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TEST OF A REINFORCED CONCRETE FLAT SLAB

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The tests on the structure described herein were a part of the investigation of multiple-panel reinforced concrete floor slabs (6)[4] conducted in the Structural Research Laboratory of the University of Illinois Civil Engineering Department. The test structure was a nine-panel flat slab, designated F2, and was one of five quarter-scale structures in the program. It followed F1, a flat plate (4).

Structure F2 (Figs. 1 and 2) was representative of ordinary flat slab construction. The background of the investigation and the relationship of F1 to it have been presented elsewhere.

DESCRIPTION OF TEST STRUCTURE

Design.—The test structure was scaled from a flat slab with 20-ft square panels designed according to the "empirical design method" of section 1004 of ACI 318-56 (1). It was designed for a live load of 200 psf and a dead load of 85 psf. The spandrel beams were designed for a wall load of 600 lb per ft in addition to the slab load. The design stresses were 1,350 psi for the concrete and 20,000 psi for the reinforcement. All details of the structure were in accordance with ACI 318-56. All dimensions of the test structure were 1/4 of the structure as designed.

Dimensions.—The test structure is shown in plan and in section in Fig. 1. Column, panel, and section designations are shown on the plan. The structure was square in plan with three 5-ft spans in each direction. Two adjacent edges

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[4] Numerals in parentheses refer to corresponding items in the Appendix I.—References.

of the structure were supported by deep beams, 2 × 6 in., and the other two edges were supported by shallow beams, 4-1/2 in. × 2-1/2 in. Dimensions of columns, drop panels, and column capitals are shown in Fig. 5.

The lower ends of the columns were pinned and the length of the columns was chosen so that their stiffness would be comparable to the columns of the original structure in which the columns extended above as well as below the slab.

Reinforcement.—The slab was reinforced with 1/8-in. square bars. The placing of both top and bottom slab reinforcement is shown in Fig. 3.

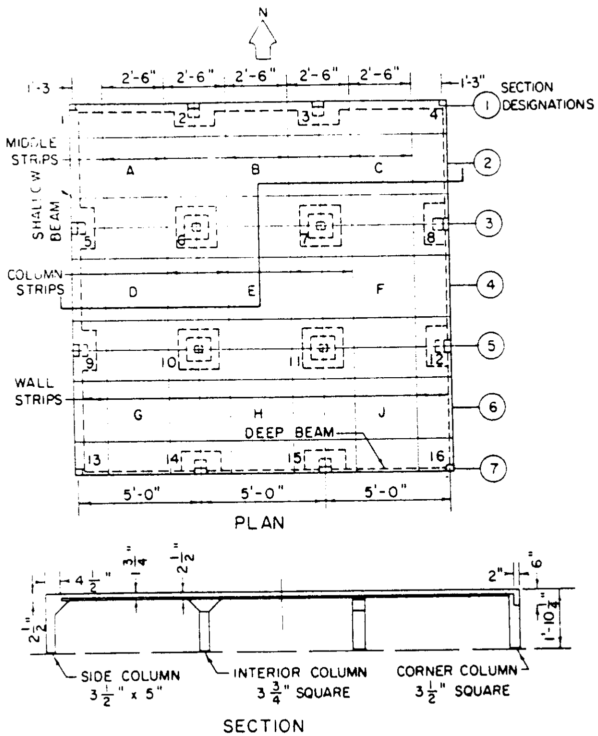

FIG. 1.—PLAN AND SECTION OF TEST STRUCTURE F2

The main reinforcement in beams and columns was 1/4-in. plain round bars. Column ties and stirrups were cut from 1/8-in. square bars. Beam and column reinforcement is shown in Figs. 4 and 5, respectively.

MATERIALS AND CONSTRUCTION

Reinforcement.—The slab reinforcement was cut from 1/8-in. square undeformed bars of cold finished B1113 steel which had been annealed at 1,300°. The average yield stress from 52 tests was 42,000 psi. The stress-strain diagram had the shape characteristic of intermediate grade steel. Strain hardening began at about .035.

The beam and column reinforcement consisted of 1/4-in. round undeformed hot-rolled bars. The yield stress was 49,000 psi based on four tests with no measurable scatter. Strain hardening began at about 0.035 and the ultimate stress was 67,000 psi at a strain of approximately 0.2. Ties and stirrups were cut from the same stock as the slab reinforcement.

All bars were washed with a 50% solution of hydrochloric acid and left in a moist room for two weeks to accelerate rusting. Loose rust was removed with a wire brush, and the resulting surface was slightly pitted and had improved bond characteristics.

Concrete.—A small aggregate concrete consisting of Type I portland cement, a blend of 80% coarse Wabash River sand and 20% fine lake sand, and water was

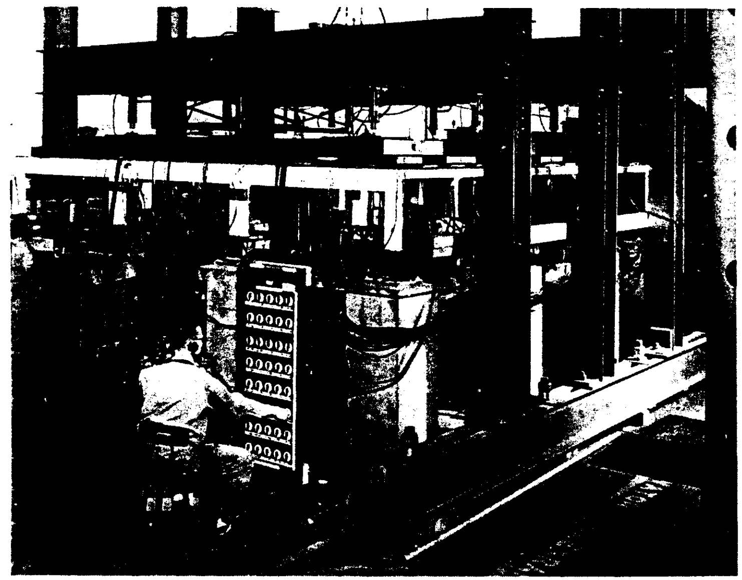

FIG. 2.—PHOTOGRAPH OF THE FLAT SLAB SHOWING THE READING STATION

used. The water/cement ratio was 0.72 and the aggregate/cement ratio was 5.7. The maximum aggregate size was 1/8-in. and the fineness modulus of the blended aggregate was 2.8.

2-in. × 4-in. control cylinders were used. The average compressive strength of the control cylinders was 2,760 psi at 78 days (28 tests) and 2,320 psi at 168 days (12 tests). The initial tangent modulus at 78 days was 2.1×10^6 psi (9 tests) and the modulus of rupture, determined from 2-in. wide by 1.75-in. deep beams load at the third points of a 15-in. span, was 602 psi (10 tests).

TEST EQUIPMENT

The loading system, load and reaction measurement devices, and strain recording equipment were the same as those used for structure F1 and have been described elsewhere (3,4,5).

Each panel was loaded through 16 symmetrically located 8-in. square pads

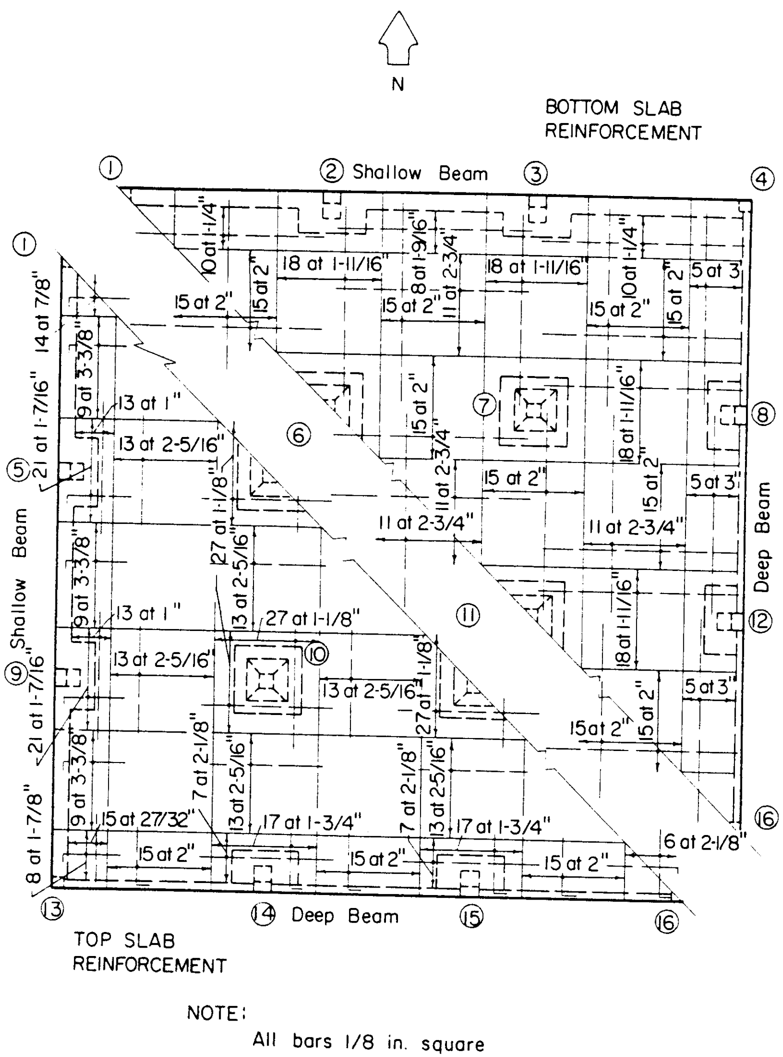

FIG. 3.—SLAB REINFORCEMENT

to simulate a uniformly distributed load. Each of the 16 columns supported by a dynamometer calibrated so that the magnitude and direction of the reactions could be determined.

Strains in the reinforcement were measured at 335 locations with electrical

resistance strain gages. Deflections were measured at all panel centers and at the midpoints of all column center lines.

TEST PROGRAM

Chronology.—Structure F2 was tested between February 20 and May 19, 1959. The 38 tests can be divided into four categories: (1) Tests 200-207 at loads

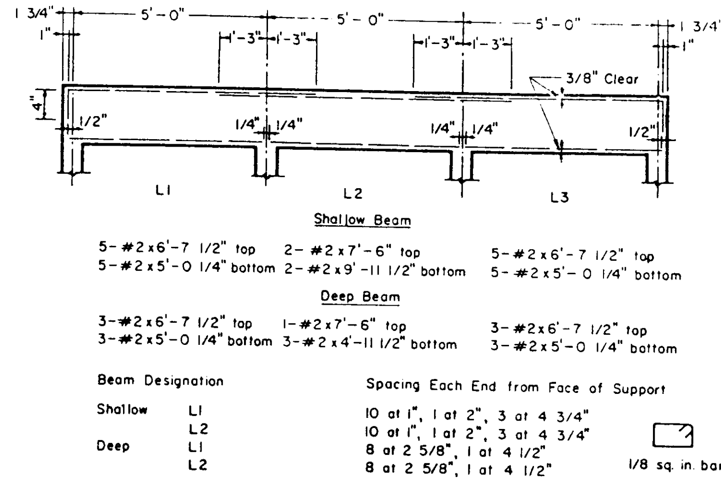

FIG. 4.—ARRANGEMENT OF BEAM REINFORCEMENT

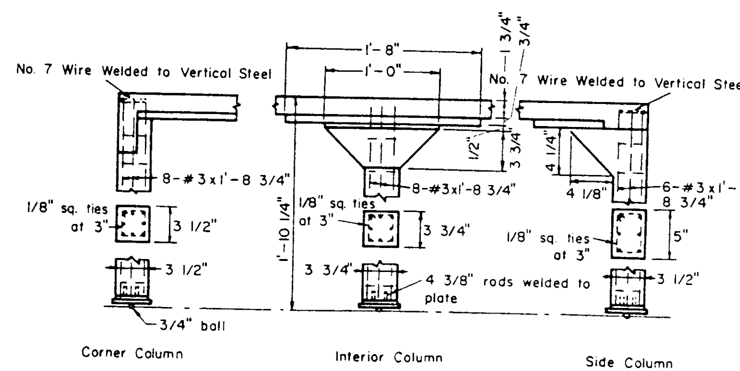

FIG. 5.—ARRANGEMENT OF COLUMN REINFORCEMENT

below the estimated cracking loads; (2) tests 208-220 at service load levels; (3) tests 221-233 under overload; and (4) tests 234-237 to failure. The first test of each of the four categories was one in which all panels of the structure were loaded. In other tests in each category, only part of the panels were loaded in various patterns.

Tests 208, 221, and 234, the first tests of categories (2), (3), and (4), are reported here.

Test Procedure.—In each test the load was applied in one or more increments and data were read and recorded after each increment. As each load increment was applied, the magnitude of load was monitored by one of the load dynamometers.

All strain and deflection gages were read before the first load increment. Load and reaction dynamometers were read after loading was stopped in each increment and again before loading was resumed in the next increment. This

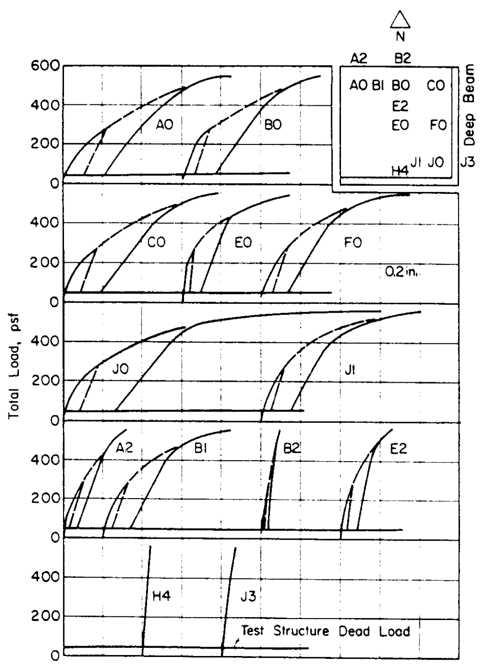

FIG. 6.—LOAD-DEFLECTION CURVES

interval was approximately 30 min to 40 min during which time strain and deflection gages were read, and the structure was inspected for cracks. After the maximum load was reached and all readings recorded, the applied load was removed in one increment and a final set of readings recorded.

Reported Data.—The response of the structure to load was evaluated by the measurement of deflections and reinforcement strains, and by the observation of cracking. These observations are reported for the following three tests in which all panels were loaded uniformly: (1) test 208, total load—280 psf; (2) test 221, total load—399 psf (nominally $DL + 1.5LL$) and (3) test 234, to failure. The dead load of the structure plus the loading system was 44 psf. In each of these

tests the given load had been reached for the first time so that the observed phenomena were not critically affected by pattern loadings at lower loads in previous tests. All quantities reported in the following sections are cumulative and include dead load and residual effects.

BEHAVIOR UNDER DESIGN LOAD

Deflections.—Representative load-deflection relationships are shown in Fig. 6. The first solid portion of each curve represents response in test 208 in loading and unloading. The second solid portion indicates the response in test 234 and the shift to the right of the origin of this portion corresponds to the amount of cumulative deflection from the end of test 208 to the beginning of test 234. The broken part of each curve indicates the response if the structure had been loaded in one test to failure. Curves for test 221 are not shown. The horizontal line at 44 psf in each diagram is the dead load of the structure and loading sys-

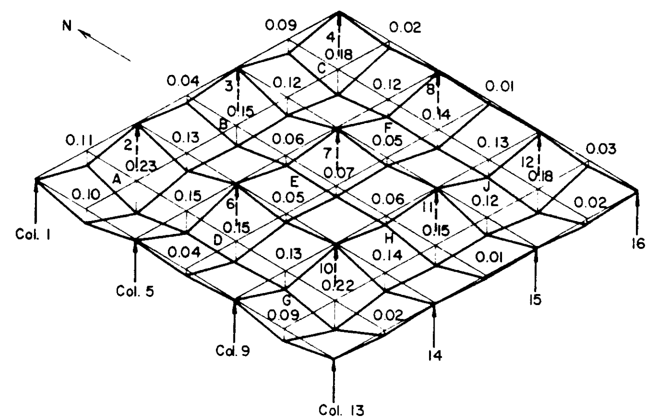

FIG. 7.—DEFLECTIONS AT 280 PSF

tem. The dead load deflections were estimated from deflections measured in tests at loads below the estimated cracking loads. The location on the structure which each curve represents is shown in the upper right of the figure.

The load-deflection curves were linear in test 208 up to about 90 psf which was the maximum load previously applied to the structure. Beyond that point curves AO, CO, and JO, representing the response at the middle of the corner panels, became nonlinear. At a load of 200 psf, only the curves for the deflection at beam midspans were still linear. These curves remained linear throughout the test. The load-deflection curves for the slab, especially those at midpanels, show a marked reduction in stiffness of the slab.

The deflections at maximum load can be compared in Fig. 7. The maximum deflection occurred at the center of panel A, the corner panel bounded by shallow beams and the minimum midpanel deflection was in panel E, the center panel. The deflection at the center of panel C was about $L/260$ and that at the center of panel E was about $L/860$. The generally smaller deflections of panels

CFJ and GHJ verify the greater stiffness of those exterior panels supported by deep beams.

Reinforcement Stresses.—The stresses for the negative-moment sections 1,3,5 and 7 are shown on Fig. 8 and those for positive-moment sections 2,4, and 6 are shown on Fig. 9.

At 290 psf, the highest stresses occurred at the interior column lines, sections 3 and 5. Stresses near 25 ksi occurred at a number of locations with the maximum stress about 30 ksi. At positive moment section 2 the stresses were about 18 ksi at a number of locations but at section 4 stresses on the

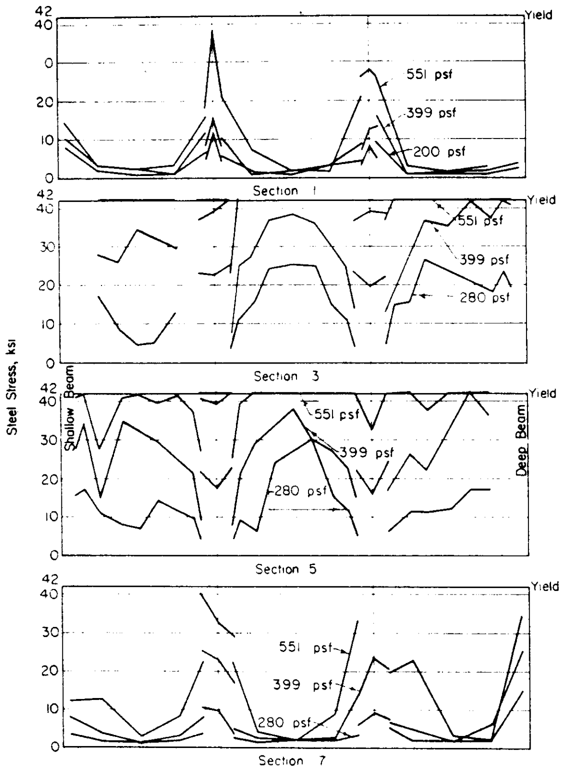

FIG. 8.—REINFORCEMENT STRESS DISTRIBUTION NEGATIVE MOMENT SECTIONS

order of 5 ksi were more characteristic. At sections 1 and 7 the highest stresses were about 10 ksi and occurred at the capital faces. Stresses away from the columns at these sections were quite low.

Cracking.—The first cracks observed in the flat slab, after careful examination with a 7-power magnifying lens, were at the exterior faces of the interior column capitals at a total load of 158 psf. Next, at a load of 203 psf, cracks were found at the edges of the interior drop panels. With successive load increments cracks formed along column lines on the top of the slab, be-

ginning at the drop panels and progressing away from the column. The crack pattern on the top of the slab at 290 psf is shown in Fig. 10.

The bottom of the slab was fairly extensively cracked in this test. The cracks were parallel to the edge at about the middle third of the exterior panels.

BEHAVIOR IN TEST 221 (1.0 DL + 1.5 LL)

Between tests 208 and 221 a series of tests with panels loaded in various patterns was conducted with a nominal maximum load of 285 psf. These tests caused extensive cracking on both the top and bottom of the slab. At least one

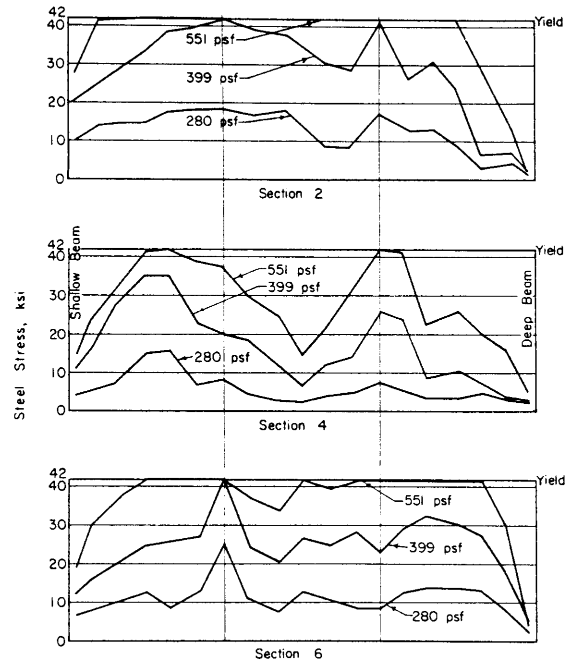

FIG. 9.—REINFORCEMENT STRESS DISTRIBUTION POSITIVE MOMENT SECTIONS

crack, and in most cases several parallel cracks, extended along the full length of each interior column line. There were also cracks at each corner column. The middle third of each row of panels had numerous cracks parallel to the panel centerlines. The exterior rows were more severely cracked than the interior rows. In test 221 the maximum total load was 399 psf. This corresponded to approximately DL + 1.5 LL.

Deflections.—The deflections at the maximum load in test 221 are shown in Fig. 11. The maximum deflection was 0.46 in. at the center of panel A. This is $L/130$. The deflection at the center of the interior panel was 0.18 in. or $L/330$.

Reinforcement Stresses.—The stresses in the reinforcement in test 221 are shown in Figs. 8 and 9. The yield stress was reached at a number of locations in this test, but the exterior edges, sections 1 and 7 were still characterized by very low stresses, generally less than 5 ksi, away from the columns.

Cracking.—The cracks observed on the top and bottom of the slab are shown in Figs. 12 and 13. Few new cracks were observed on the top of the slabs and most of the new cracks on the bottom of the slab were in the interior rows of panels parallel to the panel centerlines.

BEHAVIOR IN TEST TO FAILURE

The structure was tested to failure in test 234. Between tests 221 and 234 the structure was tested with a series of pattern loadings at a load of approx-

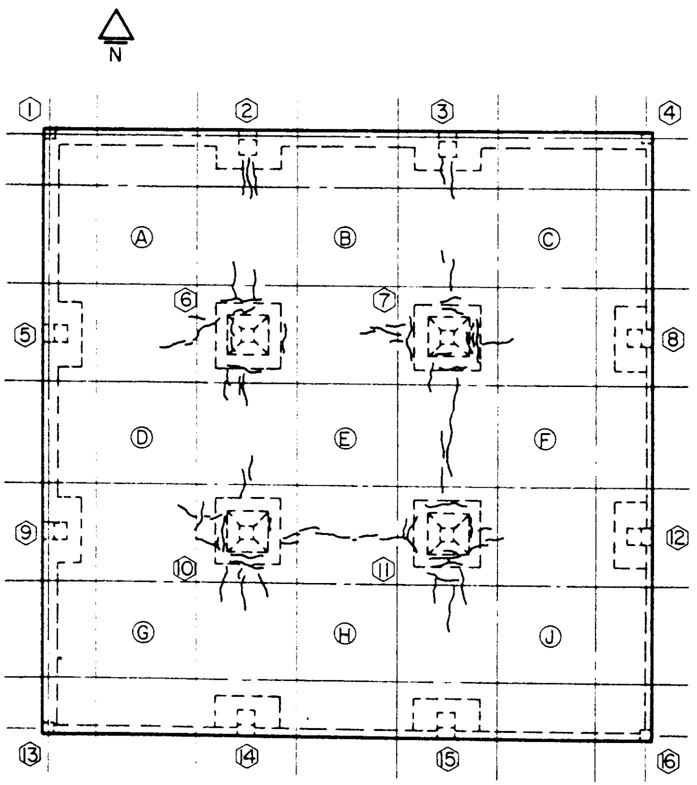

FIG. 10.—TOP CRACK PATTERN AT DESIGN LOAD

imately 385 psf. These tests caused further development and widening of the cracks. Also, the repeated strip loadings had so severely damaged the side columns that it was necessary to provide external prestressing before the test to failure. This was done by tightly binding wood blocks below the columns to

confine the concrete and providing a 10,000 lb force at the exterior edges of the columns by means of clamps.

The south bay of the structure, comprising panels G, H, and J, failed at a load of 551 psf.

Deflections.—Representative load-deflection curves for test 234 are the second solid portions of the curves in Fig. 6. Most of the curves are linear up to a load of 399 psf which was the maximum load prior to test 234. Beyond that point the deflections, with the exception of those at locations H4 and J3, increased greatly with each small increase in load.

Deflections at failure are shown in Fig. 14. The large deflections in panels G, H, and J in which failure occurred are clearly evident as are the very small deflections of the deep beams.

Reinforcement Stresses.—General yielding of the reinforcement had occurred in sections 3 and 5 (Fig. 8) and in sections 2 and 6 (Fig. 9) when the failure load was reached. However, in section 4 (Fig. 9) only local yielding had occurred and the stresses at the center of the interior panel were still

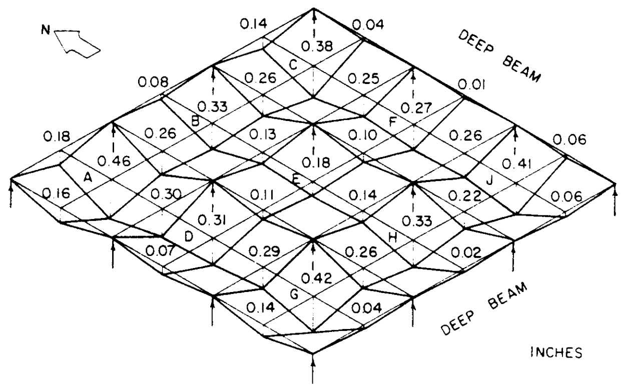

FIG. 11.—DEFLECTIONS AT 399 PSF

relatively low. At sections 1 and 7 there was only local yielding and reinforcement stresses away from the columns were low because of the low torsional strength and rigidity of the spandrel beams.

There was local yielding in sections 2, 3, and 5 during the first load increment of test 234 and in section 6 during the second load increment. The local yielding in section 7 occurred at the failure load.

Cracking.—The cracks on the top and bottom of the slab at the end of test 234 are shown in Figs. 15 and 16, respectively. The wider cracks are indicated by wider lines. Up to 472 psf, the only noticeable change in the structure was the widening of previously existing cracks. At 519 psf torsional distress in the deep beams at the south edge of the structure was quite pronounced. Torsional damage was less severe in other spandrel beams.

Up to the failure load very few new cracks were observed on the top of the slab and further cracking on the bottom consisted only of the extension of previous cracks and the formation of new cracks parallel to previous cracks further from midspan.

Failure.—At failure of the south bay of the structure, a very wide crack developed on the top of the slab at the south edge of the capitals of columns 10 and 11 extending midway between columns 9 and 10 to midway between columns 11 and 12. At the exterior side of the south bay, the deep beams twisted out of corner columns 13 and 16, and columns 14 and 15 failed with very large rotation at the bottom of the capital.

MEASURED BENDING MOMENTS

Moments in the structure were calculated from strains measured in the reinforcement. An idealized bilinear moment-strain relationship was used in

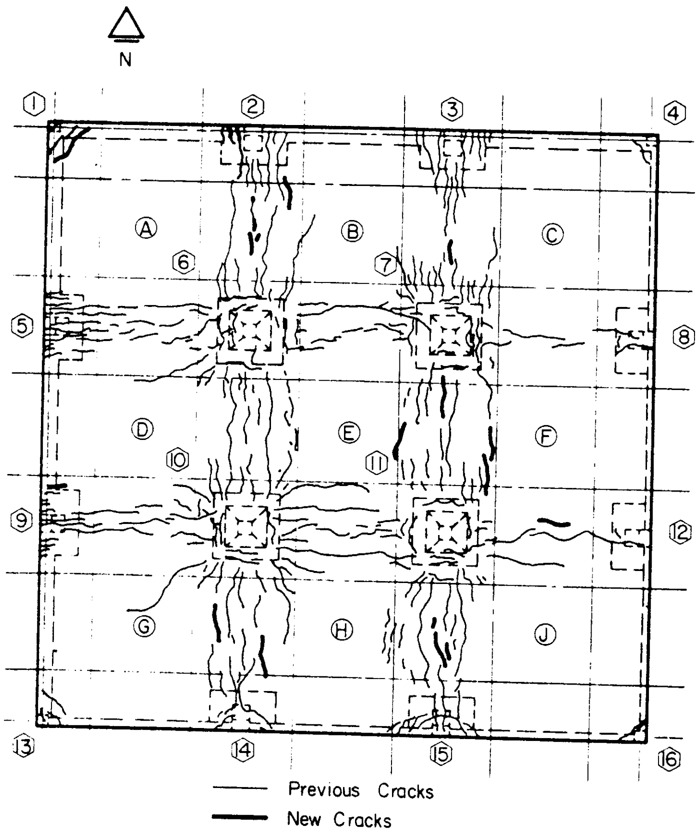

FIG. 12.—TOP CRACK PATTERN AT 1.0 *DL* + 1.5 *LL*

computing bending moments as described elsewhere (3,4,5). In computing beam moments, the deep beams were assumed to be L-beams with a ratio of flange width to slab thickness of four. Shallow beams were assumed to be rectangular. Twisting moments could not be computed from experimental measurements.

Bending Moment Distribution.—The distribution of slab unit moments at sections 1 through 7 are shown in Fig. 17. The moments were calculated from strains measured at 285 psf in test 208. The moments across the various design sections are tabulated in Table 1. Design moments are also shown and their comparison with those calculated from strains will be presented in the comparison with Design Moments section.

Comparison with Total Static Moment.—The total static moment may be defined as the average negative moment plus the midspan positive moment and can be calculated for any span if the effective position of the end reaction is known. This position was not known exactly but was determined on the basis

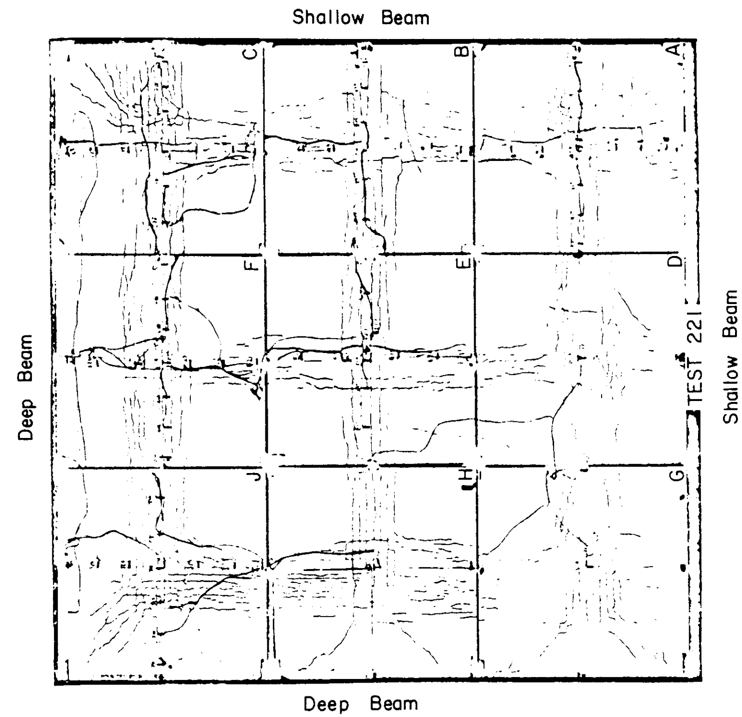

FIG. 13.—BOTTOM CRACK PATTERN AT 1.0 *DL* + 1.5 *LL*

of two assumptions: (1) Shear is uniformly distributed around the column or capital only, and (2) the shear is uniformly distributed along all supported edges including the beams.

In Table 2 the static moments in spans extending the full width of the structure are compared. A second comparison is made in Table 3 between total moments in an interior strip of spans one panel in width which includes panels BEH or DEF. Shears and twisting moments on the column centerlines which form the boundaries of the strip are ignored.

In the first comparison, the moments calculated from measured strains are at most 7% higher than the computed static moment in each span. This dis-

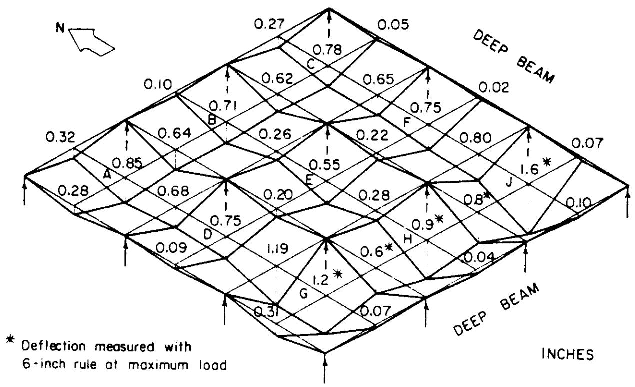

FIG. 14.—DEFLECTIONS AT 550 PSF

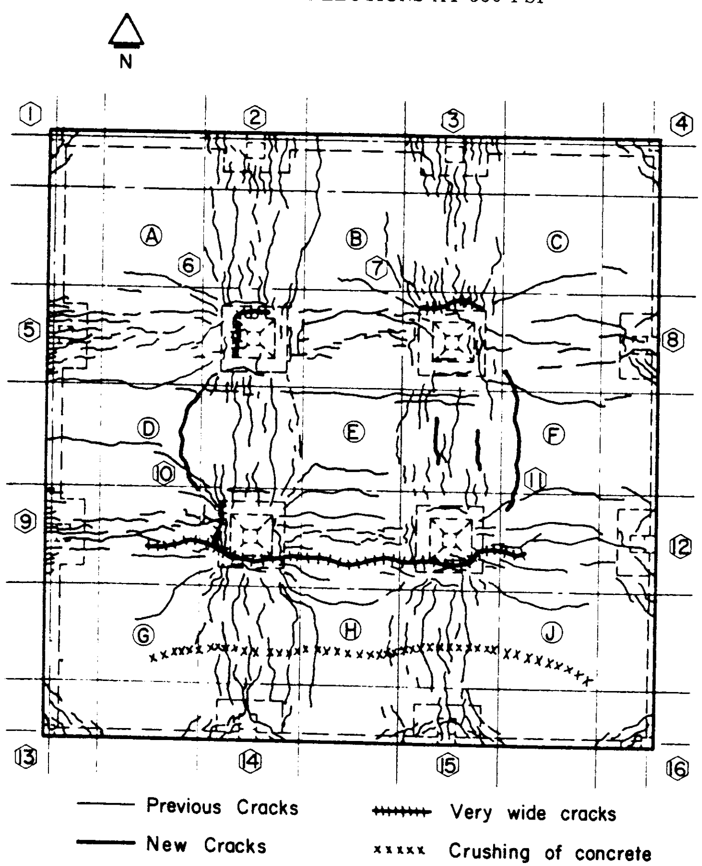

FIG. 15.—TOP CRACK PATTERN AFTER TEST TO FAILURE

crepancy can be attributed in part to uncertainties in the calculation of moments from strains in the L-shaped edge beams. In the second comparison, the uncertainties of effect of edge beams were eliminated only to be replaced by the unmeasured effect of the twisting moments. In this case, moments calculated from strains were on the order of 10% to 15% lower than the computed static moments.

Comparison with Design Moments.—Design moments (ACI 318-63)(1) are

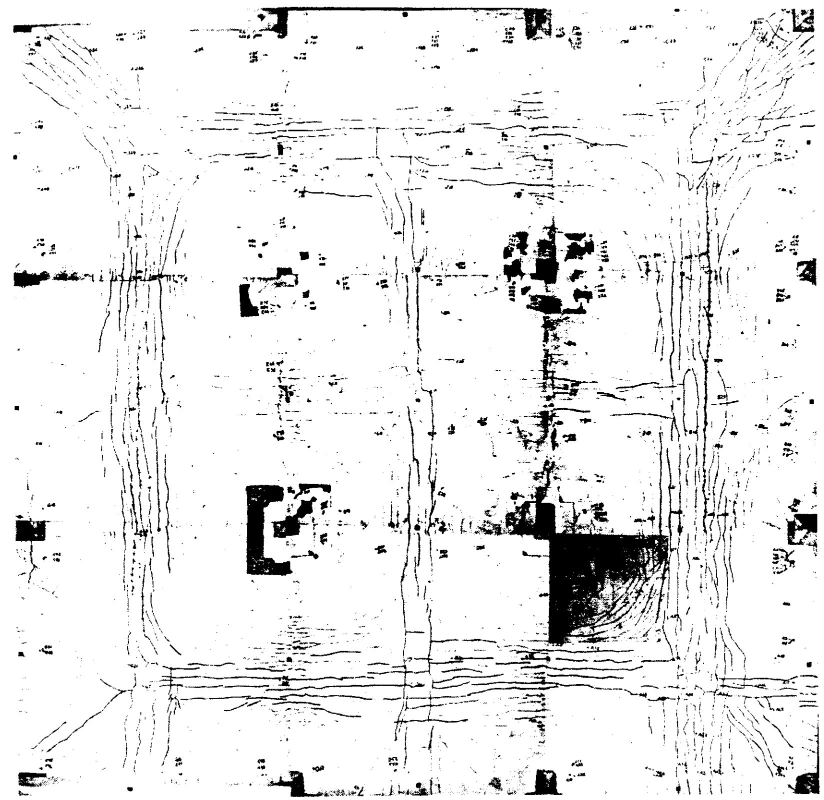

FIG. 16.—BOTTOM CRACK PATTERN AFTER TEST TO FAILURE

compared with measured moments at sections 1 through 7 in Fig. 17. Measured moments were significantly lower than design moments in the middle strips of sections 1 and 7. They were more than 50% higher than design moments at many locations in sections 3 and 5 and were also significantly higher than design moments in sections 2, 4, and 6. The measured and design moment coefficients at each design section are given in Table 1 and are compared in Fig. 18. Measured moments are plotted vertically and design moments are plotted horizontally. Points representing sections in which measured moments exceed design moments are above the 45-degree line. Column and middle strips were generally underdesigned. Those which were overdesigned were at sections 1 and 7.

Measured and design moments across the full width of the structure are compared in Table 4. Sections 1 and 7 were overdesigned by approximately 20% and most other sections were underdesigned, some by as much as 25%. These discrepancies between measured and design moments are not necessarily serious if the moment capacity of the overdesigned sections can be fully developed.

FLEXURAL STRENGTH

Two possible collapse mechanisms were considered in determining the flexural strength of the structure. Mechanism 1 was a slab failure involving

TABLE 1.—MEASURED AND DESIGN MOMENT COEFFICIENTS $M/(wL)^3$

Section (1)	Beam (shallow) (2)	Wall strip (3)	Middle strip (4)	Column strip (5)	Middle strip (6)	Column strip (7)	Middle strip (8)	Wall strip (9)	Beam (deep) (10)
1 Measured	0.012	0.006	0.007	0.027	0.006	0.020	0.005	0.006	0.024
Design	0.012	0.013	0.007	0.028	0.007	0.028	0.007	0.006	0.022
2 Measured	0.017	0.009	0.019	0.020	0.018	0.019	0.014	0.004	0.022
Design	0.016	0.009	0.013	0.016	0.013	0.016	0.013	0.004	0.025
3 Measured									
(exterior)	0.022	0.020	0.017	0.039	0.018	0.039	0.014	0.003	0.048
(interior)	0.020	0.020	0.017	0.035	0.018	0.038	0.014	0.003	0.046
Design	0.019	0.018	0.011	0.037	0.011	0.037	0.011	0.009	0.035
4 Measured	0.008	0.006	0.010	0.013	0.009	0.014	0.009	0.002	0.011
Design	0.012	0.007	0.010	0.013	0.010	0.013	0.010	0.003	0.017
5 Measured									
(exterior)	0.024	0.019	0.015	0.041	0.018	0.042	0.013	0.003	0.058
(interior)	0.020	0.019	0.015	0.040	0.018	0.040	0.013	0.003	0.046
Design	0.019	0.018	0.011	0.037	0.011	0.037	0.011	0.009	0.035
6 Measured	0.014	0.007	0.015	0.015	0.017	0.018	0.015	0.003	0.027
Design	0.016	0.009	0.013	0.016	0.013	0.016	0.013	0.004	0.025
7 Measured	0.005	0.009	0.008	0.017	0.004	0.022	0.007	0.012	0.017
Design	0.012	0.014	0.013	0.016	0.013	0.016	0.013	0.004	0.025

only the exterior panels of the structure in which the beams did not participate in the failure. Mechanism 2 involved only a single span of the structure with yield lines extending across the full width of the structure including the beams. The limiting moment at a given section was determined from

$$M_y = A_s f_y jd \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

For sections which are underreinforced, as the slab was, the difference between the yield moment and the ultimate moment is not significant.

At sections where there were two crossing layers of reinforcement, the average depth was used for the effective depth. The deep spandrel beams were assumed to be L-beams with a flange equal to four times the slab thickness. Centroids of column reactions were determined by assuming the shear to be uniformly distributed around the perimeter of supports. The only resistance

to rotation was assumed to be the limiting moment across the full length of a section.

The total collapse loads computed for the two mechanisms were: Mechanism 1, 565 psf; and Mechanism 2, 645 psf. The minimum computed collapse load was about 2.5% higher than the actual collapse load of 551 psf. The yield line pattern in the structure at collapse corresponded to that assumed for Mechanism 1 with the important exception that no yield lines developed along the face of the spandrel beams. Instead, the moment capacity of the exterior support of the slab was limited by the torsional capacity of the beams or the flexural capacity of the exterior columns.

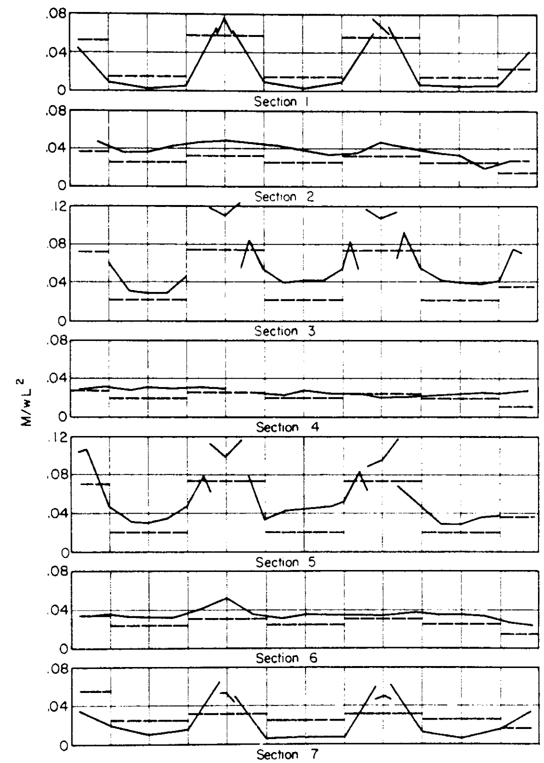

FIG. 17.—MEASURED AND DESIGN BENDING MOMENT COEFFICIENTS AT DESIGN LOAD

The external prestressing provided for the side columns of the structure has been described earlier. At about 98% of the collapse load, failure of the concrete occurred under the prestressing clamp on one of the side columns in the bay in which the failure mechanism formed. Moment-axial load interaction diagrams for the side columns with and without prestressing show that at the collapse load the moment in the columns was less than the flexural capacity with prestressing but more than the flexural capacity without prestressing.

Therefore, although it is apparent that failure of the column contributed to the collapse, it is doubtful that the collapse load would have been significantly greater if the column had not failed since the spandrel beams had also apparently failed in torsion.

GENERAL REMARKS

The ratio of the failure load to the design load for this structure was about 1.9. This is approximately the same as the factor of safety implied from the allowable working stress for the reinforcement. However, an adequate factor of safety is not the only criterion by which satisfactory performance of building structures must be measured. The structure also must be free from excessive cracking and deflections at service loads. In structures designed by working stress methods, these are controlled implicitly by limiting permissible stresses at service loads. Designing flat slabs for a total moment less than the static moment, as is permitted by the ACI Code, is equivalent to permitting higher allowable stresses at service loads. As a result, cracking and deflections should be greater in flat slabs than in continuous beams.

Cracking of the test structure at service loads was not excessive. However, a direct projection from the test structure to a full size structure cannot be made. The bond characteristics of the reinforcement used in the test structure were not as good as for deformed reinforcement. Thus, crack widths and

TABLE 2.—MOMENTS ACROSS THE ENTIRE STRUCTURE

(1)	Span adjacent to shallow beam sections 1,2,3 (2)	Interior span sections 3,4,5 (3)	Span adjacent to deep beam sections 5,6,7 (4)
Calculated from strains	$0.309wL^3$	$0.294wL^3$	$0.298wL^3$
Calculated (assumption 1)	$0.292wL^3$	$0.282wL^3$	$0.298wL^3$
Calculated (assumption 2)	$0.289wL^3$	$0.280wL^3$	$0.319wL^3$

TABLE 3.—MOMENTS IN INTERIOR STRIP

(1)	Span adjacent to shallow beam sections 1,2,3 (2)	Interior span sections 3,4,5 (3)	Span adjacent to deep beam sections 5,6,7 (4)
Calculated from strains	$0.081wL^3$	$0.079wL^3$	$0.075wL^3$
Calculated (assumption 1)	$0.090wL^3$	$0.088wL^3$	$0.090wL^3$
Calculated (assumption 2)	$0.093wL^3$	$0.088wL^3$	$0.100wL^3$

TABLE 4.—COMPARISON OF MEASURED AND DESIGN MOMENTS

Section (1)	Measured (2)	Design (3)
(*a*) End span with the Shallow Beam as Exterior Support		
1	$0.113wL^3$	$0.130wL^3$
2	$0.142wL^3$	$0.125wL^3$
3 (exterior)	$0.220wL^3$	$0.188wL^3$
(*b*) Interior Span		
3 (interior)	$0.211wL^3$	$0.180wL^3$
4	$0.082wL^3$	$0.095wL^3$
5 (interior)	$0.214wL^3$	$0.180wL^3$
(*c*) End Span with the Deep Beam as Exterior Support		
5 (exterior)	$0.233wL^3$	$0.188wL^3$
6	$0.131wL^3$	$0.125wL^3$
7	$0.101wL^3$	$0.126wL^3$

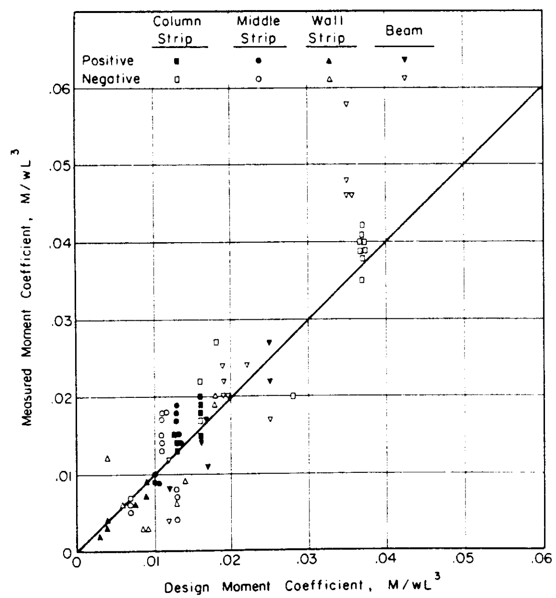

FIG. 18.—COMPARISON OF MEASURED AND DESIGN BENDING MOMENT COEFFICIENTS

spacing in the test structure could be expected to be relatively greater than in a full-size structure. However, the tensile strength of the small-aggregate concrete in the test structure was greater than that obtained from concrete made with coarse aggregate. This would result in cracking at a lower load in a full-size structure with a greater degree of cracking resulting at the service load.

The 1963 ACI Building Code (2) limits deflections of floor systems which do not support partitions to $L/360$ for short-time loads. For the test structure the deflection corresponding to this limitation is 0.17 in. This value was exceeded in each of the corner panels of the test structure in the design load test. The maximum deflection in panel A was nearly 40% greater than $L/360$. In a full-size structure in which the concrete tensile strength was lower than that in the test structure, cracking at a lower load would result in larger service-load deflections.

The effects of partial loadings have not been included in this study. However, partial loading tests at the design load caused extensive cracking, and a corresponding reduction in stiffness, throughout the structure. For 9 selected locations (7), the deflections, including dead load and cumulative residual, at the design load in test 221 were an average of 50% greater than those at the design load in test 208. The average deflection at 7 selected locations in the exterior panels at the design load in test 221 was 30% greater than the deflection corresponding to $L/360$.

The test clearly indicates that the critical design criterion is serviceability rather than safety and that the service-load performance of flat slabs designed for and subjected to high permanent live loads can be expected to be marginal. Poor quality concrete or poor construction practices or workmanship can be expected to lead to objectionable deflections. The good performance of structure F1 at the design load resulted from the structure being essentially uncracked when subjected to the relatively low design load.

In this structure, as in other structures tested in this investigation (4,5) the importance of providing adequate torsional strength and stiffness for the spandrel beams was apparent. Negative moment reinforcement provided perpendicular to the spandrel beams is wasted unless the beams have sufficient torsional stiffness and strength to develop it.

SUMMARY AND CONCLUSIONS

The behavior of a flat slab at various stages of loading to failure has been described. The test structure was one of five tested in an investigation of reinforced concrete floor slabs. The objective of the investigation (6) is the development of a unified procedure for the design of all types of reinforced concrete floor slabs.

The plan and details of the test structure and of the reinforcement are shown in Figs. 1 through 5. The total design load was 285 psf and the design was based on the "empirical design method" of ACI 318-56.

Load was applied at 16 points in each of the 9 panels of the structure. A series of tests was conducted at various levels with various combinations of panels loaded.

Representative load-deflection curves are shown in Fig. 6. Deflections throughout the structure in each of the three tests are shown in Figs. 7, 11, and 14. Steel stress distributions are shown in Figs. 8 and 9 and crack patterns are shown in Figs. 10, 12, 13, 15, and 16.

At the design load, the slab was rather extensively cracked, deflections in exterior panels were of the same order of magnitude as the maximum value permitted by the ACI Code, and steel stresses in the exterior bays were at about the design stress level, except at the exterior edge. The interior bay still had small deflections, very little cracking, and low steel stresses. Above the design load level, stresses and deflections increased at an increasing rate as cracking progressed. Failure occurred in the south bay of the structure at a load of 551 psf.

Bending moments were evaluated using measured steel strains and an idealized moment-strain relationship based on tests of beams similar to strips of the slab. The moments obtained in this way are compared with design moments in Figs. 17 and 18. The comparisons indicated that the total amount of reinforcement provided was not deficient, but that it was inefficiently distributed so that some sections had too much and others too little reinforcement.

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APPENDIX II.—NOTATION

The following symbols are used in this paper:

A_s = cross-sectional area of reinforcement in a given section;
DL = dead load;
d = effective depth of section;
f_y = yield stress of reinforcement;
jd = internal moment arm of cracked section;
L = span center-to-center of columns;
LL = live load;
M_y = yield moment; and
w = unit load on the slab.