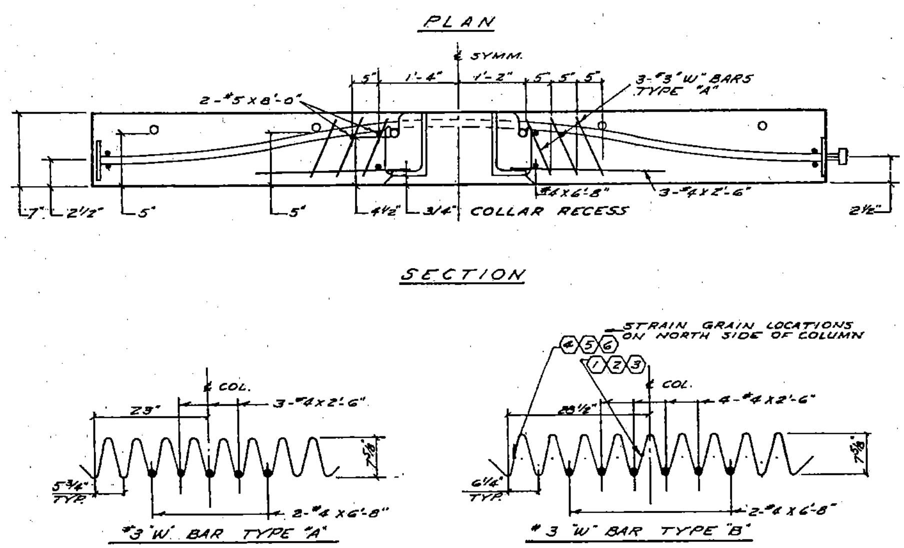

Fig. 7. Special shearhead reinforcement used in lift-slab specimen L-8

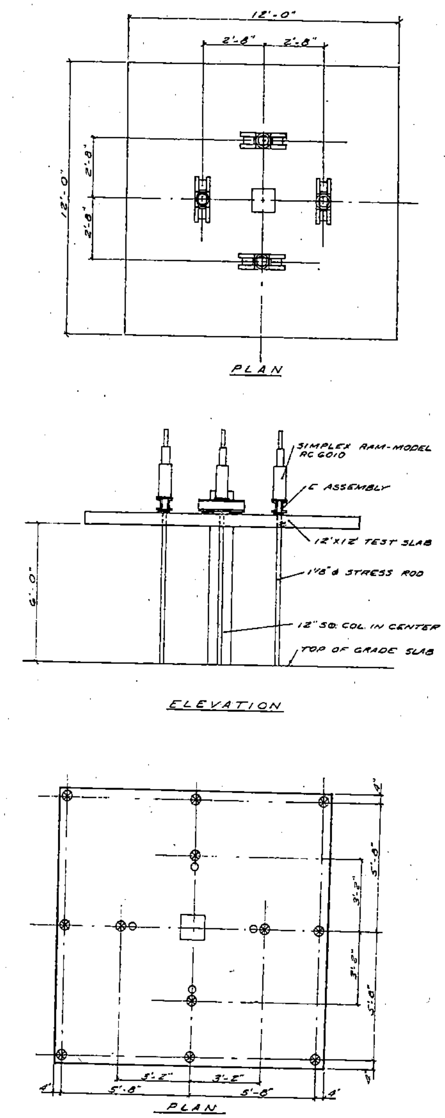

Fig. 8. Load application points and deflection indicator locations

mens, the concrete is free to pull away from the lifting collar at the top surface of the slab. This inability to develop tension and transfer moment is thought to reduce the value of f_t.

Table 2, Lines 15 and 16, lists test ultimate load (primary failure loads) and the ultimate shear loads predicted by the University of California test formula. The differences in the tabulated values of calculated ultimate shear capacity are due only to concrete strengths at time of testing. All of the other variables are constant for each group.

The empirical equation developed at the University of California makes no attempt to include effects of supplementary reinforcing steel around the support, nor will an attempt be made here except for a few qualitative comments. Table 2 indicates that the specimens with welded wire fabric performed better than others in their respective groups (with the exception of L-8). It is believed that the mesh controlled the cracking due to flexure more readily than the eight No. 5 (16 mm) bars, thereby providing greater shear capacity. Specimens with four No. 9 (29 mm) bars at the bottom of the slab performed no better in this respect than those with no reinforcement because the steel was too low to resist any of the cracking caused by flexure. Specimen L-8 performed well as was to be expected. The increase in ultimate capacity over L-1 (no supplementary reinforcing) is 79.4 kips (36,000 kgf). This is an example of the increased shear capacity which can be built into a slab if the situation is warranted.

The reasons for the lower values of R_{ult} in Table 2, Line 16, for the lift-slab specimens, are felt to be the same as discussed above for lower values of f_t/f'_c, namely, reduced effective perimeter and moment capacity.

Flexural capacity. All of the specimens tested failed due to principal tension around the support. None exhibited flexural failures. It is meaningless then to compare calculated ultimate capacity with tested values other than to conclude that the loads at which principal tension failures occurred produced moments exceeding those calculated. For specimens C-1 and L-1 (no supplementary reinforcing) the ultimate moment capacity as determined by ACI 318-63, Section 2608, is 126 ft.-kips (17,400 m-kgf) for the 12-ft. (3.7 m) specimen width. This is based on 5300 psi (372 kgf/cm^2) concrete and a value of 192 ksi (13,500 kgf/cm^2) for the ultimate prestressing wire stress as opposed to the 159 ksi (11,200 kgf/cm^2) value (144 + 15) specified in Section 2608. The actual moments at time of principal tension failure were 154 ft.-kips (21,300 m-kgf) for C-1 and 170 ft.-kips (23,500 m-kgf) for L-1, well above the calculated ultimate moments. Calculations for the above values are in the Appendix.

Current design practice allows tension in the concrete during maximum design moment loading conditions. One criterion allows a maximum value of $3\sqrt{f'_c}$ or 189 psi (13.3 kgf/cm^2) for 4000 psi (281 kgf/cm^2) concrete. These specimens were proportioned to have a value of 190 psi (13.4 kgf/cm^2) tension in the concrete due to the moment produced by the 94 kip (42,600 kgf) column design load. Hair-line cracks radiating from the corners of the slab supports did occur in the top surface of the slab at loads approximately equal to the 94 kip design load. These hair-line cracks were due, no doubt, to concentrated stresses at the corners much higher than the calculated value of 190 psi (13.4 kgf/cm^2). However, as a guide to ultimate capacity, it is felt safe to conclude that allowing reasonable values of concrete tensile stresses is an acceptable design procedure.

Reserve capacity. The term "reserve capacity" as used here refers to the ability of the specimen to sustain load after the primary failure has occurred. The primary failure, or ultimate capacity if it occurs, would normally be brought about by a sudden, unsustained overload condition. The importance of the reserve capacity comes into play after ultimate failure has occurred, and the loading on the specimen returns to a value equal to or less than the design load. If the structure does possess adequate reserve capacity the chances of a total collapse are minimized.

After the primary failure load had occurred, the load was removed and reapplied until individual wires in the prestressing tendons started to break. The specimen was unable to support additional load at this point and the test was stopped. This point is referred to as the maximum post-failure "reserve" load.

Reserve capacity was attained by running the two center tendons over the column in the cast-in-place specimens and hooking them over the lifting collars in the lift-slab specimens. In specimen C-4 the No. 9 (29 mm) bars passed over the column in compliance with Los Angeles city code requirements and the tendons were outside the column to avoid interference with these bars. Specimen L-4 had tendons hooked over the collar and the No. 9 (29 mm) bars passing through holes in the collar in compliance with the Los Angeles requirements.

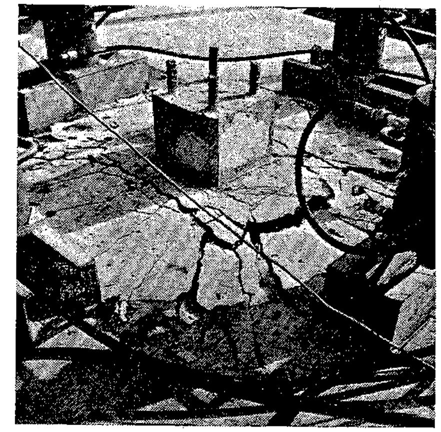

Fig. 9. Specimen L-8 at 184 kip (83,600 kgf) load, after primary failure load had occurred—maximum post-failure reserve load was 242 kips (110,000 kgf)

As seen from Table 2, Lines 17 and 18, all specimens supported reserve loads greater than the 94 kip (42,600 kgf) design load. The ability of the specimens to do this was undoubtedly due to the tendons hooking over the columns or collars, thereby preventing the specimens from exhibiting a complete collapse. The four No. 9 (29 mm) bars passing over the column in lieu of the tendons in Specimen C-4 had the same effect as the tendons in the other specimens.

Specimens without supplementary reinforcement carried the smallest reserve load in their respective groups. There is no pattern of reserve capacity for the other specimens, with the exception of L-4 and L-8. Specimen L-4 was able to carry a higher reserve load due to both the No. 9 (29 mm) bars and the tendons resisting the collapse of the specimen. Specimen L-8 (Fig. 9) supported the highest reserve load due to the contribution of the No. 3 (10 mm) W-bars. These bars, crossing the failure plane, worked with the tendons into the plastic range.

The fact that the tendons passed under the ram support assemblies is felt to have helped to a certain degree in preventing a total collapse of the test slabs in that the tendons were not free to pull or tear out of the top surface of the slab. However, in an actual floor system, the effect of bending would be more pronounced, resulting in a smoother deflected slab configuration, thereby minimizing the tendency of the tendon to pull out of the slab and also avoiding the sharp bend in the tendon as it passes over the supporting column or lifting collar. Also, the effect of the two-way tendon system or "mat" of tendons tied together in a full size structure would tend to eliminate this pull-out effect. These characteristics of the full size structure should result in reserve capacities at least equal to that obtained in the tests.

General. Although deflection measurements were taken throughout the loading cycle there may be little to be learned from them. The specimens were loaded in a manner resulting in little bending. As the specimens approached their maximum reserve loads they began to tilt due to the yielding of the prestressing steel and the conventional reinforcing steel. This tilting or unsymmetrical loading occurred at the end of the loading cycle and consequently had little effect on the maximum reserve load carried by the specimens.

The deflection measurements did show the full recovery of the specimens after the first 100 kip (45,000 kgf) loading cycle. Also, they indicated the more plastic or ductile nature of the specimens having supplementary reinforcing steel. In addition to the recovery after the first 100 kip loading cycle, the specimens exhibited approximately an 80 percent recovery even after the maximum reserve load had been removed and some prestressing wires were broken. This indicates the large strain energy characteristics of the unbonded system of post-tensioning.

Strain measurements on the prestressing tendons confirm the fact that the tendons nearest the column lines are subject to higher increases in strain due to the external loading than are tendons near the edges of the specimens. The strain measurements obtained are not considered to be quantitatively significant due to the size of the specimens, the manner of applying the jack loads, and the resulting deformations of the specimens.

Strains measured on the supplementary reinforcing located in the top of the slabs revealed that it is active in resisting stresses resulting from applied loads. The cracking of the specimens observed during the tests illustrated the efficiency of supplementary reinforcing steel in restricting crack width. Specimens C-1 and L-1, which did not have any supplementary reinforcing, and specimens C-4 and L-4, which had supplementary reinforcing near the bottom of the specimens, all exhibited cracks of relatively great width.

Specimens C-3 and L-3 which had the welded wire fabric and deformed wire reinforcement exhibited the narrowest cracks. The specimens having No. 5 (16 mm) bars in the top surface of the slab for supplementary reinforcing (C-2, L-2 and L-8), developed moderately wide cracks at maximum load. Specimen L-7, which had grouted tendons with

no supplementary reinforcing, sustained cracks of the same general character and width as was found for the specimen having non-bonded tendons and No. 5 (16 mm) bars for supplementary reinforcing.

Specimen L-7 with bonded tendons performed in a manner similar to that of the other specimens. It is impossible to draw any definite conclusions from this specimen's performance although it appears that the bond achieved by the grouting did increase the maximum reserve capacity somewhat over the other specimens without supplementary reinforcing. Other characteristics such as deflection and recovery were similar to the unbonded specimens.

Although these tests were not designed to test or evaluate the prestressing anchorage system, it is interesting that even under severe loading conditions there were no signs of distress in either the buttonheads or the hardware itself, indicating a high degree of anchorage reliability. Neither the lifting collar, which was designed for 100 kips (45,000 kgf), nor the supporting insert in the precast column showed any significant signs of stress or deformation. Only in specimen L-8 (primary failure load of 276.6 kips or 125,000 kgf) did the horizontal legs of the angles show any signs of deflection.

CONCLUSIONS

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

1. Current shear design practice for post-tensioned flat plates is adequately conservative. Limiting design values of principal tension to 120 psi (8.4 kgf/cm^2) results in safety factors in the order of 2.9 to 4.5, depending on supplementary reinforcing steel used.
2. Based on average shear at sections $d/2$ away from face of column or shear head, the primary failure load ranged from $4.1\sqrt{f'_c}$ to $6.2\sqrt{f'_c}$ with different flexural reinforcement details.
3. Special forms of supplementary reinforcing steel configurations can be used to greatly increase the ability of post-tensioned flat plates to transmit loads to a column.
4. Supplementary reinforcing steel will help control cracking and increase the structure's ductility.
5. Passing post-tensioning tendons over the column or lifting collar results in the connection having a reserve capacity or an ability to transmit load to the column after failure of the concrete adjacent to the column or lifting collar.

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APPENDIX

The slab is cracked at ultimate and therefore would not behave elastically. However, one way to compare the ultimate performance levels for the test specimens is to use the elastic formula for principal tension stress at an index value. The following sample calculations illustrate the computation method used to obtain the "principal tensile stress index values" (f_t) presented in Table 2, Line 13.

Principal tension at failure

$$f_t = \sqrt{v_t^2 + \left(\frac{f_c}{2}\right)^2} - \frac{f_c}{2}$$

where $v_t = 1.5\, v_p$

v_p = punching shear at a distance of $d/2$ from the face of the support

f_c = average compression in the concrete due to prestressing

For Specimen C-1:

Primary failure load = 177.2 kips

$$\text{Shear perimeter} = \left[12 + \left(\frac{7}{2}\right)2\right] \times 4 = 76 \text{ in.}$$

$$v_p = \frac{177.2}{76 \times 7} = 333 \text{ psi}$$ (Average stress on shear perimeter $= 4.56\sqrt{f'_c}$)

$v_t = 1.5 \times 333 = 500$ psi

$f_c = 250$ psi

$$f_t = \sqrt{500^2 + \left(\frac{250}{2}\right)^2} - \frac{250}{2} = 515 - 125 = 390 \text{ psi}$$

For Specimen L-1:

Primary failure load = 197.2 kips

$$\text{Shear perimeter} = 2\left(25.75 + \frac{5.5}{2} \times 2\right) + 2\left(20.75 + \frac{5.5}{2} \times 2\right)$$

$$= 2\,(31.25) + 2\,(26.25) = 115 \text{ in.}$$

(5.5 in. equals distance from top surface of slab to top surface of horizontal angle)

$$v_p = \frac{197.2}{115 \times 5.5} = 312 \text{ psi}$$ (Average stress on shear perimeter $= 4.28\sqrt{f'_c}$)

$v_t = 1.5 \times 312 = 468$ psi

$f_c = 250$ psi

$$f_t = \sqrt{468^2 + \left(\frac{250}{2}\right)^2} - \frac{250}{2} = 483 - 125 = 358 \text{ psi}$$

Calculated ultimate shear

$$\frac{P_u}{bd f'_c} = 0.175 - 0.0000242\, f'_c + 0.000020\, F_e/s$$

where: P_u = ultimate shear capacity, kips

b = perimeter of lifting collar, column or column capital, in.

d = depth of post-tensioning tendons, in.

F_e/s = post-tensioning force, lb. per in.

f'_c = concrete strength, psi

For Specimen C-1:

$b = 4 \times 12 = 48$ in.

$d = 7 - 1\frac{3}{4} = 5\frac{1}{4}$ in.

$f'_c = 5300$ psi

$$F_e/s = \frac{6 \times 42}{12 \times 12} = 1750 \text{ lb. per in.}$$

$$\frac{P_u}{48 \times 5.25 \times 5300} = 0.175 - 0.0000242 \times 5300 + 0.000020 \times 1750$$

$P_u = 109{,}500$ lb.

For Specimen L-1:

$b = 2(25.75 + 20.75) = 93$ in.

$d = 5\frac{1}{4}$ in. $- \frac{3}{4}$ in. recess $- \frac{1}{2}$ in. angle $- \frac{1}{4}$ in. offset $= \frac{3}{4}$ in.

$f'_c = 5300$ psi

$F_e/s = 1750$ lb. per in.

$$\frac{P_u}{93 \times 3.75 \times 5300} = 0.175 - 0.0000242 \times 5300 + 0.000020 \times 1750$$

$P_u = 151{,}600$ lb.

Flexural capacity

Ultimate flexural capacity per Section 2608, ACI 318-63:

$$M = \phi[A_s f_{su} d\,(1 - 0.59\,q)] = \phi\left[A_s f_{su}\left(d - \frac{a}{2}\right)\right]$$

where: M_u = ultimate flexural capacity

ϕ = capacity reduction factor

A_s = area of prestressing steel

f_{su} = ultimate stress in prestressing steel

d = distance from extreme compression fiber to centroid of prestressing force

$a = A_s f_{su}/0.85 f'_c b$

f'_c = concrete strength

b = width of compression face

Calculated M_u for Specimens C-1 and L-1:

$\phi = 0.90$

$A_s = (6 \times 0.049)\,6 = 1.76$ in.^{2} for 12 ft. width

$f_{su} = 192$ ksi

$d = 5\frac{1}{4}$ in.

$$f'_c = 5300 \text{ psi}$$

$$b = 12 \times 12 = 144 \text{ in.}$$

$$a = \frac{1.76 \times 192}{0.85 \times 5.3 \times 144} = 0.52$$

$$M_u = \frac{0.90}{12}\left[1.76 \times 192\left(5.25 - \frac{0.52}{2}\right)\right] = 126 \text{ ft.-kips}$$

Actual M_u at primary failure, Specimen C-1:

$$M_u = \text{slab dead load moment} + \text{jacking load moment}$$

$$= \left(\frac{0.088 \times 36}{2}\right) \times 12 + \left(\frac{177.2 - 12.6}{4}\right)(2.67 + 0.61)$$

$$= 19.0 + 135.0 = 154.0 \text{ ft.-kips}$$

Actual M_u at primary failure, Specimen L-1:

$$M_u = \left(\frac{0.088 \times 36}{2}\right) \times 12 + \left(\frac{197.2 - 12.6}{4}\right)(2.67 + 0.61)$$

$$= 19.0 + 151.0 = 170.0 \text{ ft.-kips}$$

Discussion of this paper is invited. Please forward your comments to PCI Headquarters by March 1 to permit publication in the March-April 1972 issue of the PCI JOURNAL.