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TEST OF A FLAT SLAB FLOOR OF THE WESTERN NEWSPAPER UNION BUILDING

BY
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TEST OF A FLAT SLAB FLOOR OF THE WESTERN NEWSPAPER UNION BUILDING

1. *Preliminary.*—This bulletin gives the results of the test made on a four-way reinforced concrete flat slab floor of the Western Newspaper Union Building in Chicago in August and September, 1917. A load of 913 lb. per sq. ft. was applied over four panels. The building, which was nine years old at the time of the test, was to be torn down to clear the site for the new Union Passenger Station; the opportunity was utilized to apply a test load much greater in proportion to the design load than had been used in previous tests of buildings. The test was carried far enough to give stresses in the reinforcing bars and concrete markedly higher than have been obtained in other building tests. The information on the action of the slab in its various parts given by the strain measurements has an important bearing on the design of the flat slab structure.

2. *Acknowledgment.*—The test was made as investigative work of the Engineering Experiment Station The testing work was done under the direct supervision of Mr. Gonnerman. He and Mr. N. E. Ensign, Associate in Theoretical and Applied Mechanics, acted as observers. The results have been reduced and prepared for publication as a bulletin of the Engineering Experiment Station.

Acknowledgment of valuable aid received in carrying out the test is made. The Portland Cement Association furnished the labor for preparing for the test and for hauling the loading material and loading and unloading the floor. The Universal Portland Cement Company assisted in making arrangements and gave assistance on the test. The pig iron used for loading material was lent by the Illinois Steel Company. The freight on the pig iron was borne jointly by the Pennsylvania Railroad and the Portland Cement Association. Opportunity to use the building for the purpose of the test was given by the Chicago Union Station Company; the test was made at the suggestion of A. J. Hammond, Principal Assistant Engineer. The Condron Company provided an assistant for tracing and checking the transfer of the loading material.

3. *The Building.*—The Western Newspaper Union Building was an eight-story reinforced concrete structure located at Clinton and Adams Streets, Chicago. The building was erected in the spring of 1909 by the George Hinchcliff Company, contractors, according to plans furnished by S. N. Crowen, architect, and Ritter and Mott, engineers. It had been in use by a printing company until 1916. The floor tested had been occupied by printing presses. Fig. 1 is a view of the building at the time of the test; the wrecking of the building had begun.

Two types of floor construction were used in the building. The first five floors were slab and girder type; the sixth, seventh, and eighth floors were Turner mushroom flat slab type (four-way reinforcement). The floors of the building were divided into panels 17 ft. 5½ in. by 19 ft. 4½ in. The test was made on the sixth floor. This floor was designed for a live load of 250 lb. per sq. ft. and was nominally 8½ in. thick. A considerable variation in thickness was found, the measured thickness over the test area ranging from 7.5 to 9.8 in. Fig. 3 gives the thickness of the floor at a number of places as determined by readings with an engineer's level. In general, the thickness was greater away from the columns than in the vicinity of the columns. The interior columns were octagonal in form, 24 in. in short diameter below the floor tested and 21 in. in short diameter above it. The inside diameters of the hooping of the columns on the fifth and sixth floors are given on the plans as 21 in. and 18 in., respectively. The column capitals were pyramidal; the short diameter at the top of the capital was 54 in. The building plans called for 15 ⅝-in. round bars in each of the four bands of reinforcement in the floor slab and indicated that over most of the columns in the test area there were laps in certain bands. After the test was made, the floor was broken into and the location and extent of all laps and the position of reinforcing bars with respect to the surfaces of the slab were found. Fig. 4 shows the arrangement of the reinforcement found over the test area, including the position of the laps. In several places the arrangement of reinforcement differs from that given in the building plans. In three places in rectangular bands the reinforcement for positive moment was double that given on the plans (30 bars instead of 15). The lapping of bars at columns was generally greater than that indicated on the plans. At column 15 three bands were lapped; at columns 14, 16, 21, 26, 27, and 28, two bands; and at columns 22 and 23, one band. In most cases, the length of lap and its position were such that the extra

Fig. 1. View of Western Newspaper Union Building at Time of Test

Fig. 2. View Showing Full Load on Floor

metal was effective in regions of greatest moment. In some cases the laps were poorly arranged, as at columns 15, 16, and 27. No reason is apparent for the lapping of bars between columns. There was no reinforcement for negative moment in the region midway between columns. The eight 1¼-in. column rods were bent out into the slab, and two circumferential ring rods (circles of 5 ft. 6 in. and 8 ft. 6 in. diameters) rested on these and supported the lower layers of reinforcing bars. The measurement of position of bars with respect to

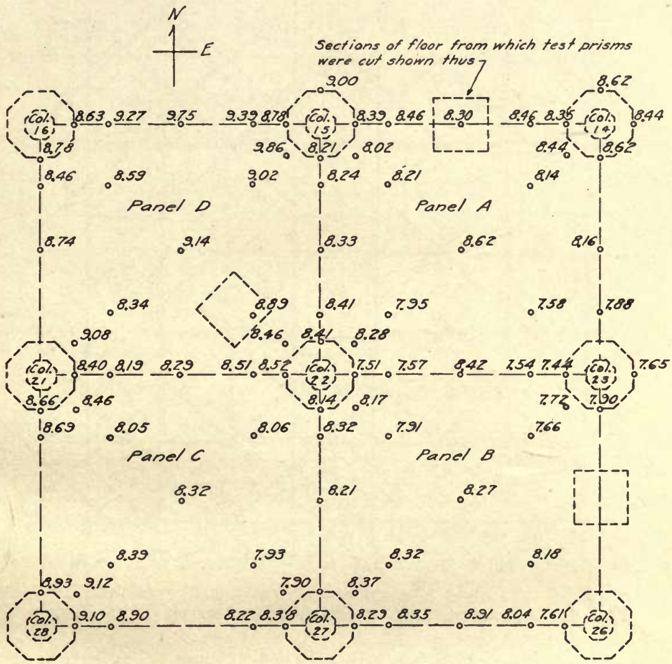

FIG. 3. THICKNESS OF FLOOR IN TEST AREA

the surface of the slab showed considerable variation at the several columns. The method of lapping was not always the same; in some cases the laps of a given band were not at the same level—at columns 15 and 22, for example, there were five layers of ⅝-in. bars besides the circumferential and bent-out column bars. At the columns the distance of the centers of the bars of the top layer from the upper surface of the slab varied from 0.90 in. to 2.00 in., and that of the lower layer from 3.60 to 4.00 in. At points between columns the centers of the bars of the rectangular bands were from 2.30 in one case to 4.20 in. above the lower surface of the slab. In the region of the center of the panel the centers of the lower layer of diagonal bars were from 1.20 to 2.20 in. above the lower surface of the slab.

The variation in amount of reinforcement available at the different sections of the slab due to the diversity in amount and position

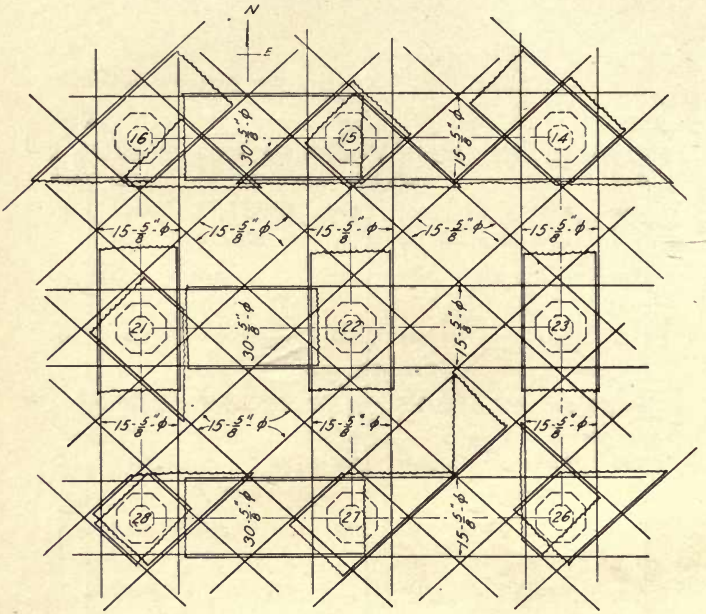

Fig. 4. Arrangement of Reinforcement in Test Area

of laps, and the variation in depth of reinforcing bar and thickness of slab, as well as variations in the quality and stiffness of the concrete in different parts of the loaded area, may be expected to cause some lack of uniformity in the stresses and deflections at points similarly located on the test area.

The concrete in the slab was 1-2-4 mix; in the columns 1-1-2 mix was used. The coarse aggregate was gravel. At the time of the test the building was eight years old. Pieces of the concrete were cut out from the floor and sawed into test prisms approximately 5 in. square and 16 in. long; one face of the prism was coincident with the upper surface of the slab. The concrete was taken from parts of the slab (indicated in Fig. 3) which had not been highly stressed and which was relatively free from reinforcing bars and cracks. The results of the tests are given in Table 1. The strength of these test prisms

TABLE 1

COMPRESSION TESTS OF CONCRETE PRISMS

Specimen	From Panel	Length inches	Section inches	Loaded Area sq. in.	Maximum Applied Load pounds	Unit Compressive Strength lb. per sq. in.	Modulus of Elasticity lb. per sq. in.
A1	A	16.9	4.6 by 4.8	22.1	71 000	3210	4 500 000
A2	A	16.8	4.7 by 4.7	22.1	119 300	5400	5 100 000
A3	A	16.2	4.9 by 3.9	19.1	104 400	5460	4 800 000
					Average	4690	4 800 000
B1	B	18.0	6.5 by 4.5	29.3	93 400	3190	4 200 000
B2	B	15.0	4.7 by 4.7	22.1	79 100	3580	3 500 000
					Average	3385	3 850 000
D1	D	16.0	4.5 by 3.0	13.5	48 600	3600	4 600 000
D2	D	16.5	5.1 by 2.5	12.8	42 800	3340	4 500 000
D3	D	10.0	4.9 by 3.1	15.2	60 600	3990	....
D4	D	9.5	5.0 by 3.1	15.5	59 700	3850	....
D5	D	8.0	4.5 by 3.2	14.4	48 200	3350	....
					Average	3626	4 550 000

ranged from 3190 lb. per sq. in. in panel B to 5460 lb. per sq. in. in panel A, and the initial modulus of elasticity from 3 500 000 to 5 100-000 lb. per sq. in. When the floor was broken up after the test, a noticeable difference was found in the quality of the concrete in the four test panels. The concrete in panels A and D appeared much stronger and harder than that in panels B and C. That in panel D was very hard.

Steel coupons were cut from reinforcing bars at different places in the tested floor. The results of the tension tests of these bars are given in Table 2. The bars gave an average yield point by drop of beam of 63 600 lb. per sq. in. and an average ultimate strength of 101 300 lb. per sq. in.

4. *The Test.*—The method of testing was similar to that used in previous buildings tests, as described in Bulletin No. 64 of the University of Illinois Engineering Experiment Station, "Tests of Reinforced Concrete Buildings Under Load." The loading material was pig

TABLE 2

TENSION TESTS OF REINFORCING BARS

Specimen No.	Average Diameter inches	Yield Point lb. per sq. in.	Ultimate Strength lb. per sq. in.	Elongation in 8 Inches per cent	Reduction of Area per cent
1	.602	66 800	103 500	17.2	37.6
2	.603	59 900	96 200	17.8	34.0
3	.619	69 100	115 600	17.8	33.4
4	.625	55 400	86 700	21.5	48.0
5	.626	64 000	100 000	16.2	39.4
6	.625	66 500	115 000	14.6	45.3
7	.625	64 900	101 600	15.1	45.3
8	.615	60 000	99 000	18.6	45.3
9	.621	65 700	102 700	16.9	41.7
Average		63 600	101 300	17.2	41.1

iron. The pig iron was hauled from the freight yards to the building in auto trucks. The net weight of each truck load of iron was obtained by weighing on a certified scale before it was hauled to the building. At the building the pig iron was loaded on hand trucks, hoisted to the test floor by means of an electric freight elevator, and then placed on the test area by hand. A record was kept of the number of truck loads placed on each panel and the total weight on each panel was obtained from the truck weights.

The load was applied over the four interior panels of the sixth floor. Fig. 5 gives the location of the panels tested. The load on each panel was divided into quarters by means of aisles 6 to 8 in. wide extending at right angles to each other along the center lines of the panels and along the boundaries between panels. The space occupied by the aisles and by the boxes built on the floor to afford access to the gage lines amounted to 17 per cent of the panel areas. The final load on the slab was 913 lb. per sq. ft., a total load of 308 400 lb. per panel. Fig. 2 shows the full load in place.

Gage lines were prepared in advance of the test—103 on the reinforcing bars and 75 on the concrete. Fig. 6 and 7 show the location of the gage lines on the upper and lower sides of the slab. To insure reliability of initial readings, three sets of strain gage readings (and more on many of the gage lines) were taken before the load was applied. Strain gage readings were taken at loads of 234, 425, 637, 855, and 913 lb. per sq. ft. of panel area. In each case except for the first load two complete sets of strain readings were taken on the reinforcing bars and the concrete, and sometimes more. The deflection of the slab was measured at 20 points. The location of the deflection points are shown in Fig. 8. The appearance of cracks was also noted. Readings of deformation and deflection at the more important points were taken from time to time as the load was being placed on the floor.

Readings were also taken three days after the maximum load had

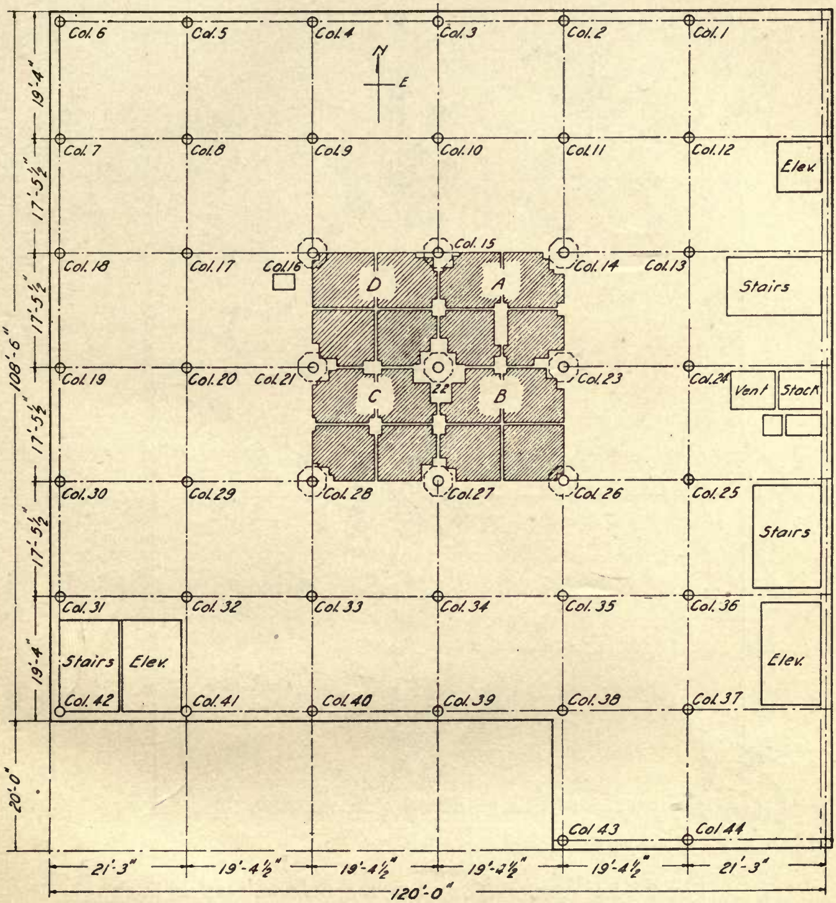

FIG. 5. LOCATION OF TEST PANELS

been placed on the floor in order to get information on the time effect of the load upon deformations and deflections. After the removal of the load readings were taken to find the amount of recovery in the floor.

The test area was chosen where there would be the least effect of floor openings and where the building plans showed laps in only the rectangular bands over the column in the center of four panels. In a building test the considerable time required to reduce the data of the

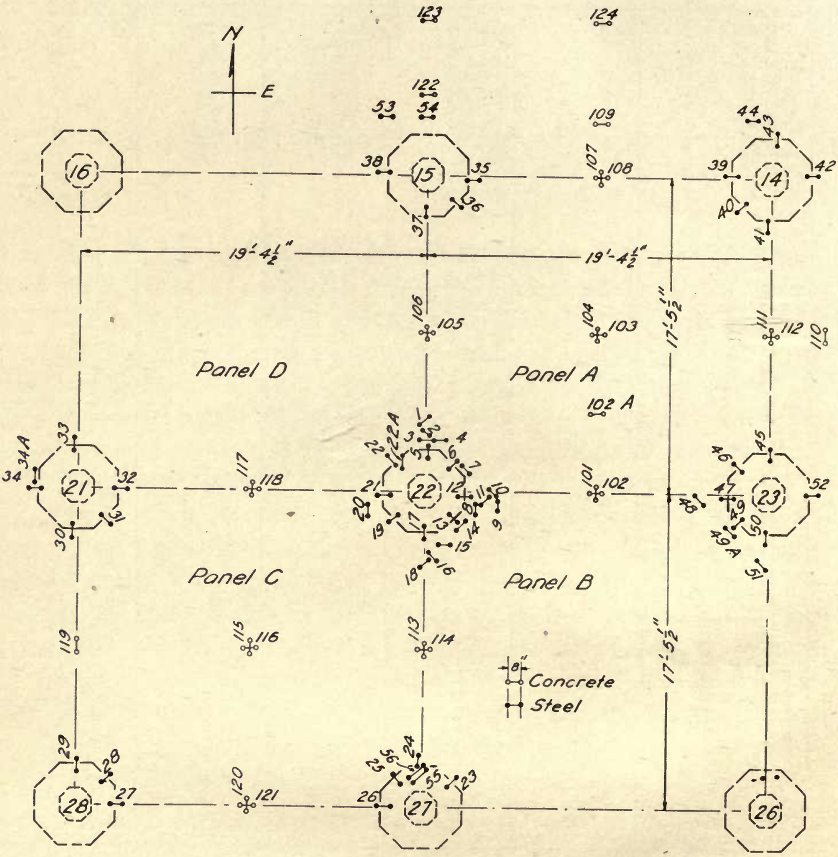

FIG. 6. LOCATION OF GAGE LINES ON UPPER SIDE OF SLAB

readings into form for analysis renders it necessary to restrict the number of gage lines and so their distribution over the test area becomes a matter of importance. The gage lines in this test were placed with a view of getting some information on (1) the amount and distribution of the stresses in the reinforcement along sections through

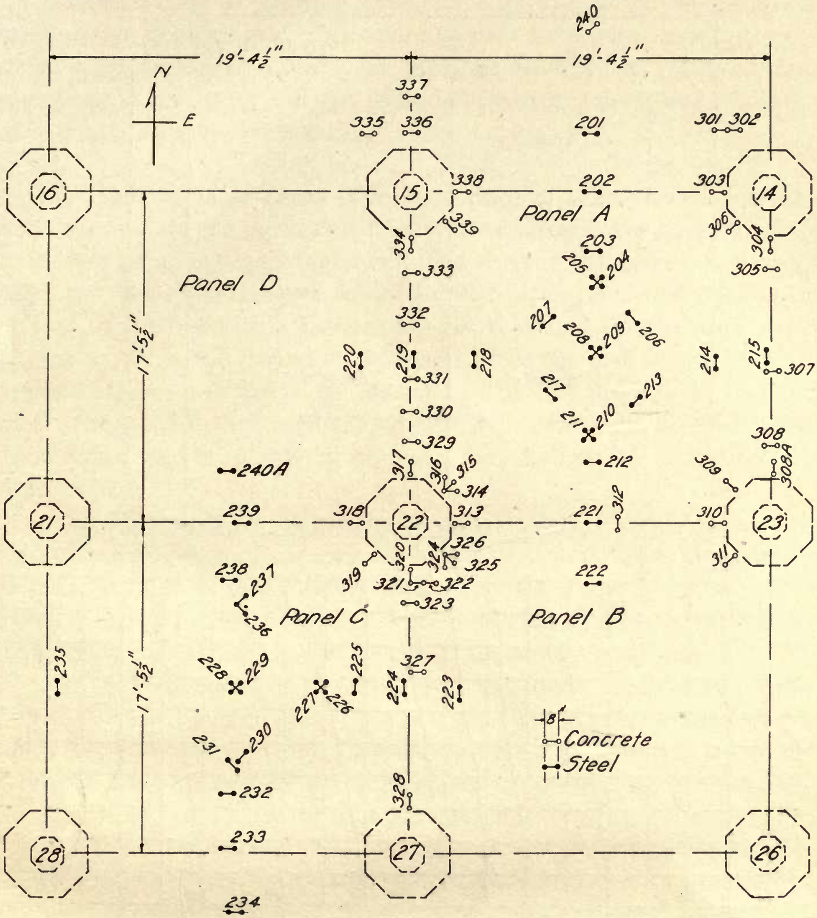

FIG. 7. LOCATION OF GAGE LINES ON LOWER SIDE OF SLAB

the panel centers and through the panel edges, and at other points, (2) the strain in the concrete at the more important points in the slab, (3) the moment of resistance accounted for by the stresses in the reinforcing bars which cross sections through the panel centers and through the panel edges, and (4) something on the stresses in columns at the edges of the loaded area due to bending under the applied load.

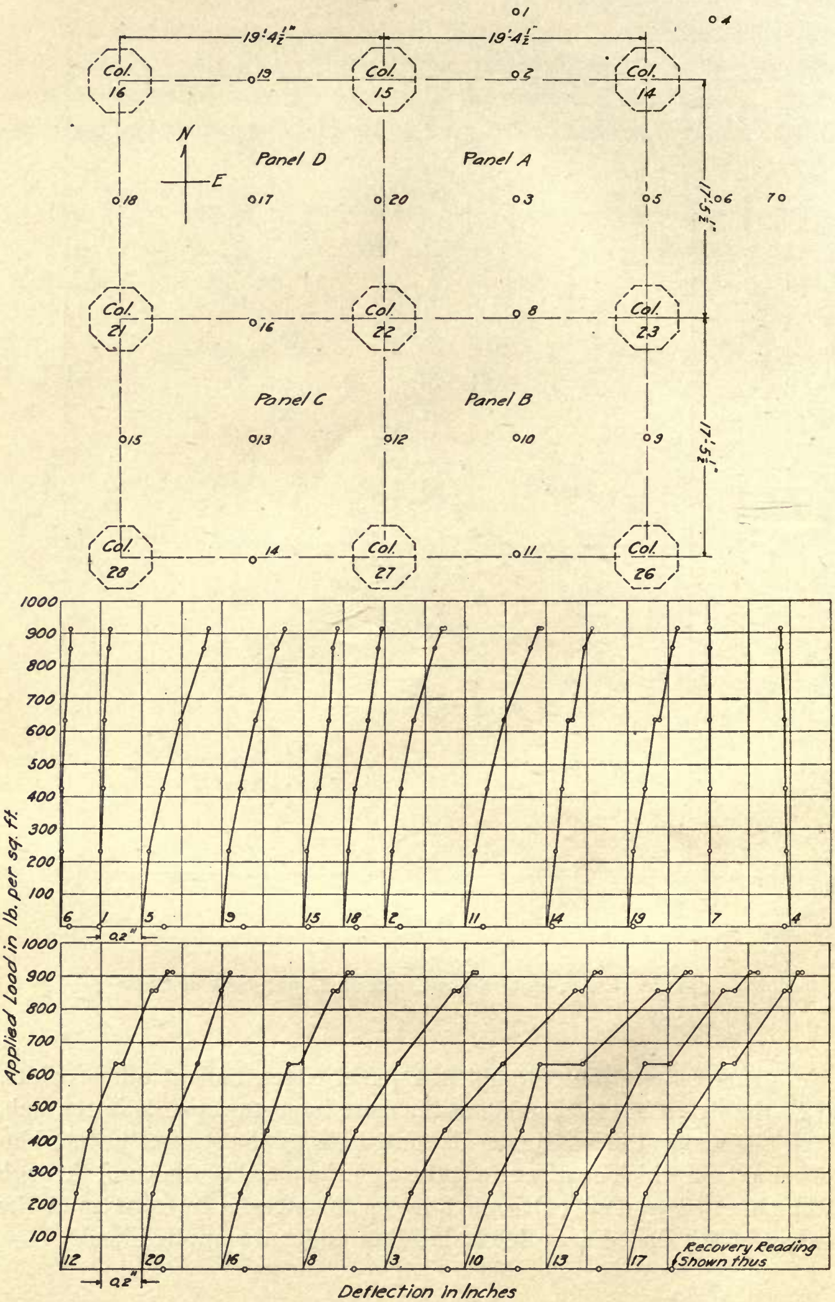

FIG. 8. LOAD-DEFLECTION DIAGRAMS AND LOCATION OF DEFLECTION POINTS

The gage lines in panels A and C with few exceptions were laid out in duplicate in order that readings obtained on the gage lines in the one panel might serve as a check on readings obtained at corresponding gage lines in the other, and also to find out whether there was similarity of action in the two panels.

5. *Deflection of Slab.*—The deflections of the slab at the several deflection points are plotted in Fig. 8, the diagrams for points similarly located being grouped together. The recovery of deflection one day after the load was removed is indicated by the points plotted at the bottom of the diagram. The second point plotted for the load of 637 lb. per sq. ft. is the deflection 16 hours after the last of the load was applied; the second point for the load of 855 lb. per sq. ft., 37 hours after; the second point for the load of 913 lb. per sq. ft., 66 hours after. Where no second point is plotted, the change in deflection was negligible.

The deflections at the centers of panels A, B, C, and D under the load of 913 lb. per sq. ft. were 1.06, 1.12, 1.04, and 0.87 in., respectively. It may be noted in this connection that panels C and D had a greater amount of reinforcement than A and B and that panel D was thicker. The concrete of panels A and D was of unusually good quality.

Of the deflections at the middle of the inner edges of the loaded panels, that at point 8 (Fig. 8) was considerably greater than that at point 16, and that at point 12 was more than at point 20. The differences are explainable by differences in quality of concrete and in amount and arrangement of reinforcement.

At the outer edges of the loaded area, the deflections at points 5 and 9 were considerably greater than at points 15 and 18; and the deflections at points 2 and 11 were greater than at points 14 and 19—the same circumstances explain these differences in deflections.

It may be noted that points 1 and 6 (Fig. 8) distant one-quarter of the panel lengths from the panel edges gave a measurable deflection. Point 7 (center of adjacent panel) remained stationary and point 4 showed an upward movement.

6. *Cracks in Slab.*—It was noted before the load was applied that there were numerous checks in the upper surface of the slab in the regions around the column and along the panel edges. Most of these were evidently surface checks; others were tension cracks formed under previous loads. The latter opened upon the application of load.

Fig. 9 gives the location of the more important cracks on the upper side of the slab which either opened or formed under the test load. These cracks were all open cracks—much more marked than hair cracks. Under the load of 913 lb. per sq. ft. they ranged in width from 0.02 to 0.06 in. These cracks show the regions of high tensile stress in the top of the slab. The main cracks at the columns were generally at or near the edges of the column capital; they branched out to join the cracks extending along the panel edges between the columns. The cracks at the capitals of the columns bordering the loaded area were

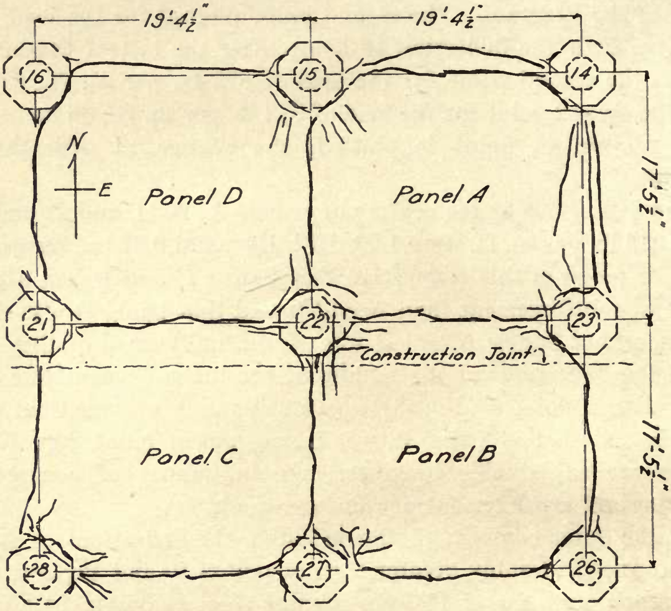

Fig. 9. Location of Main Cracks in Upper Side of Slab

fully as wide as those at the capital of the central column. The cracks along the panel edges were generally as pronounced as those around the capitals.

Fig. 10 shows the location of the more important cracks on the lower side of the slab in panels A and C, the panels in which the principal tension gage lines on the lower side of the floor were placed. In panels B and D, in addition to those noted in the figure, there were cracks over the panels in positions similar to those noted in panels A and C. In panel D the cracks were not so wide nor so numerous as in the other panels, even at the maximum load. Panel B had larger and more numerous cracks than the other panels. Most of the cracks on the lower side of the slab were found in bands extending in rectangular directions. In a small area at the panel centers cracks extended in diagonal directions. No cracks or checks had been noted on the lower side of the slab before loading was begun. The first hair cracks here were observed at a load of 234 lb. per sq. ft. At a load of 425 lb. per sq. ft., the cracks were fairly well defined and for higher

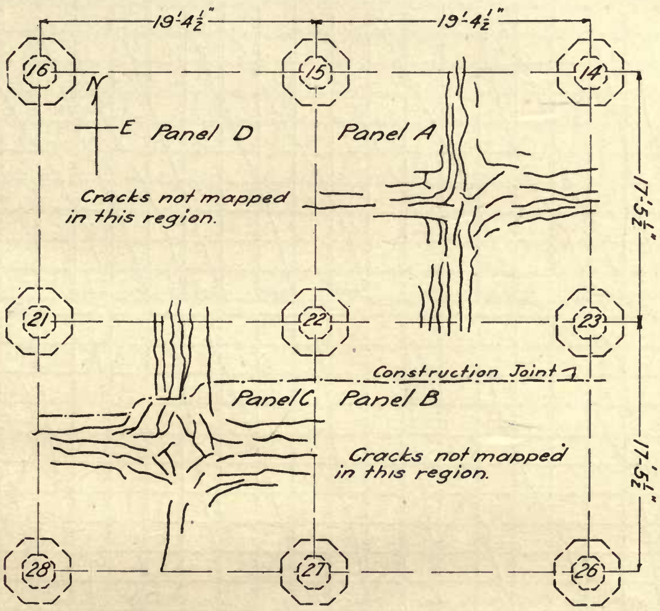

FIG. 10. LOCATION OF MAIN CRACKS IN LOWER SIDE OF SLAB

loads they gradually opened up and extended. The main cracks on the lower side of the slab did not open so much as the main cracks on the upper side of the slab. The construction joint shown in Fig. 9 and 10 opened appreciably under the application of load.

Upon the removal of the load the cracks closed, giving the slab an appearance similar to that which it had before the load was applied.

7. *Load-strain Diagrams.*—In Figs. 11 to 13 the load-strain diagrams are given for gage lines on the upper and lower sides of the

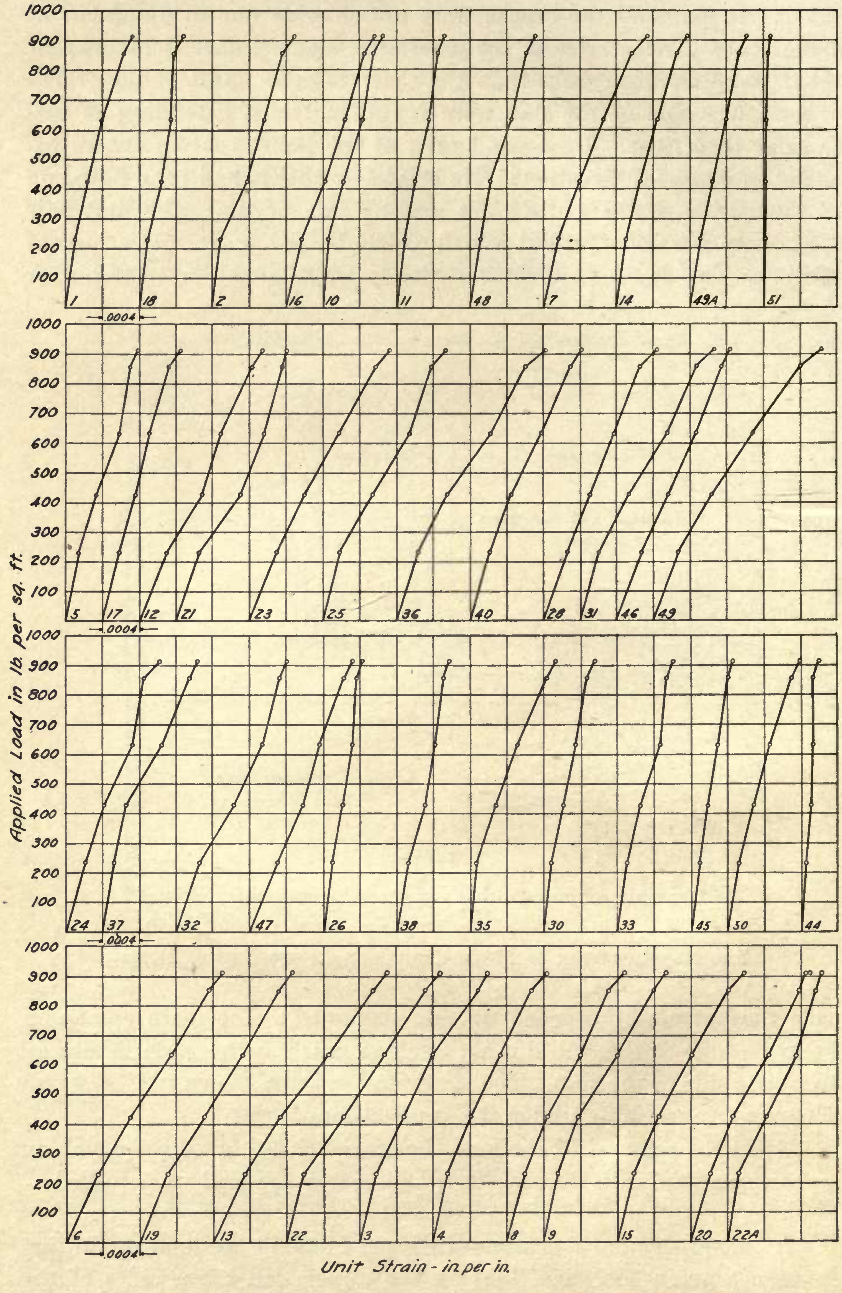

FIG. 11. LOAD-STRAIN DIAGRAMS FOR GAGE LINES ON REINFORCING BARS ON UPPER SIDE OF SLAB

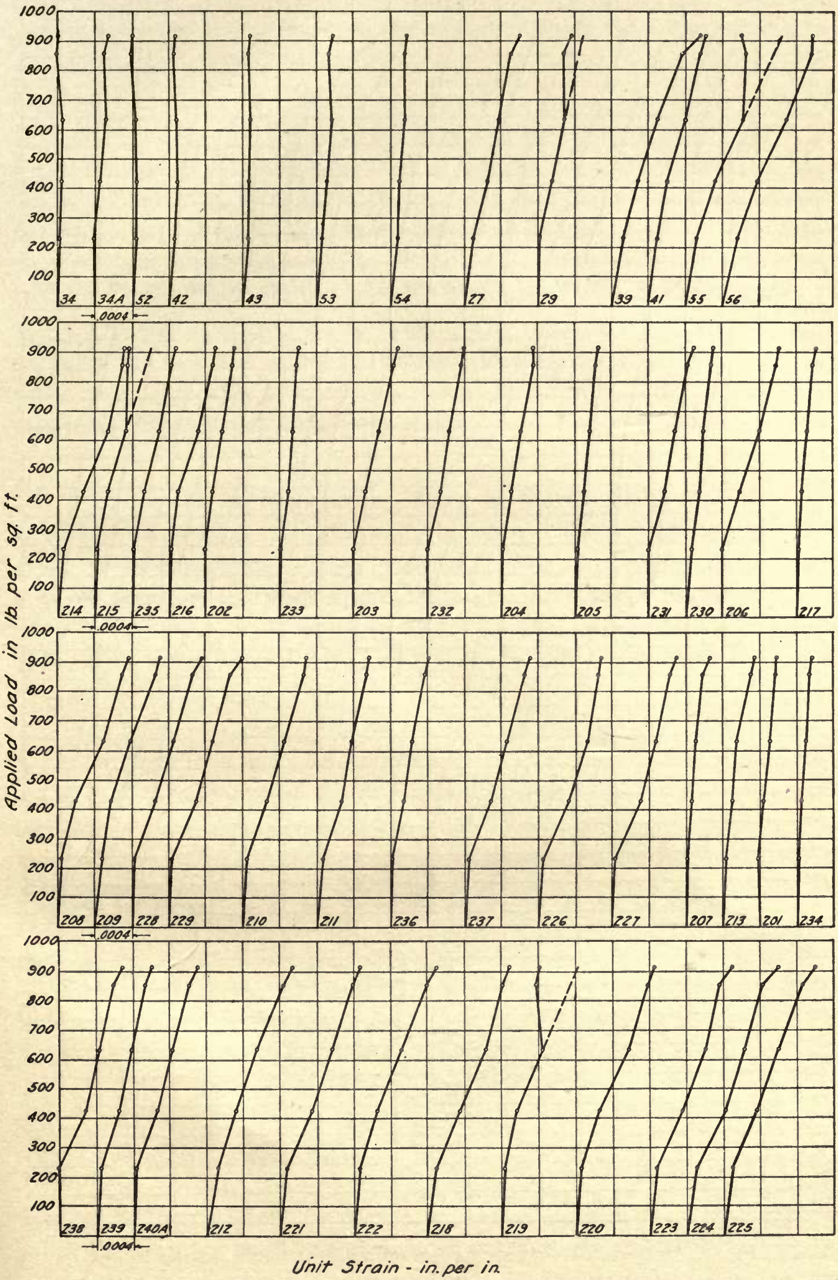

FIG. 12. LOAD-STRAIN DIAGRAMS FOR GAGE LINES ON REINFORCING BARS ON UPPER AND LOWER SIDES OF SLAB

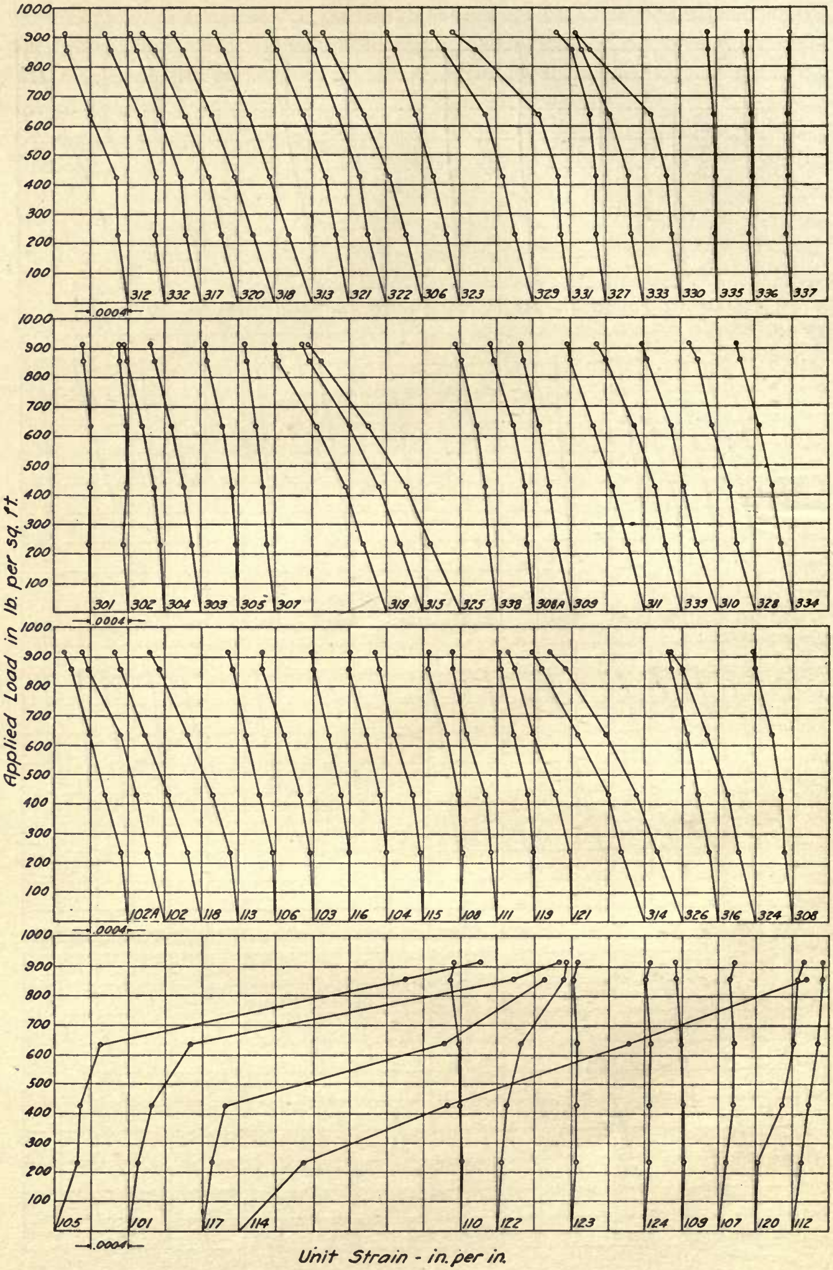

FIG. 13. LOAD-STRAIN DIAGRAMS FOR GAGE LINES IN CONCRETE ON UPPER AND LOWER SIDES OF SLAB

slab. In these diagrams tensile strains are plotted to the right and compressive strains to the left of the axis. Gage lines having similar positions on the floor have been grouped. Averages of the several readings were used in computing the strains. It will be noted that for gage lines on bars in the upper side of the slab the diagrams show deformations of considerable amount at the load of 234 lb. per sq. ft; the diagrams for gage lines on bars on lower side show markedly smaller deformations. For loads greater than 234 lb. per sq. ft. the deformations in bars on the lower side of the slab increased fairly uniformly with increase of load; it has already been noted that the first cracks on the lower side appeared at this load. The diagrams for gage lines on the concrete on both upper and lower sides of the slab show deformations of considerable amount at the first load and a fairly uniform increase in deformation for higher loads.

8. *Stresses in Reinforcing Bars in Upper Side of Slab.*—The stresses in the reinforcing bars at the several loads may be computed from the strains given in Fig. 11 and 12. For convenience of comparison the stresses corresponding to the strains for the maximum load (913 lb. per sq. ft.) are given in Fig. 14. The values shown at points around the columns are in bars at the upper side of the slab; the others are in bars at the lower side.

For bars on the upper side of the slab the greatest stresses were found at gage lines located on diagonal bars at the edge of the column capitals. At column 22 (the central column of the loaded area) the stresses in diagonal bars over the edge of the column capital ranged from 49 200 to 57 300 lb. per sq. in. The stresses in diagonal bars at column 22 at gage lines located some distance from the column capital ranged from 15 000 to 34 500 lb. per sq. in. The stresses in diagonal bars at the columns bordering the loaded area ranged from 36 600 to 54 900 lb. per sq. in. In general, the stresses in the diagonal bars at the columns bordering the loaded area were nearly as great as those observed at corresponding gage lines at the central column. The stresses given do not include the stress due to the load of the slab itself; allowing for this, it is apparent that the yield point of the steel was not reached in even the most highly stressed bars.

The stresses in the east and west rectangular band at column 22 averaged about 41 400 lb. per sq. in. The stresses in the north and south rectangular band at column 22 ranged from 23 700 lb. per sq. in.

at the edge of the column capital to 40 500 lb. per sq. in. at gage lines near the edge of the band on either side of the column. The bars in this band were lapped at column 22, the bars ending about 80 inches north and south of the column center. It will be noted that at gage lines near the edges of the rectangular bands the stresses were greater

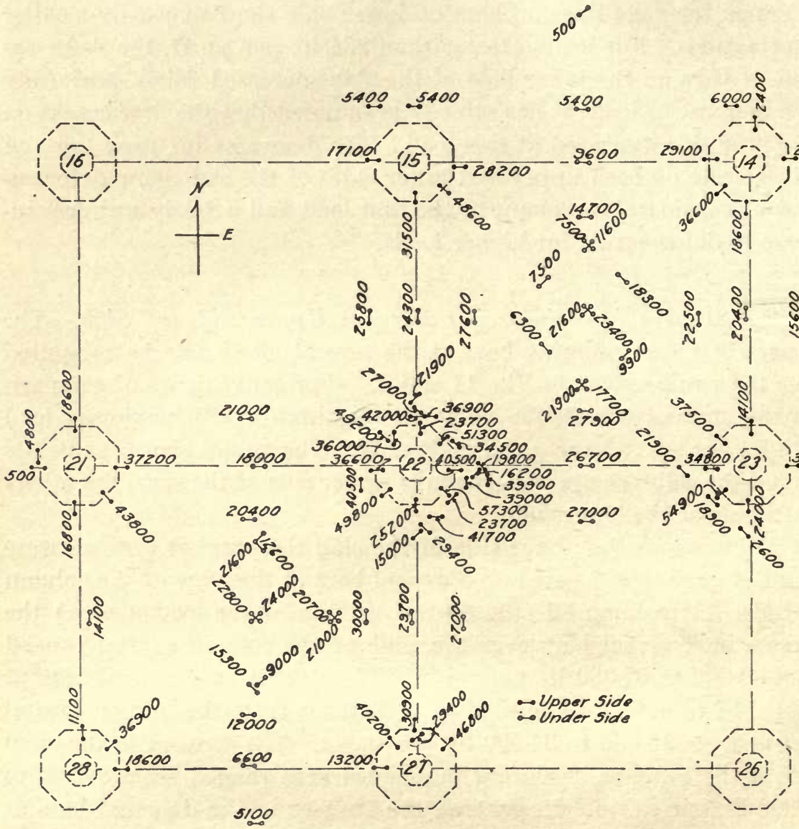

FIG. 14. STRESSES IN REINFORCING BARS IN POUNDS PER SQUARE INCH AT LOAD OF 913 POUNDS PER SQUARE FOOT

than in bars at the middle of the band. In bars outside the loaded area and near the edge of a rectangular band stresses were found as great as 6000 lb. per sq. in. It is evident that portions of the slab outside the loaded area contributed measurably to the resistance developed in the slab.

9. *Stresses in Reinforcing Bars in Lower Side of Slab.*—On the lower side of the slab the greatest stresses were found in the bars of the rectangular bands (Fig. 14). At the maximum load stresses from 24 300 to 30 000 lb. per sq. in. were observed in rectangular bands within the loaded area. In the one which had 30 bars instead of the usual 15, stresses from 18 000 to 21 000 lb. per sq. in. were found.

The stresses in the rectangular bands at the edge of the loaded area varied from 5100 lb. per sq. in. in a bar outside the loaded area to 22 500 lb. per sq. in. in a bar inside the loaded area. At one gage line outside the loaded area and near the edge of the band a stress of 15 600 lb. per sq. in. was found.

The stresses in the diagonal bands in the region of the center of the panels were smaller than those in bars of the rectangular bands in the region between columns. The stresses in diagonal bars at gage lines away from rectangular sections which pass through panel centers ranged from 6300 to 18 300 lb. per sq. in.

The effect of position of reinforcing bars with respect to surface of slab on the stress developed is discussed in another place.

10. *Strains in Concrete at Upper Surface of Slab.*—The unit-strains in the concrete at the several loads are plotted in Fig. 13. For convenience of comparison, the unit-strains at the maximum load are recorded in Fig. 15. The values for points around the columns are for gage lines on the lower side of the slab; a few gage lines which cross the panel edges between columns are also on the lower side. The remaining gage lines are on the upper side of the slab.

On the upper surface of the slab the greatest compressive strains were found at gage lines along the inner panel edges midway between columns. Strains from 0.00089 to 0.00097 in. per in. were observed at these gage lines. Assuming a modulus of elasticity of concrete of 4 000 000 lb. per sq. in. and a straight-line stress-strain relation the stresses in the concrete corresponding to these deformations would be 3560 and 3880 lb. per sq. in. Strains as great as 0.00054 in. per in. (corresponding stress in the concrete on the assumption just given, 2160 lb. per sq. in.) were found at gage lines at the panel centers.

11. *Strains in Concrete at Lower Surface of Slab.*—The greatest strains in concrete on the lower surface of the slab were found close to the edge of the capital of column 22. The strains at this column at

the diagonal gage lines (see Fig. 15) ranged from 0.0012 to 0.0016 in. per in. at the maximum load. These strains are as great as the strains which were found at failure in the tests of the concrete prisms cut from the slab; they represent the range in deformation at the ultimate load usually found in compression tests of concrete. Spalling or chipping

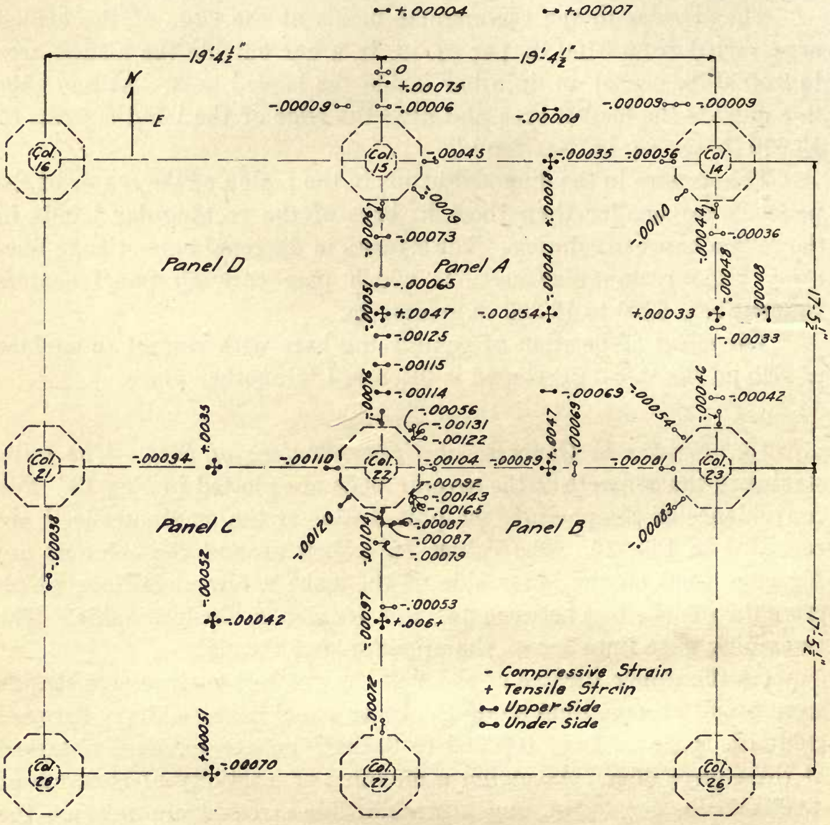

FIG. 15. UNIT-STRAINS IN CONCRETE AT LOAD OF 913 POUNDS PER SQUARE FOOT

of the concrete surface was plainly visible near the edge of the capital of column 22 in panel A. At rectangular gage lines near the capital of this column a strain in the concrete of 0.0014 in. per in. was observed. These high deformations indicate that the concrete near the capital of column 22 was highly stressed and that at certain gage lines it was stressed to its ultimate strength, the action of the surrounding concrete preventing its complete failure.

Near the edges of the capitals of columns bordering the loaded area strains from 0.00054 to 0.0011 in. per in. were observed at diagonal gage lines, and strains from 0.00044 to 0.00081 in. per in. at rectangular gage lines. With a modulus of elasticity of concrete of 4 000 000 lb. per sq. in. the stresses corresponding to these strains would range from 1760 to 4400 lb. per sq. in.

At the gage lines crossing the inner panel edges at a section of negative moment between columns 15 and 22, compressive strains about one-half those found near the column capital were observed at the lower loads, even though there was no tension reinforcement in the upper side of the slab in this region. For the highest load when the concrete at the capital had begun to crush there was a relatively great increase in the strains in this middle region, a value as great as 0.0012 in. per in. being found.

It should be noted that there were compressive strains of some amount in regions of negative moment outside the loaded area.

12. *Influence of Position of Bar.*—The stresses in Fig. 14 are given without reference to the position of the bar with respect to the surface of the slab and without reference to its position in the band. It may be expected that the stresses in the several layers of bars will vary. The average depths of the layers of bars at the edge of the capital of column 22 ranged from 0.90 in. below the upper surface of the slab for the upper layer to 3.65 inches for the lower layer. At this column the layers of bars in the order of their position with respect to the upper surface of the slab were as follows: (1) north and south rectangular bars (one lap), (2) diagonal bars running to northwest, (3) east and west rectangular bars, (4) diagonal bars running northeast, (5) north and south rectangular bars (second lap), (6) circumferential ring bars, (7) radial column bars. (See Fig. 20.)

The strains in steel and concrete at gage lines near the capital of column 22 for the load of 913 lb. per sq. ft. have been plotted in Fig. 16 to show the position of the neutral axis for the various layers of bars on which readings of deformation were taken. The strains found at gage lines 5 and 17 are markedly smaller than those found at gage lines 8, 9, 20, and 22A which were placed on bars of the same layer. Gage line 22A, like gage lines 5 and 17, was placed over the edge of the capital and it will be noted that the deformation found at this gage line is more in harmony with the deformations found at other gage lines on bars of the same layer than are the deformations at 5 and 17.

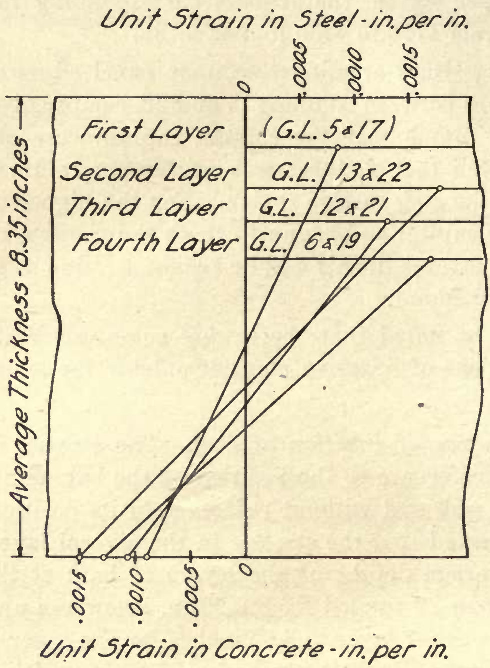

FIG. 16. POSITION OF NEUTRAL AXIS FOR THE SEVERAL LAYERS OF BARS AT COLUMN 22

No reason for the markedly smaller value in gage lines 5 and 17 is apparent. For the gage lines other than 5 and 17 the position of the neutral axis with respect to the under side of slab ranged from 0.43 to 0.46 of the effective depth of the several layers. The value of j (which represents the ratio of the distance between the bar and the centroid of compressive stresses to the distance between the bar and the face of the slab) for these bars is, therefore, about 0.85. The value of j found in a similar way for gage lines located at columns bordering the loaded area was 0.87. It will be noted in Fig. 16 that the strain in the lower layer of diagonal bars is nearly as great as that in the upper layer of diagonal bars. The strain in the layer of rectangular bars between the two layers of diagonal bars was less than that found in the diagonal bars.

No measurements of strain were made on the rings nor on the column bars bent out radially into the slab. These bars were placed low in the slab and near where the neutral axis may be expected to be. It is probable that they were stressed somewhat. As has been stated elsewhere these bars were not taken into account in the calculations of resisting moment.

For the diagonal bars in the central area of the panels, the stresses in the bars of the lower layers were generally considerably greater than those in the upper layer. In the one case where the lower bar shows considerably less stress, the gage lines were near the end of lapped bars.

The bars in the rectangular bands at sections of positive moment were farther from the under surface of the slab than were the bars of the diagonal bands, but the stresses in the former were greater than in the latter except where laps occurred. It should be noted that the bars at the middle of the rectangular bands were farther from the under surface of the slab than were the outer bars. Differences in the magnitude of stresses in bars in similar places at sections of positive moment are partly accounted for by differences in the position of the bars; for example, the bar at gage line 236 was 2.7 in. from the lower surface and the one at 237 was 1.15 in. from the lower surface, while the stresses were 12 600 and 21 600 lb. per sq. in. respectively. Similarly, the bars for gage lines 219, 221, and 239 are higher in the slab than the corresponding bars at the edges of the band.

Gage lines 2, 7 and 10 were placed on the same diagonal bar. Similarly, gage lines 11, 14, and 18 were on another diagonal bar. The bar having gage lines 2, 7, and 10 was about 1.95 in. below the upper surface of the slab and the other bar about 3.15 in. below the upper surface. The stresses at gage lines 2, 7, and 10 were 27 000, 34 500 and 19 800 lb. per sq. in., respectively; the stresses at gage lines 11, 14, and 18 were 16 200, 23 700, 15 000 lb. per sq. in., respectively. It will be seen that the stresses at the gage lines opposite the column capital (7 and 14) were greater than those observed at the other gage lines.

13. *Resisting Moment Accounted for by Stresses in Reinforcing Bars.*—It will be of interest to find the magnitude of the resisting moments developed in sections of the slab and to compare the values

with the bending moment due to the external forces. As the part played by the tensile stresses in the concrete is unknown and uncertain, only that part of the resisting moment found by using the measured stress in the reinforcing bars can be considered. The resisting moment based on the tensile stresses in these bars may not be expected to account fully for the bending moment. The two sections considered will be (1) a section across the panels midway between columns (AB, Fig.

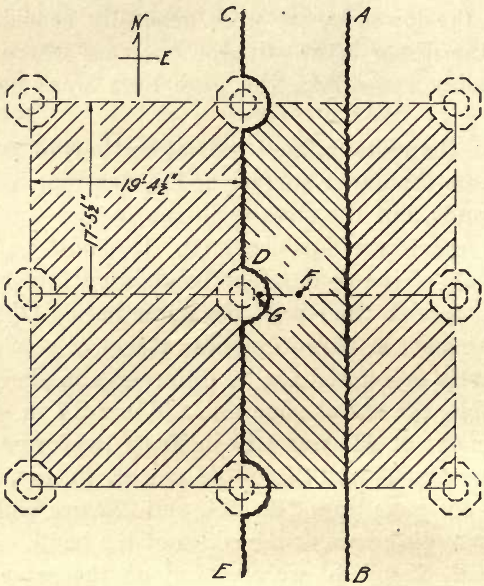

FIG. 17. SECTIONS OF POSITIVE MOMENT AND NEGATIVE MOMENT CONSIDERED IN THE CALCULATIONS

17) and (2) one along an edge of the panels parallel to the first section but skirting the part of the periphery of the column capitals at the corners of the panels (CDE, Fig. 17). It will be noted that the edges of the area here considered are along lines of zero shear, except around the column capitals. The external forces acting on the row of half panels are the load on the two half panels and the reaction or external shear at the three column capitals. The moment of the couple formed by these two external forces will be resisted by the numeri-

cal sum of the resisting moments developed in the two sections AB, and CDE. The moment of the internal stresses at the section of the panel midway between columns is referred to as the positive resisting moment and that at the edge of the panel as the negative resisting moment. The actual distribution of these resisting moments along the sections need not be considered in making the desired comparison.

The point of application of the resultant of the load on the two half panels is shown at F; that of the resultant of the reaction of the three supports at G. The moment of the couple formed by these two resultants is the bending moment due to the external forces and is the moment to be considered. Analysis shows that, for a uniformly distributed load, and round columns, the value of this bending moment for a load two panels wide is given quite closely by the equation

$$M = \frac{2}{8} W l_1 \left[1 - \frac{2}{3} \frac{c}{l_1} \right]^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

for sections at right angles to the long way of the panel, and

$$M = \frac{2}{8} W l_2 \left[1 - \frac{2}{3} \frac{c}{l_2} \right]^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

for sections at right angles to the short way of the panel where

M = bending moment for a width of two panels
W = load on one panel
l_1 = long side of an oblong panel measured from center to center of column
l_2 = short side of an oblong panel measured from center to center of column
c = diameter of column capital.

With the load of 913 lb. per sq. ft. over four panels, the bending moment for a width of two panels, as obtained by equations (1) and (2), is 12 820 000 lb.-in. for the long way of the panels (resisted at north and south sections) and 11 060 000 lb.-in. for the short way (resisted at east and west sections).

The positive moment (the resisting moment at the section across the panels midway between columns) and the negative moment (the resisting moment at the section at the edge of the panels) together must resist this bending moment. With a condition of ends of slab for which the tangent to the curve of flexure at the edges of the panels remains horizontal when the load is applied (usually termed fixed ends), the condition assumed in the analysis, the positive moment will be found by analysis to be one-third of the total resisting moment and the negative moment two-thirds, provided the slab is uniformly stiff throughout. If the tangents at the panel edges deflect somewhat, the positive moment will be greater than one-third and the negative moment less than two-thirds. In the comparisons to be made it will be assumed that the proportions given by analysis are one-third for the positive moment and two-thirds for the negative moment. As stated in "14, Bending of Columns," calculations of the resisting moment developed at sections of the slab at the boundaries of the loaded area as accounted for by the observed stresses in the reinforcing bars, the results of analytical determinations of the moments at sections at the edges and at the middle of the loaded area, and the amount of flexure in the columns all go to show that there is little inaccuracy in this assumption in the case under consideration. The analytical value of the positive resisting moment will be termed the analytical positive moment and that of the negative resisting moment the analytical negative moment. For the north and south sections the magnitude of these analytical moments becomes 4 270 000 and 8 550 000 lb.-in. respectively, and for the east and west sections 3 700-000 and 7 400 000 lb.-in. respectively.

The sections used in obtaining the resisting moment accounted for by the stresses observed in the reinforcing bars are shown in Fig. 18. QKC-BJT is the east and west section of positive moment used, and MNOP the east and west section of negative moment; similarly, IJO-NKL is the north and south section of positive moment, and ABCD and EFGH the north and south sections of negative moment. Sections of negative moment in the north and south direction are taken on two sides of the column capitals, because the available reinforcement differed in the two sections. The sections of positive moment were taken as shown because they cross the greatest number of gage lines.

In the calculation of the resisting moments developed, lapped bars were considered as contributing to the resisting moment whereever the bars extended beyond the section a sufficient distance to insure adequate anchorage with respect to the magnitude of the stress developed in the bar. Many of the laps were made in such a way that the additional section may not be expected to contribute much to the resisting moment. As measurable stresses were found in bars near the edge of the band outside the loaded area, they were taken into account in the calculations. The ring rods around the columns and the column bars which were bent out into the slab were not included in the reinforcement, for no measurements of strain were made on these bars. For the diagonal bars the component of the stress was taken in a direction at right angles to the direction of the panel edge. The average of the stresses at the principal critical gage lines was generally used. For the bands at the edges of the loaded area the stresses were considered to vary over the band from gage line to gage

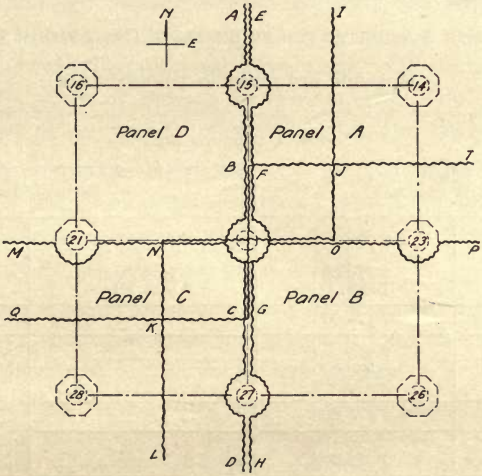

FIG. 18. SECTIONS OF POSITIVE AND NEGATIVE MOMENT CONSIDERED IN THE CALCULATION OF THE RESISTING MOMENTS ACCOUNTED FOR BY STRESSES IN THE REINFORCING BARS

line. Some judgment had to be used in determining the stress variation as well as the availability of lapped bars, but it is believed that the assumptions made give results well below the actual resistance developed in the slab. It should be noted that the maximum stresses observed in bars of negative moment were 15 per cent greater than the average of the stresses used in computing the resisting moment accounted for by these stresses, not counting bars in bands near the edges of the loaded area, and similarly the maximum stresses in bars at sections of positive moment were 25 per cent greater than the average of the stresses used in computing the positive moment. The measured position of the bars with reference to the face of the slab and the measured thicknesses of the slab were used. The position of the neutral axis was determined from the strains measured in the reinforcing bars and in the concrete at the face of the slab; knowing this distance, the values of *jd,* the distance from the bar to the center of gravity of the compressive stresses, were computed in the usual way. The value of *jd* was generally about 0.87 of the distance from the center of gravity of the bars to the face of the slab.

TABLE 3

RESISTING MOMENTS ACCOUNTED FOR BY STRESSES OBSERVED IN REINFORCING BARS

(Moments are given in pound inches)

NORTH AND SOUTH SECTIONS

Negative Moment		Positive Moment	
Width AB	2 760 000	Width IJ	560 000
Fig. 18			
Width BC	2 720 000	Width JO–NK	1 570 000
Width CD	1 690 000	Width KL	750 000
Total	7 170 000	Total	2 880 000
Width EF	2 950 000	....	....
Width FG	2 890 000	....	....
Width GH	2 030 000	....	....
Total	7 870 000	....	....
Average total negative moment	7 520 000	....	....

EAST AND WEST SECTIONS

Negative Moment		Positive Moment	
Width MN	2 190 000	Width QK	600 000
Width NO	2 950 000	Width KG–FJ	1 340 000
Width OP	1 680 000	Width JT	730 000
Total	6 820 000	Total	2 670 000

Table 3 gives the resisting moments accounted for by the stresses observed in the reinforcing bars as calculated by the method described. A comparison of these moments with the values of the analytical moments already given indicates that about 88 per cent of the analytical negative moment is accounted for by the stresses in the reinforcing bars in the case of the north and south section and 92 per cent in the case of the east and west section. For positive moments 68 per cent of the analytical positive moment is accounted for in the north and south section and 72 per cent in the east and west section. The average percentages for the sections in the two directions are 90 for the negative moment and 70 for the positive moment.

The reinforcing bars are not the only source of tensile resistance in the slab; tensile strength of concrete assists. It is evident that at sections of positive moment tension in the concrete may make up a considerable part of the resistance offered by the slab, and even at sections of negative moment it may have an effect. This tension will be especially influential in regions away from cracks. At points of the sections which are outside the loaded area and where the stresses in the bars are small it may be expected to form a not inconsiderable part of the total resisting moment developed. Of course, if all panels were loaded, the effect of tension in the concrete would be much less, since its greatest proportional effect is outside the loaded area.

In making comparisons, it must be kept in mind that the unit-strain computed from the measurements gives the average strain over the gage length and that at a crack the stress in the bar will be more than the average stress over the gage length.

Measurements made in beam tests in various laboratories generally show that up to loads near the ultimate load the measured stress in the reinforcing bars is less than that necessary to account for the full bending moment due to the applied load. At the lower stresses the deficiency is considerable; at stresses near the yield point of the steel it may not be much, and the measured stresses may even be larger than necessary to account for the bending moment. The effect is particularly noticeable in concrete of high quality.

On the whole the analytical values of the moments are closely approached, as closely as may be expected in tests of this kind. The negative moment of course is most fully accounted for. The positive moment is not wholly accounted for; but as the section of positive moment involves relatively low stresses the effect of the tensile resistance of the concrete on the measured stresses may be considerable, especially in the part of the slab outside the loaded area. It must not be overlooked, also, that the stress in the bars at the cracks will be larger than the average stress over the gage length.

It should be noted that in the case of negative moments the magnitudes of the moments found for the part of the sections adjacent to the central column are smaller than those for the corresponding parts of the sections adjacent to the outer columns. In the case of column 15, part of this difference is due to the additional amount of reinforcement furnished by the extra laps at this place, and it should be noted too that the larger number of laps at the outer columns makes this portion of the slab stiffer than the portion around the central column. In general, however, most of it appears to be due to the stresses developed in slab and reinforcing bars outside the loaded area, a part of the slab which has not usually been considered to contribute to the resisting moment. In the sections of positive moment, the magnitudes of the resisting moments found are less proportionally at the ends of the sections than at the middle, a condition which may be due to the greater proportional effect of the tensile resistance developed in the concrete in these regions.

The foregoing comparisons have been made on the basis of the full analytical value of the bending moment and by considering one-third of it as positive moment and two-thirds as negative moment. The Joint Committee on Concrete and Reinforced Concrete recommended for the sum of the positive and negative moments a value which is about 85 per cent of the analytical value heretofore used and recommended that the distribution be three-eighths positive moment and five-eighths negative moment. It may be of interest to note that the sum of the positive and negative moments accounted for by the measured stresses in the reinforcing bars has nearly the same value as the sum of the moments recommended by the Joint Committee. The negative moments so accounted for are 13 per cent higher than the value recommended by the Joint Committee, and the positive moments are about 73 per cent of the committee's values. In judging of the results, it should be remembered that at the section of positive moments tensile stresses in the concrete have a considerable influence at the stage of the loading indicated by the stresses in the bars in the loaded area and outside of it. The reference to the tensile resistance of the concrete as contributing to the resisting moment of the slab in the test should not be taken to mean that it will be effective as a resistance when the ultimate load is reached. It is apparent, also, that requirements less than those of the Joint Committee will not provide for all the moment developed by the bars of the slab.

In making a comparison with methods used in design it should be borne in mind that the principal observed maximum stresses were from 15 to 25 per cent greater than the average of the observed stresses which were used in computing the resisting moments accounted for by the stresses in the bars; in designing, a uniform stress over the section is assumed.

14. *Bending of Columns.*—A few gage lines were placed on columns bordering the loaded area. In Fig. 19 the location of these is shown, together with the load-strain diagrams. Although for some

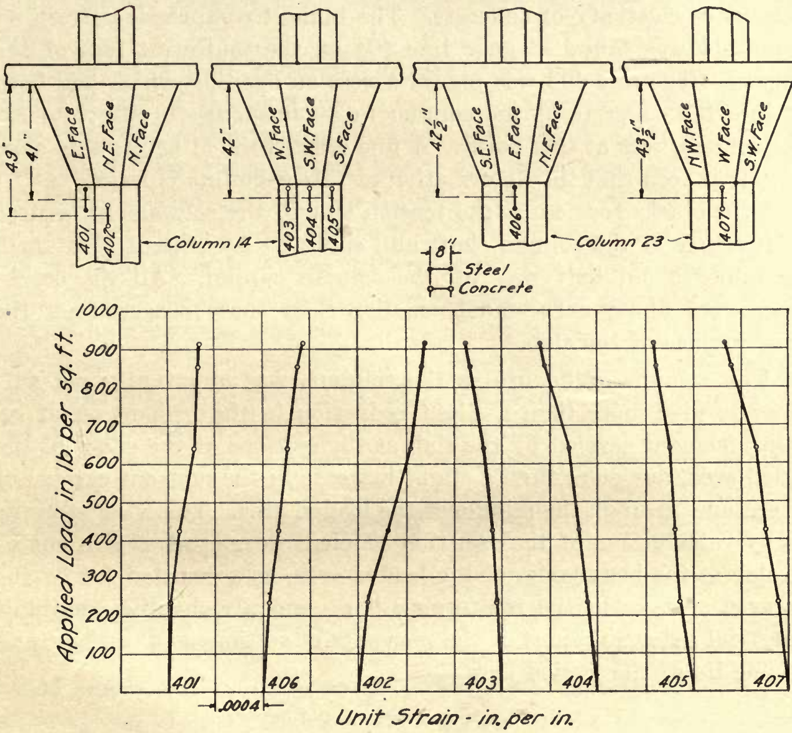

FIG. 19. LOAD-STRAIN DIAGRAMS FOR GAGE LINES LOCATED ON COLUMNS

of the gage lines the strains show as tensile strains, it is probable that for the gage lines given as in tension the compressive strain in the column due to the weight of the floors above was sufficient to overcome the tensile strain measured by the instrument.

The direction of the bending was in the direction which would be expected from the condition of loading. For gage line 406 on a column reinforcing bar the deformation at the load of 913 lb. per sq. ft. corresponded to a flexural tensile stress of 10 500 lb. per sq. in. and for gage line 407 on the concrete on the opposite face of the column the strain was 0.00054 in. per in. corresponding to a stress of 2160 lb. per sq. in. with a modulus assumed as 4 000 000 lb. per sq. in. Similarly at gage line 401 on a column reinforcing bar at the east face of column 14 a stress of 8100 lb. per sq. in. was found, and for gage line 403 on the concrete on the opposite face of the column the compressive stress was 1200 lb. per sq. in. based on the same modulus of elasticity of concrete. The highest compressive strain on column 14 was found at gage line 404 on the southwest face of the column. Gage line 402 was placed across an opening which had been cut into the concrete of the column in an unsuccessful effort to expose column bars at this point. A fine crack formed across this gage line. It is seen that the deformation at this gage line is large.

Fine cracks formed on the tension side of the columns at or near the juncture of the column shaft and capital. At column 15 a crack was found about half way up the column capital. All the cracks noted were fine cracks—much smaller than those observed on the lower surface of the slab.

The amount of flexure in the columns was apparently not sufficient to give more than a slight reduction in the proportion of resisting moment carried by the slab at the sections at the edges of the loaded area, nor more than a slight increase in the moment carried at the sections through the middle of the loaded area. This view is borne out by calculations of the resisting moment developed at sections of the slab at the boundaries of the loaded area, as accounted for by the observed stresses in the reinforcing bars, and also by the results of analytical determinations of the moments at sections at the edges and the middle of the loaded area.

15. *Time Effect of Load and Recovery upon Removal of Load.*—As has been stated, two or more sets of strain gage and deflection readings were generally taken after an increment of load had been applied,—the first set immediately after the last of the load had been placed and another set 12 hours thereafter. A third set was taken 66 hours after the load of 913 lb. per sq. ft. had been applied.

The effect of time on the strains in the steel and concrete was very small; the reading in a few of the gage lines increased slightly after 12 hours, but generally not enough to allow two points to be plotted on Fig. 11 to 13 and not more than may be considered to be within the error of observation. This was true for both steel and concrete even in the most highly stressed places at the maximum load after a period of 66 hours had elapsed.

The deflection readings were affected but little through the 12-hour period until a load of 637 lb. per sq. ft. was reached; Fig. 8 shows that at this load a marked change in deflection occurred at the centers of panels B, C, and D in the 12 hours' time. At the maximum load the increase in deflection through the 66-hour period was small.

Four days were consumed in removing the load. Four hours after the last of the load was removed, readings were taken on the gage lines and deflection points, and twenty hours later another set was taken. The cracks in both upper and lower surfaces of the slab had closed so that they were not easily traced. The recovery in deflection is given by the plotted points at the bottom of the diagrams in Fig. 8. The recovery at the centers of the panels was about 75 per cent; at other points it was generally greater. The recovery in strains in steel and concrete was also large. Even where high stresses and open cracks had been observed the residual strain in the steel was not more than that to be expected from the lack of interlocking of particles at the cracks. In the region of high compressive stress in the concrete the recovery at the principal gage lines was 75 per cent or more; in the reinforcing bars the recovery averaged about 80 per cent. In general the action of the slab was that of concrete of high quality.

16. *General Comments.*—Although there were differences in the stresses and deflections found at corresponding points in the four loaded panels, these differences can generally be accounted for by (1) difference in the arrangement and number of the reinforcing bars, (2) differences in position of the bars with respect to the surfaces of the slab, and (3) differences in strength and stiffness of concrete in the four panels. The large amount of steel in the region of the column capitals bordering the loaded area (where there were two or three bands lapped) added greatly to the stiffness of the slab in this

region. The effect of this added stiffness was to cause higher relative stresses to be developed at the columns bordering the loaded area, and relatively large negative resisting moments were developed at columns where lapped bars were numerous. Another effect of the lapping of bars at columns and in certain of the rectangular bands was to decrease the deflection of the slab in the panels affected by the laps. The influence of the position of the reinforcing bars has already been discussed. Higher stresses in reinforcing bars and greater deflections in the slab were found in the panels for which the compression tests showed weaker and less stiff concrete. Cracks, also, were wider and more numerous in these panels. It is apparent that the tensile resistance of the concrete at sections of positive moment, particularly near the outer edges of the loaded panels, contributed to the resistance of the slab even at the maximum load.

With reference to the design of the slab, it may be noted that the slab was strongly reinforced, though the reinforcing bars were not distributed to the best advantage and the laps were not placed so as to be fully effective. Taking the total reinforcement found over the loaded area and immediately outside, and including such lapped bars as had sufficient anchorage beyond critical sections to be effective (there were many lapped bars which were not counted as effective), and considering the thinness of the slab, the amount of reinforcement for negative moment was on the average as much as that required for the negative moments recommended by the Joint Committee on Concrete and Reinforced Concrete. The amount of reinforcement available for positive moment was on the average more than 50 per cent greater than that required for the positive moments recommended by the same committee. The distribution of the reinforcing bars was, however, quite different from that recommended by this committee. It may be noted also that the amount of reinforcement was greater than that required by most building regulations.

Although the nominal thickness of the slab (8½ in.) was less than that required by building regulations, the slab fulfilled the common requirements for compressive and shearing stresses in concrete of the high quality shown in the tests of prisms taken from the slab. The provisions of the Joint Committee on Concrete and Reinforced Concrete for bending moments and working stresses in concrete of 3000 lb. per sq. in. strength give a thickness about the same as the designed thickness of the slab.

In view of the large number of lapped bars not shown on the building plans, the use of high-carbon steel instead of the mild steel specified, and the unexpectedly high strength of the concrete, it is not strange that the floor carried a much higher load than was anticipated when the test was begun.

17. *Wrecking of the Floor.*—As soon as the load was removed, portions of the floor in the four panels of the loaded area were broken into to uncover the reinforcing bars at important sections. Measurements of the position and location of the reinforcing bars were made for use in calculation. The thickness of the floor was measured at a number of points to check the values obtained from level readings. Photographs were taken to give a record of the position and location of reinforcing bars and their laps; Fig. 21 is a sample. As the wrecking of the floor by the contractor progressed, further measurements of the position of the reinforcing bars and their laps, including those in the bands outside of the loaded area, were taken.

The wrecking of the building by the contractor was an interesting operation. Fires were first built around the columns on the floor below the one to be wrecked; the effect on the concrete at the base of the column after several hours application of heat was to crack and loosen the concrete shell and expose the reinforcement. To assist in cracking the concrete and separating it from the steel, in many cases water was thrown over the columns after they were well heated. A heavy iron pear-shaped weight (about 1600 pounds) was dropped on the floor immediately over the column capital close to where the column of the story above had been. After the column capital had been sheared and shattered by this operation, the portion of the floor surrounding the column and that directly between columns was broken up with the weight and with sledge hammers. After the concrete of sections of the floor had been removed in this way, the reinforcing bars were cut with the oxyacetylene flame. Many of the bars were taken out in good condition. The process was continued until the entire floor was wrecked. The longitudinal reinforcing bars in the column were then cut near the floor below and the columns were pulled down on the floor and broken up with the heavy weight and hammers.

18. *Summary.*—The following deductions have been drawn in the foregoing presentation of the results of the test:

(1) The tests of samples of the concrete from the slab, as well as the hardness and toughness of the concrete observed in breaking up the slab, indicate that the concrete was of unusually good quality and that it had high strength and stiffness. The action of the slab under load was that to be expected with high grade, well-seasoned concrete. The effect of time on the stresses in steel and concrete and on the deflection of the slab under a sustained load was slight, even over a period of 66 hours under the maximum load of 913 lb. per sq. ft.,—conditions which would not exist at an early age of concrete. Upon removal of load, the recovery in deflection at the centers of the panels was about 75 per cent of that under load, and at other points generally more; the recovery in strains in steel and concrete was as large.

(2) The position of the important cracks on both upper and lower sides of the slab may be expected to indicate the region of high tensile stresses in the reinforcing bars; it is also an indication of the general action of the slab in flexure. The cracks on the upper side at the load of 913 lb. per sq. ft. opened to a width of 0.02 to 0.06 in.; those on the lower side were not so wide. Upon removal of the load the cracks closed, leaving the surfaces of the slab with the appearance which they had before the load was applied.

(3) For reinforcing bars in the upper side of the slab in the regions of negative moment, the stresses in bars of diagonal bands were greater than those in bars of rectangular bands, a stress of 57 300 lb. per sq. in. being observed in a diagonal bar and one of 42 000 lb. per sq. in. in a rectangular bar at the maximum load. Stresses were found in both rectangular and diagonal bars at the columns bordering the loaded area nearly as great as those at corresponding points at the central column. Stresses of some magnitude were found in bars outside the loaded area. The stresses given do not include the stress due to the load of the slab itself.

(4) For reinforcing bars in the lower side of the slab in the regions of positive moment, the stresses in bars of rectangular bands were greater than those in bars of diagonal bands even though the former were farther above the lower surface of the slab than the latter; in the one apparent exception, the presence

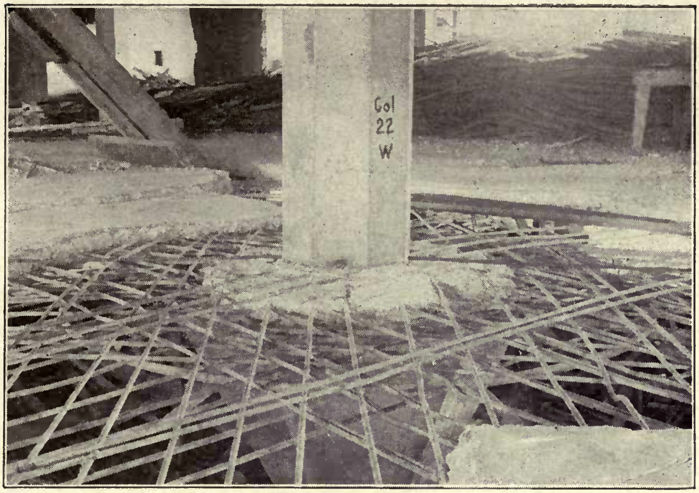

FIG. 20. VIEW SHOWING ARRANGEMENT OF REINFORCEMENT AT COLUMN 22

FIG. 21. VIEW OF SLAB AFTER REINFORCING BARS WERE EXPOSED

of laps doubled the usual number of bars. At the maximum load stresses of 24 000 to 30 000 lb. per sq. in. were observed in bars of rectangular bands and of 20 000 to 24 000 lb. per sq. in. in bars of diagonal bands. A stress of 15 600 lb. per sq. in. was observed in a bar outside the loaded area at the edge of a rectangular band.

(5) On the upper surface of the slab the greatest compressive strains were found at gage lines along the inner panel edges midway between columns, ranging from 0.0009 to nearly 0.001 in. per in. at the maximum load; at the centers of the panels values about half as great were found. On the lower surface the greatest strains were found at the middle column; these ranged from 0.0012 to 0.0016 in. per in., values which are as great as the strains found at failure in the tests of the concrete prisms cut from the slab and as great as are ordinarily found in compression tests of concrete at the ultimate load. In some places there was chipping and spalling of the concrete. It is evident that the action of the surrounding concrete assisted in preventing failure. At gage lines crossing the inner panel edges high compressive strains also were found, even though there was no tension reinforcement in the upper side of the slab in this region. It may be noted that the maximum compressive strains on the upper surface of the slab were one-half to three-fifths those found on the lower surface. The intensity of the strains at various points along sections of positive moment and negative moment will give some measure of the distribution of intensity of moments along those sections.

(6) The observed stresses in the reinforcing bars accounted for about 90 per cent of the analytical negative moment and about 70 per cent of the analytical positive moment, as given by the methods used. It should be noted that the observed stresses used are average stresses over the gage length, and the stress at a crack may be expected to be greater than the average over the gage length. It seems probable that tensile resistance of the concrete contributed to the resistance of the slab, particularly in the sections of positive moment and in regions near the edges of the loaded area. A similar influence of the tensile resistance of concrete, when the stresses in the steel are well below its yield point, has been observed in numerous beam tests. That the tensile re-

sistance of the concrete contributed to the resisting moment of the slab in the test should not be taken to mean that it will be effective in resisting moment when the ultimate load is reached. It may be noted also that the sum of the positive and negative moments accounted for by the measured stresses in the reinforcing bars has almost the same value as the sum of the positive and negative moments recommended by the Joint Committee on Concrete and Reinforced Concrete, and that the negative moments so accounted for are about 113 per cent and the positive moments about 73 per cent of the moments recommended by this committee. In making a comparison with methods used in designing it should be borne in mind that the principal maximum stresses were from 15 to 25 per cent greater than the average stresses which were used in computing the resisting moments accounted for by the stresses in the bars; in designing, a uniform stress over the section is assumed.

Although the arrangement of bars was not as recommended, the amount of reinforcement for negative moment, considering all available bars over the area used, was as much as that required for the negative moments recommended by the Joint Committee on Concrete and Reinforced Concrete, even though the slab was thinner than recommended for ordinary concrete. The amount of reinforcement for positive moment was more than 50 per cent greater than that required for the positive moments recommended by the committee. Although the nominal thickness of the slab was less than that required by building regulations, it fulfilled the provisions of the committee for bending moments and working stresses for concrete of a test strength of 3000 pounds per square inch.

(7) The action of the floor slab under test should give added confidence in the suitability and reliability of the flat slab as a load-carrying structure.

LIST OF
PUBLICATIONS OF THE ENGINEERING EXPERIMENT STATION

Bulletin No. 1. Tests of Reinforced Concrete Beams, by Arthur N. Talbot. 1904. *None available.*

Circular No. 1. High-Speed Tool Steels, by L. P. Breckenridge. 1905. *None available.*

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Circular No. 2. Drainage of Earth Roads, by Ira O. Baker. 1906. *None available.*

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