THE EFFECT OF THE COLUMNS ON THE MOMENTS IN FLOOR SLABS WITH SPANDRELS DUE TO VERTICAL LOADS

by

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I. INTRODUCTION

Today, with the cost of construction steadily rising and our resources of raw material being consumed at such an alarming rate, it is of the utmost importance that the designer have a complete knowledge of his structure so that he may design with as little waste as possible.

The design trend at the present day for building structures leans toward a greater use of reinforced concrete. The use of reinforced concrete in most cases results in a continuous frame structure. Consequently, there is in such structures a reduction of critical moments and shears in the slab and supporting beams, and an increase in column moments and shears, as compared with simply connected structures.

The design procedures for this type of structure in general are liberal. This is because of our limited knowledge of the behavior of the structural frame.

With the advent of the electrical resistance strain gauge we are able to determine the stresses and strains of the indeterminate type of structure more readily and, in consequence, are able to be more accurate in our analysis, and to attain a greater understanding of the frame's behavior.

The purpose of this thesis is to construct and load a scale model of a continuous frame, a slab supported by spandrel beams and columns, and to show the distribution of the stresses in the slab and into the beams and columns.

The results of this investigation are shown with tables and curves. From those tables and curves conclusions have been drawn insofar as the information warranted.

II. PRELIMINARY DESIGN

The model as shown in Figure 1 was governed by several factors. First, it was decided to use as large a model as possible, and the largest model that could be tested in the testing machine was a 28-inch square model.

As the model was to be used in the experimental work for the first thesis by Eugene L. Miller, "The Effect of Floor Slabs on the Moments in Columns",[1] and than for this thesis, "The Effect of the Columns on the Moments in Floor Slabs with Spandrels Due to Vertical Loads", it was designed to satisfy several diverse sets of requirements.

Structural steel was chosen as the material to be used in the construction of the model because of the ease of fabricating the model. Another deciding factor was that electrical resistance strain gauges could be fastened to the model without any difficulty. This model can be compared with a reinforced concrete structure since the behavior of the two are similar, if the proper allowance is made for the difference of the two materials.

As the model was to be typical of a building bay, it was decided to place the columns so the slab would have an overhang on all four sides, which tended to give the slab a certain degree of continuity. Before determining the

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[1] Miller, Eugene L. "The Effect of Floor Slabs on the Moments in Columns." Thesis, Virginia Polytechnic Institute, Blacksburg, Virginia. 1952.

exact location of the columns, it was necessary to determine the number and location of the electrical resistance strain gauges and the loading positions. Since the model was symmetrical, it was necessary to place the gauges in one quadrant only. By choosing a quadrant seven inches by seven inches, it was possible to give the overhanging portion of the slab a width equal to one half the plate width. The maximum number of rectangular rosette type gauges that could be fastened to the slab and leave sufficient room for loading points was 16. This gave a grid of 1/6 for the gauges on the slab. Now by loading at the 1/4 points on the slab with concentrated loads, we could get the effect of a uniform load with very little error. This can be shown by loading a simple beam at the 1/4 points and comparing the resulting moment to the moment of a uniformly loaded beam.

The overall dimensions of the model were determined by proportioning them to a typical reinforced concrete building. The spandrel beams were slightly heavier than would be expected in a typical structure. These heavier beams were used in the model to make the effect of the beams quite marked and to make the beams more nearly consistent with the one inch columns used.

The model as shown in Figure 1 consisted of a square plate 28 inches by 28 inches and 1/2 inch thick, supported by beams and columns. The columns were one inch in diameter, 10 inches long and on 14 inch centers.

In order to prevent the columns from displacing laterally at the bottom when the load was placed on the model, some means had to be devised to hold the base of the column in position and also allow the column to rotate at the base. This was accomplished by using a common base plate 17 inches by 17 inches by 1/2 inch. The column ends were rounded and set in countersunk holes. It was hoped that the friction developed here against rotation would be very small.

The results of the preliminary design are shown in Figures 1, 2, and 3.

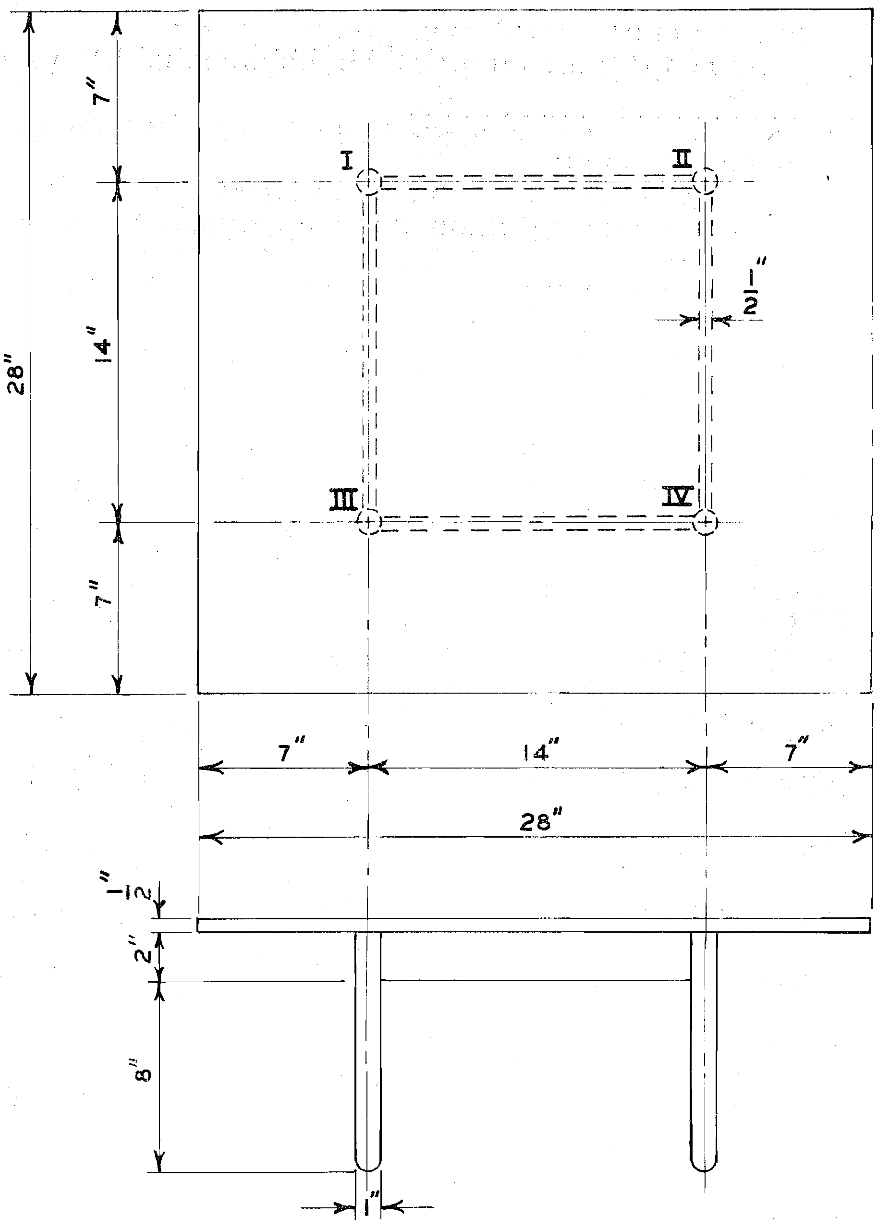

FIGURE 1
THE MODEL

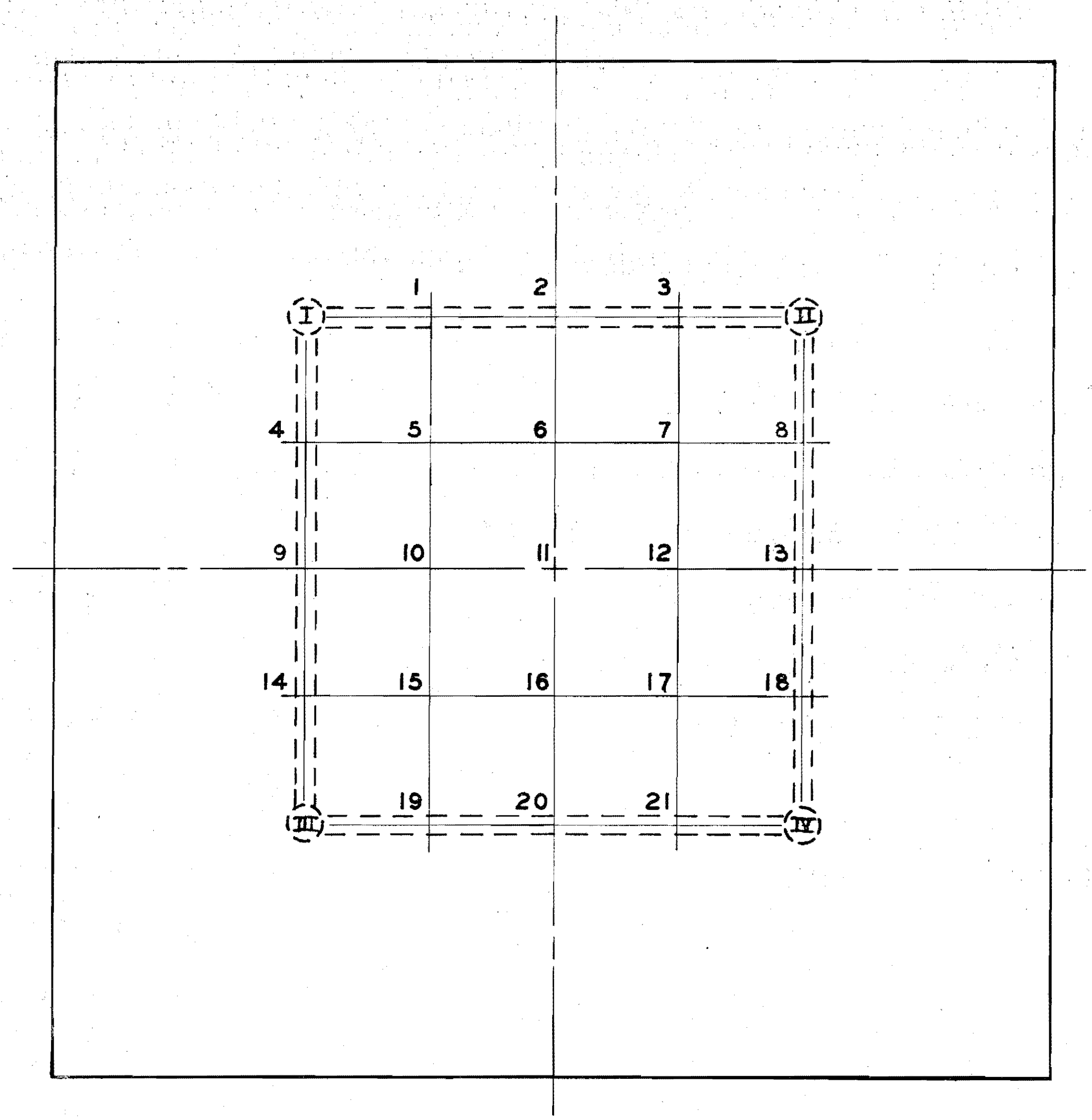

FIGURE 2

THE LOADING POSITIONS

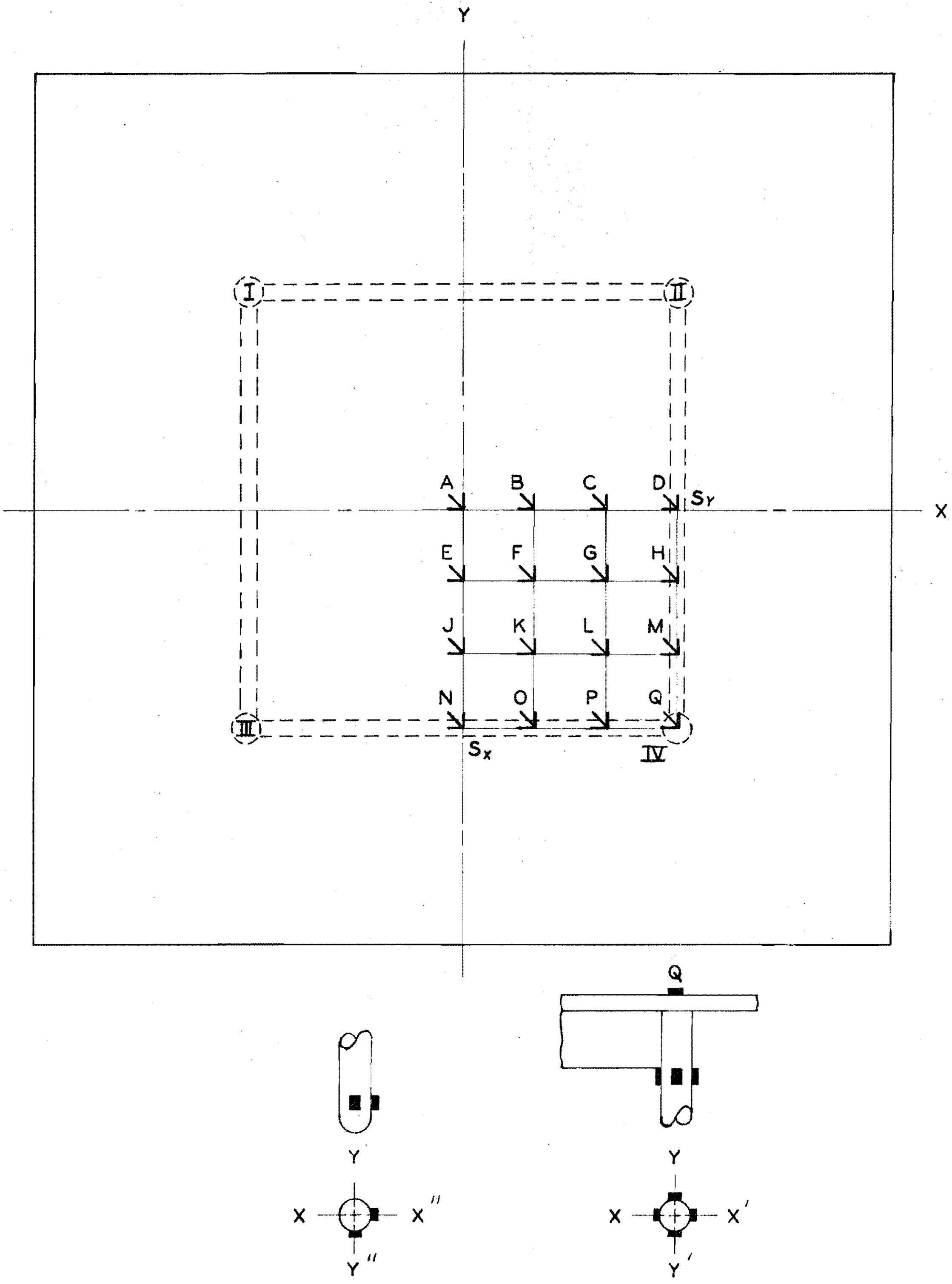

FIGURE 3

LOCATION OF STRAIN GAUGES

III. CONSTRUCTION OF THE MODEL

As previously stated, the model had been used by Mr. E. L. Miller[2] in the preparation of his thesis. Since the writer is familiar with the difficulties encountered by Mr. Miller,[3] he feels that they should be mentioned here along with his own.

The steel used in the construction of the model was of intermediate grade obtained from the Virginia Bridge Company of Roanoke, Virginia. The slab and base plate were sheared to the dimensions stated in Chapter II. The columns were one inch round bars and cut to approximate length. The model was assembled in the Virginia Polytechnic Institute Machine Shop. The column ends were turned down in a lathe so that the ends were slightly tapered and then rounded at the tips. After this operation, the columns were cut to exact length. The slab plate was placed bottom side up on a flat table and the columns set in position, then the base plate was placed on the columns and clamped into position with the pinned ends seated in the countersinks to prevent movement and to assure correct alignment while the columns were being electrically butt welded to the slab plate. A considerable amount of warping was noticed in the plate immediately after

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[2] Miller, Eugene L. "The Effect of Floor Slabs on the Moments in Columns." Thesis, Virginia Polytechnic Institute, Blacksburg, Virginia. 1952.

[3] Miller, Eugene L. Ibid.

welding the columns. As the plate cooled, the greater part of this warping disappeared leaving the plate slightly warped on the overhanging edges and the columns slightly out of alignment.

Since the base plate was sheared, a slight warping was noticed in the plate. To bring the base plate to a horizontal plane it was decided to weld buttons on the underside of the plate opposite the countersunk holes and to finish the buttons to a single horizontal plane in a milling machine. This is the model that Mr. Miller[4] used in the investigation for his thesis.

The beams that were to be welded to the model for the investigation of this thesis were beveled on both ends and along the edges that were to be welded to the underside of the slab plate. The model was then taken back to the shop and the beams were welded continuously along the underside of the slab plate and at the ends adjoining the columns. Due to the high amount of heat dissipated into the plate while the beams were being welded, a considerably greater amount of warping took place than had previously occurred. The warping was so great that the columns would not seat in the countersinks of the base plate. It was decided to use hydraulic jacks to re-align the columns in their countersinks. The base of the hydraulic jack was placed against the

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[4] Miller, Eugene L. "The Effect of Floor Slabs on the Moments in Columns." Thesis, Virginia Polytechnic Institute, Blacksburg, Virginia. 1952.

beams for support while forcing the columns back into proper alignment. Several operations were required for each column. At the end of each operation, the columns were checked for alignment. This method proved very successful on all columns except one. It was felt that this column would seat itself under initial loading of the model.

IV. PREPARING THE MODEL FOR INVESTIGATION

The first step necessary in preparing the model for the investigation was to clean the scale off the plate where the gauges were to be fastened. This was accomplished by means of a wire brush and by rubbing with emery cloth. The grid lines for the location of the gauges and the loading positions were then scribed on the plate. The rectangular rosette gauges were glued to the underside of the plate except for the gauges where the beams and the column interferred with the grid positions. These gauges (noted as D, H, M, N, O, P, and Q in Figure 3) were placed on the top side of the plate in their respective grid position. Four linear type gauges were glued symmetrically around the top of the column. Two linear type gauges were glued near the pinned end of the column and two other linear type gauges (noted as Sx and Sy in Figure 3) were glued at the center on the underside of the beams framing the quadrant which contained the rosette gauges. Figure 3 shows the location of these gauges.

The lead wires used to connect the gauges to the recording apparatus were No. 22 copper wire. These leads were approximately 36 inches long and were carefully tagged at each end so as to eliminate error. A twenty-four hour waiting period was necessary at this time to allow the gauges to thoroughly dry. After this waiting period, the model was placed in the testing machine and all leads connected to the recording apparatus and the gauges were tested. A test load of four thousand pounds was used in checking the gauges. The loading position for checking the gauges was at 11. Several gauges were found to be defective. These gauges were removed and replaced and the new gauges allowed to dry for the prescribed period. Again the gauges were checked and were all found to be in good order.

While checking the gauges, it was noted that the column which was slightly out of alignment would seat itself properly under a load varying from 600 to 1,000 pounds depending upon the loading position.

If the loading pin was moved to any of the loading positions other than 11, the two columns farthest away from the loading pin tended to rise out of their seats. Since no method could be devised to anchor the columns in their seats, it was deemed necessary to load the model with symmetrical pairs. This greatly reduced the loading time and cut the number of loading positions in half. To load the model by symmetrical pairs, an I-beam with two moveable loading pins placed at equal distances from a fulcrum at the center of the beam was used. By using this method, the model could be loaded at all loading positions without moving the model or the I-beam. It was only necessary to rotate the I-beam about the fulcrum point and place the loading pins at their respective symmetrical loading positions.

V. THE INVESTIGATION

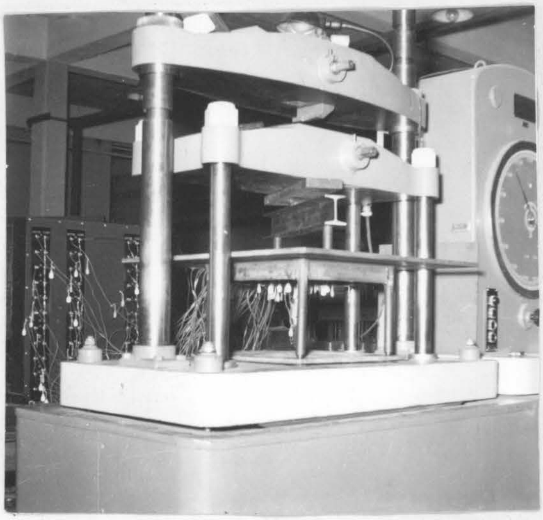
As previously mentioned, it was necessary to apply an initial load of from 600 to 1,000 pounds to seat one of the columns properly. With this initial load, all the rosette gauges and linear gauges on the beams and columns, and the compensating gauge were zeroed in with a reading of 1,000 micro-inches per inch on the strain recording apparatus. By choosing 1,000 micro-inches per inch, positive and negative strains could be recorded. If the reading was greater than 1,000 micro-inches per inch, the strains were positive. If the readings were less than 1,000 micro-inches per inch, the strains were negative.

After zeroing the gauges in, a load of two to three thousand pounds was applied to the model and the gauges were checked to determine the gauge of the greatest reading. The critical gauge having been determined, the load was increased until the strain recorder indicated a reading of approximately 1,000 micro-inches per inch to either side of the zeroing point. This produced a stress at the gauge location of approximately 30,000 pounds per square inch. The load was released to 1,000 pounds and all the gauges were checked to see if they were zeroed in. It was found that several gauges indicated creep. This appeared due to the nature of the gauge, or the glue which secured the gauge to the plate, or possibly the wire connections. The load was again applied and increased until the critical gauge showed a reading of approximately 1,000 micro-inches per inch positive or negative. The load was then released again and all gauges checked. If no creep was indicated, the critical load was than applied and all gauge readings recorded. This method of loading and checking was used for all the loading positions.

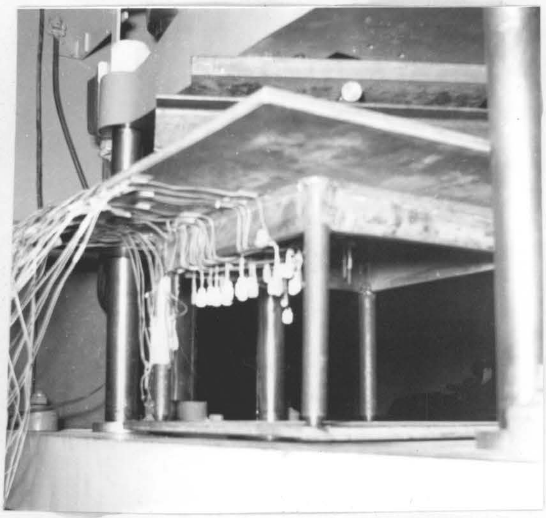
Plate 1 shows the loading technique used, the loading machine, the gauges and wire leads fastened to the model, and the recording apparatus.

It was decided to load the model at loading position number 11 with a load of 5,500 pounds and to record the deflections of the plate and the beams. This was done by the use of extensometers. It was felt that these deflections would be of value to future investigations dealing with this subject. Figure 4 shows the deflections in inches and the location at which they were recorded.

The Loading Technique



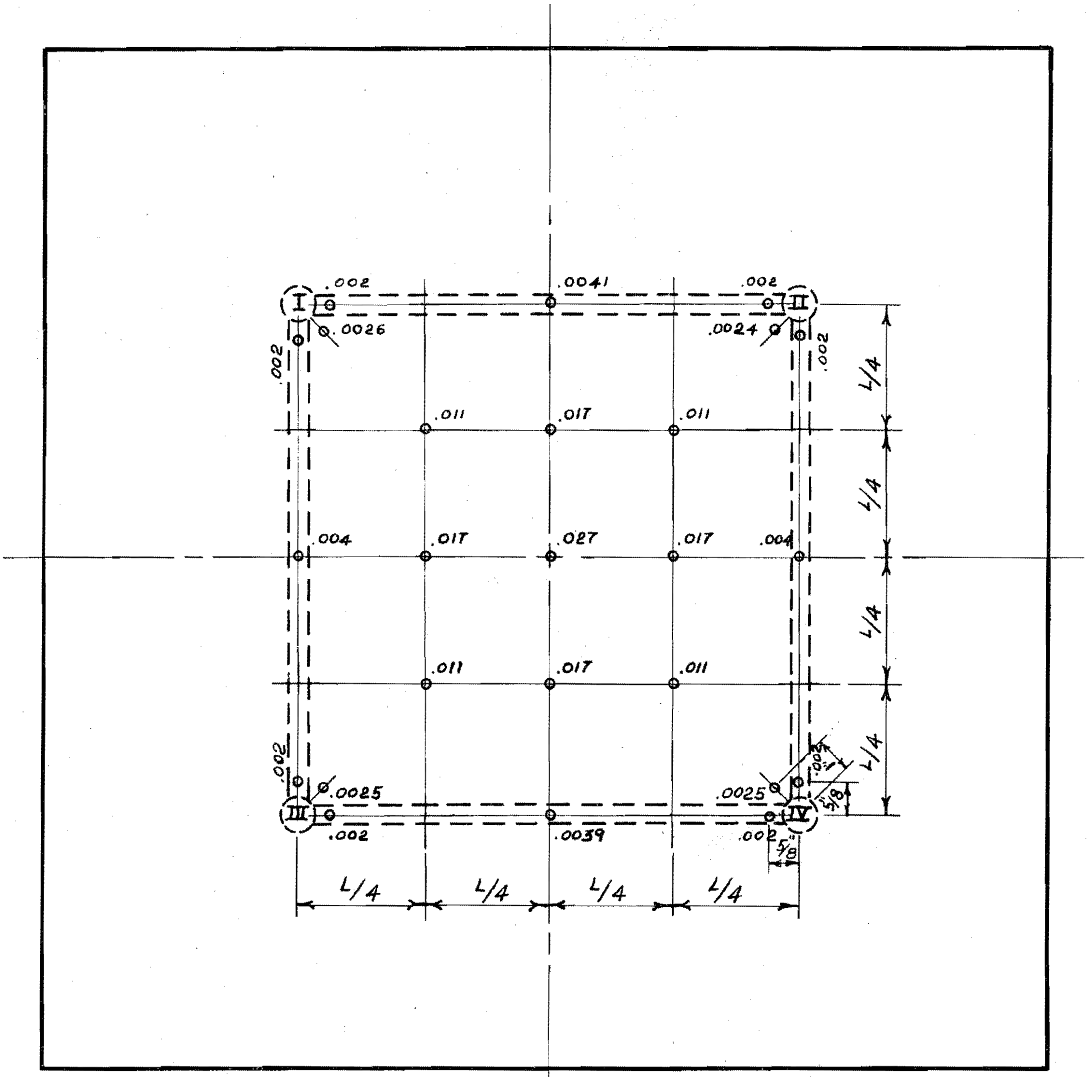
The Loading Machine



Wire Leads Connecting Strain Gauges and Recording Apparatus

The Recording Apparatus

PLATE



DEFLECTIONS IN INCHES.
LOAD OF 5,500 POUNDS AT POSITION II

FIGURE 4
SHOWING DEFLECTIONS

VI. RESULTS

The results of this investigation appear in Tables 1 and 2 and Figures 5 through 10.

The strains given in Table 1 are the measured surface strains recorded for the various loading positions and altered as stated below. As previously stated, gauges D, H, M, N, O, P, and Q were fastened to the top side of the plate. In order to get the effect of having all the gauges on the underside of the plate, it was necessary to change the sign of the recorded strains of the gauges on the top side of the plate. These converted readings appear in Table 1 as being recorded strains. To get the effect of a uniform load on the model, it was necessary to proportion the recorded strains to a common load since the various loading positions did not have loads of equal size. After proportioning all the recorded strains to a common load of 5,500 pounds, the effect of a uniform load was obtained by adding algebraically the strains for the loading positions.

Table 2 shows the maximum and minimum principal stresses and their directions. The principal stresses are in thousands of pounds per square inch, and their directions in degrees measured counter-clockwise from the "X" axis for positive angles and clockwise for negative angles. The maximum and minimum principal stresses were determined from the following formula:[5]

$$\epsilon_{p,q} = \frac{\epsilon_x + \epsilon_y}{2} \pm \frac{\sqrt{2}}{2}\sqrt{(\epsilon_x - \epsilon_{xy})^2 + (\epsilon_{xy} - \epsilon_y)^2}$$

Then correcting for Poisson's ratio the maximum and minimum principal stresses, P and Q respectively, were obtained from the following formula:[6]

$$P = \frac{E(\epsilon_p + \nu\epsilon_q)}{1-\nu^2}$$

$$Q = \frac{E(\epsilon_q + \nu\epsilon_p)}{1-\nu^2}$$

P and Q are the maximum and minimum principal stresses respectively; ν is Poisson's ratio (.30 for steel); E is the modulus of elasticity for the steel (30,000,000 psi); ϵ_x, ϵ_{xy}, and ϵ_y are the strains recorded by the components of the rectangular rosette gauges. The directions of the maximum and minimum principal stresses were calculated from the following formula:[7]

$$TAN\ 2\theta_{P,Q} = \frac{2\epsilon_{xy} - (\epsilon_x + \epsilon_y)}{\epsilon_x - \epsilon_y}$$

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[5] Froch, Max Mark. Photoelasticity, Volume I. John Wiley & Sons, Inc., New York. 1948.

[6] Froch, Max Mark. Ibid.

[7] Froch, Max Mark. Ibid.

The direction of the maximum and minimum principal stresses are determined by inspection since θ must make an angle of less than 45° with the algebraically greater strain ϵ_x or ϵ_y.

The maximum and minimum principal stresses as given in Table 2 are shown in Figures 5, 6, 8, and 9. The curves shown in these figures were obtained by connecting points of equal stress.

Figures 7 and 10 show the isoclinics for the single load at 11 and the uniform load respectively. The isoclinics were obtained first by plotting the angle θ as given in Table 2. The direction of the maximum principal stress must make an angle of less than 45° with the greater algebraically normal strain ϵ_x or ϵ_y. The direction of all the maximum principal stresses were plotted for all the gauge locations. Since the minimum principal stress is perpendicular to the maximum principal stress, its direction is then known. Now measuring from the positive end of the "X" axis in a counter-clockwise direction to the nearest principal stress, we obtain the direction of one of the principal stresses. By constructing curves through points of equal angles, the isoclinics as shown in Figures 7 and 10 were obtained.

TABLE I

RECORDED STRAINS

GAUGE	LOAD APPLICATION											
	1 & 21			2 & 20			3 & 19			5 & 17		
	ϵ_x	ϵ_{xy}	ϵ_y	ϵ_x	ϵ_{xy}	ϵ_y	ϵ_x	ϵ_{xy}	ϵ_y	ϵ_x	ϵ_{xy}	ϵ_y
A	40	0	-15	60	0	-25	40	30	-25	130	0	150
B	35	-10	-10	20	-5	-35	10	10	-30	200	80	65
C	25	20	0	-5	0	-10	10	5	5	75	160	20
D	20	10	5	35	15	-15	30	10	-20	-145	60	110
E	20	-15	-15	45	5	-30	30	35	-20	75	20	130
F	28	0	-23	25	15	-30	10	20	-5	330	130	350
G	20	20	0	10	10	-20	5	10	10	180	295	235
H	23	23	-5	20	20	0	25	15	-10	-180	-70	145
J	10	-10	-5	65	20	-10	20	35	10	5	135	70
K	28	-10	-10	40	30	5	10	10	-5	255	300	190
L	10	28	-15	-5	-5	0	-5	-5	-5	120	-230	130
M	20	28	0	20	20	0	15	20	-10	-160	-240	85
N	110	-40	-28	395	105	-80	70	55	-30	140	50	-150
O	150	65	-28	195	55	-50	65	40	-20	150	-100	-290
P	100	55	-10	70	35	-25	25	25	-5	60	-255	-170
Q	-55	-28	-10	-40	-25	-10	-20	0	-10	30	-85	35
S_x	380			730			370			370		
S_y	-15			-15			-20			360		
X	-295			-300			-170			-400		
Y	-235			-190			-115			-235		
X′	-60			20			20			105		
Y′	-110			-40			10			-30		
X″	-210			-115			-85			-170		
Y″	-195			150			-70			-180		

RECORDED STRAINS IN MICRO INCHES PER INCH

TABLE I CONT.
RECORDED STRAINS

GAUGE	LOAD APPLICATION											
	6 & 16			7 & 15			11			UNIFORM		
	ϵ_X	ϵ_{XY}	ϵ_Y	ϵ_X	ϵ_{XY}	ϵ_Y	ϵ_X	ϵ_{XY}	ϵ_Y	ϵ_X	ϵ_{XY}	ϵ_Y
A	695	385	85	175	305	150	1000	1005	1020	2165	2140	2180
B	220	185	180	195	240	70	105	260	350	935	1385	1415
C	-95	5	100	90	-40	15	-125	7	145	50	562	1000
D	-240	-80	115	-130	-90	110	-165	-40	80	-1148	-10	1190
E	730	600	215	55	170	130	410	200	80	1470	1495	775
F	130	255	350	85	60	75	140	-30	105	1040	740	1015
G	-110	-115	130	5	-105	10	-80	-150	70	160	-220	573
H	-200	-155	105	-45	-90	70	-75	-105	70	-700	-452	950
J	640	360	45	10	-70	70	150	15	-80	985	515	35
K	70	-205	155	10	-100	0	80	-145	-70	593	-195	180
L	-90	-310	20	-30	-150	-25	-45	-210	-50	-45	1174	-35
M	-100	-190	25	-15	-85	25	-28	-120	35	-333	-692	405
N	230	-95	-415	140	-95	-140	55	-60	-200	1235	-45	-1198
O	165	-250	-170	60	-100	-65	60	-125	-130	935	-512	-885
P	50	-240	-45	10	-95	-20	20	-115	-28	352	-712	-348
Q	15	-80	40	15	-35	15	28	-35	28	-17	-421	-12
S_X	650			360			235			3425		
S_Y	380			360			235			3415		
X	300			-260			-215			-2830		
Y	-350			-150			-110			-2450		
X′	35			90			105			275		
Y′	120			10			0			-45		
X″	-155			-100			-70			-955		
Y″	-150			-110			-75			-930		

RECORDED STRAINS IN MICRO INCHES PER INCH

TABLE 2

PRINCIPAL STRESSES & DIRECTIONS

GAUGE	LOAD APPLICATION											
	1 & 21			2 & 20			3 & 19			5 & 17		
	MAX.	MIN.	θ	MAX.	MIN.	θ	MAX	MIN.	θ	MAX.	MIN.	θ
A	1.5	-0.10	-12°	1.63	-0.85	-11°	1.08	-.45	-17°	9.23	2.77	43°
B	1.3	-0.21	-23°	0.32	-0.99	2.5°	0.24	-1.1	23°	7.69	3.8	-19°
C	0.98	0.10	15°	-0.16	-0.55	35°	0.29	-0.15	22°	4.72	-0.44	38°
D	0.76	0.43	-9°	1.4	-0.11	13°	0.80	-0.37	14°	2.63	-4.3	-15°
E	0.65	-0.47	-23°	1.51	-0.44	-1°	1.43	0.85	25°	6.48	2.42	36°
F	0.65	-0.54	-2°	0.71	-0.86	16°	0.52	-0.38	33°	19.44	9.75	43°
G	0.32	0.09	23°	0.27	-0.71	23°	0.26	0.41	-23°	11.20	10.70	-36°
H	0.87	-0.13	23°	1.07	0.21	23°	0.81	0.099	12°	3.18	-4.63	9°
J	0.43	-0.25	-30°	3.07	0.64	-6°	1.15	0.13	39°	4.06	-0.55	-36°
K	0.87	-0.24	-23°	1.98	0.15	12°	0.27	-0.08	23°	11.53	7.55	33°
L	0.65	-0.90	33°	-0.03	-0.25	23°	0.29	0.29	0°	13.62	-2.75	44°
M	0.54	-0.10	29°	1.07	0.27	23°	0.63	-0.35	23°	3.84	-7.05	30°
N	1.6	-9.0	-25°	17.5	2.75	-6°	2.27	-0.57	18°	3.32	-3.75	10°
O	4.7	0.93	1°	8.35	0.99	-4°	2.07	-0.07	10°	2.1	-8.04	-3°
P	3.2	0.65	6°	2.82	0.03	8°	0.98	-0.12	23°	3.74	-7.80	-30°
Q	-0.87	-2.0	5°	-0.80	-1.49	0°	0.28	-1.0	-36°	4.2	-1.32	44°
S_x	11.4			21.9			11.1			11.1		
S_Y	-0.5			-0.5			-0.6			10.8		
X	-8.9			-9.0			-5.1			-12.0		
Y	-7.1			-5.7			-3.5			-7.1		
X′	-1.8			0.6			0.6			3.2		
Y′	-3.3			-1.2			0.3			-0.9		
X″	-6.3			-5.0			-2.6			-5.1		
Y″	-5.9			-4.5			-2.1			-5.4		

PRINCIPAL STRESSES IN THOUSANDS OF POUNDS PER SQUARE INCH

TABLE 2 CONT.

PRINCIPAL STRESSES & DIRECTIONS

GAUGE	LOAD APPLICATION											
	6 & 16			7 & 15			11			UNIFORM		
	MAX.	MIN.	θ	MAX.	MIN.	θ	MAX.	MIN.	θ	MAX.	MIN.	θ
A	23.8	9.9	-1°	10.32	3.7	42°	43.4	43.2	13°	93.8	92.3	34°
B	9.2	8.0	-17°	8.57	2.9	29°	12.8	6.9	-7°	57.8	43.2	-21°
C	2.3	-2.16	-1°	4.61	0	-34°	3.6	-2.7	1°	33.4	11.4	-2°
D	-1.41	-4.02	3°	2.85	-3.7	16°	0.99	-4.7	-1°	27.9	-25.9	1°
E	26.9	13.6	13°	5.93	2.1	-32°	14.3	6.5	-9°	60.0	36.5	24°
F	12.9	7.8	-4°	4.04	3.0	-38°	8.8	1.5	-42°	50.7	37.4	-44°
G	5.8	-3.5	23°	2.96	-2.2	44°	3.8	-4.3	31°	30.0	1.43	35°
H	4.4	-6.25	18°	3.18	-2.1	30°	2.7	-3.1	27°	28.5	-17.9	19°
J	21.6	7.9	1°	4.28	-0.88	37°	5.2	-0.9	5°	33.3	11.0	1°
K	14.2	-2.64	41°	2.59	-2.2	43°	4.1	-3.6	-32°	30.7	2.1	-35°
L	7.7	-8.02	39°	1.42	-4.1	44°	1.7	-5.7	-45°	24.2	-27.9	45°
M	3.98	-5.4	33°	2.36	-1.94	38°	3.3	-2.7	37°	20.5	-17.3	30°
N	5.5	-10.3	-1°	3.95	3.95	-17°	0.11	-6.2	3°	28.9	-27.2	-1°
O	6.8	-7.0	-28°	2.52	-2.86	-28°	1.7	-4.5	-22°	25.4	-23.2	-15°
P	5.8	-5.6	-40°	1.93	-2.36	-40°	2.5	-2.9	-39°	18.5	-18.1	-32°
Q	3.8	-1.21	42°	3.51	-3.51	45°	2.6	-0.33	-45°	8.6	-10.0	45°
S_x	19.5			10.8			7.1			102.8		
S_y	11.4			10.8			7.1			102.5		
X	-9.0			-7.8			-6.5			-84.9		
Y	-10.5			-4.5			-3.3			-73.5		
X′	1.1			2.7			3.2			8.9		
Y′	3.6			-0.3			0			-1.4		
X″	-4.7			-3.0			-2.1			-28.7		
Y″	-4.5			-3.3			-2.3			-27.9		

PRINCIPAL STRESSES IN THOUSANDS OF POUNDS PER SQUARE INCH

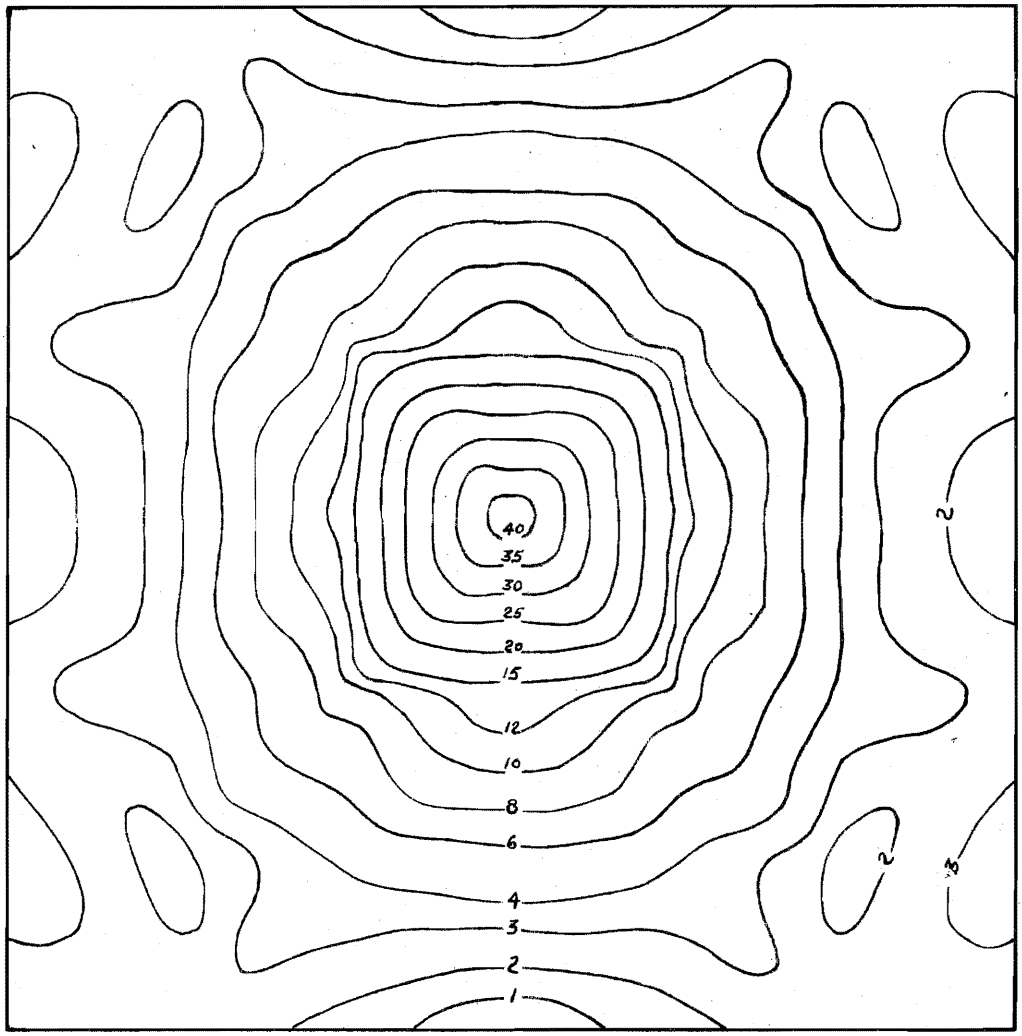

FIGURE 5
SINGLE LOAD AT 11
MAX. PRINCIPAL STRESSES

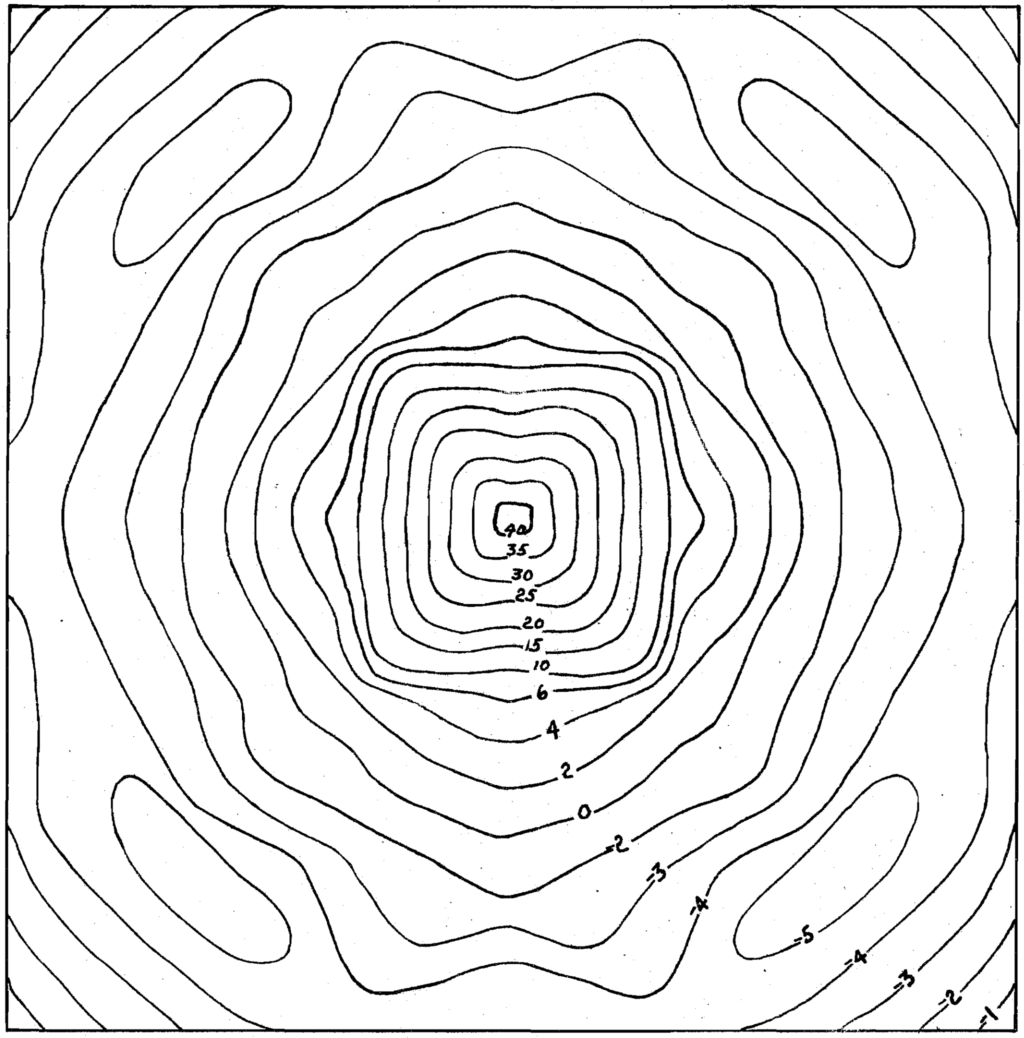

FIGURE 6
SINGLE LOAD AT 11
MIN. PRINCIPAL STRESSES

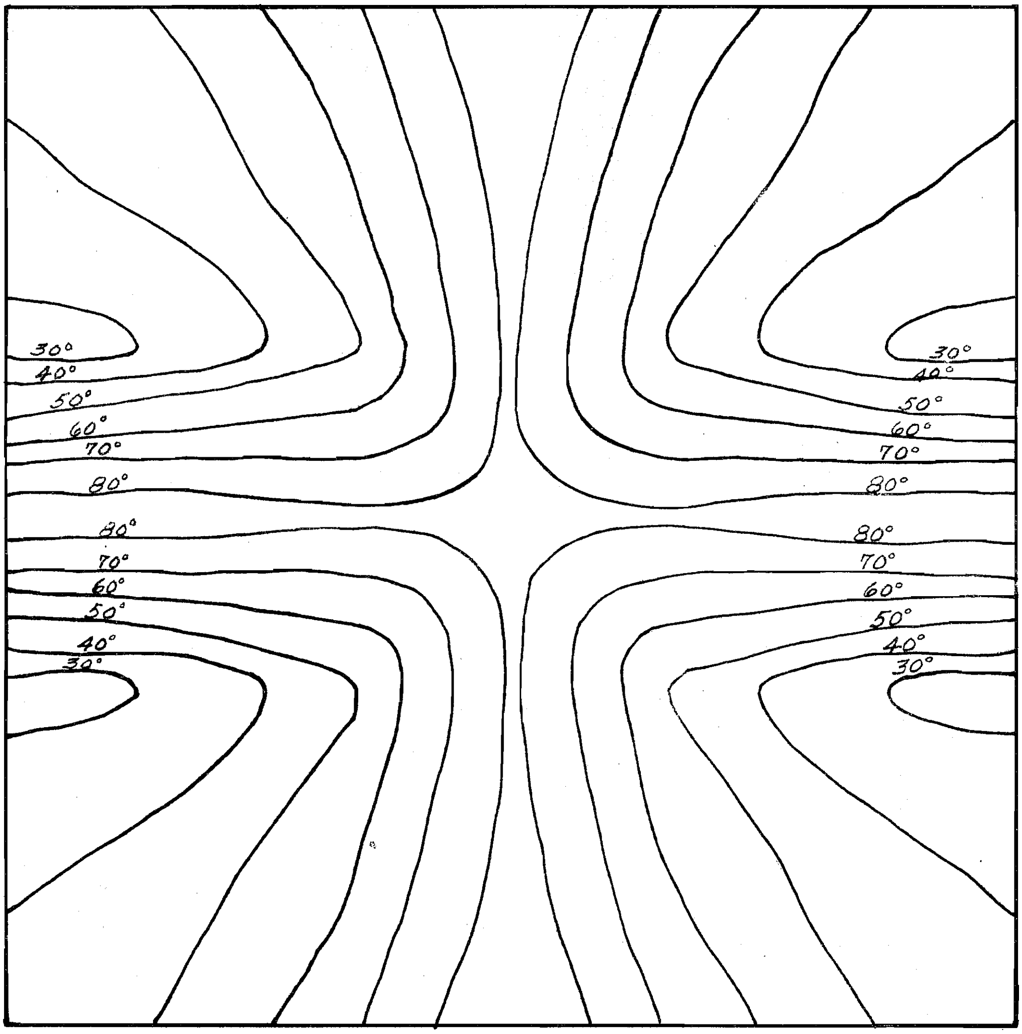

FIGURE 7
SINGLE LOAD AT 11
CURVES OF ISOCLINICS

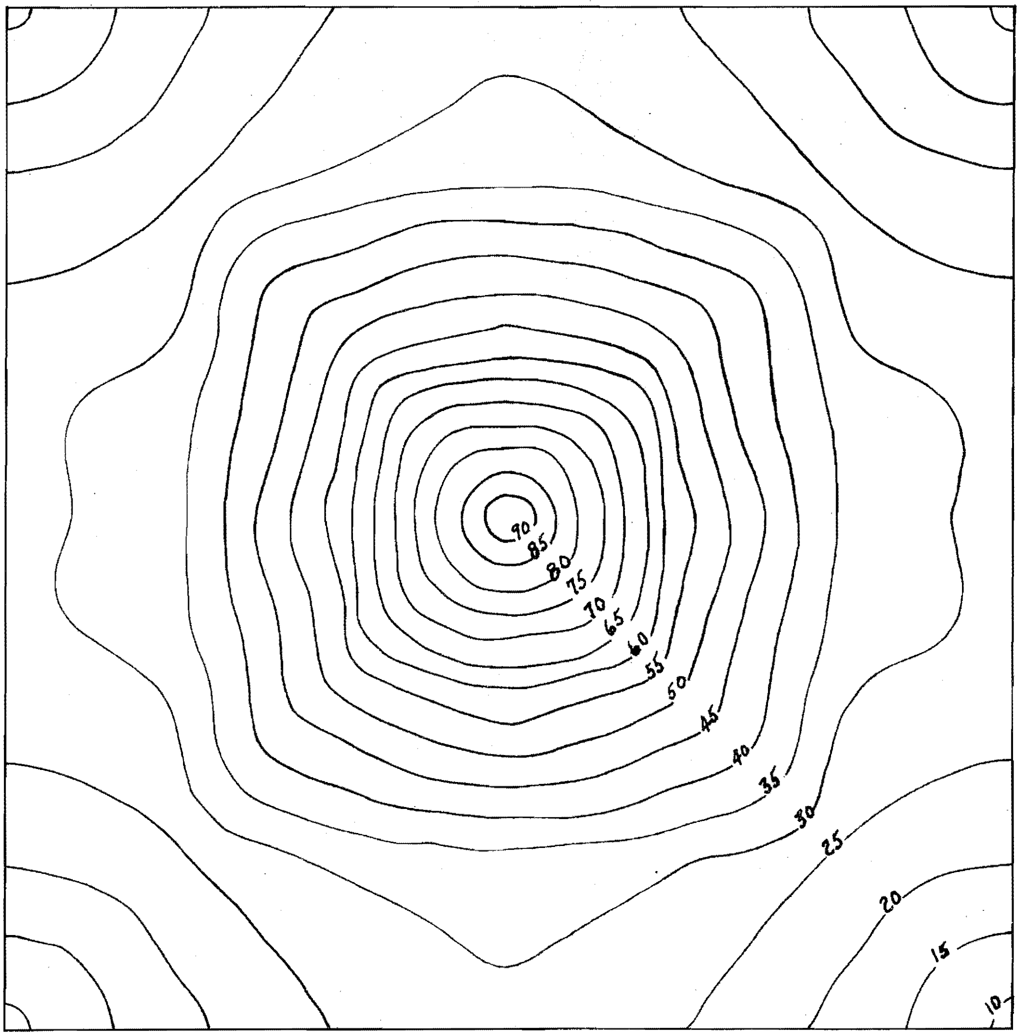

FIGURE 8
UNIFORM LOAD
MAX. PRINCIPAL STRESSES

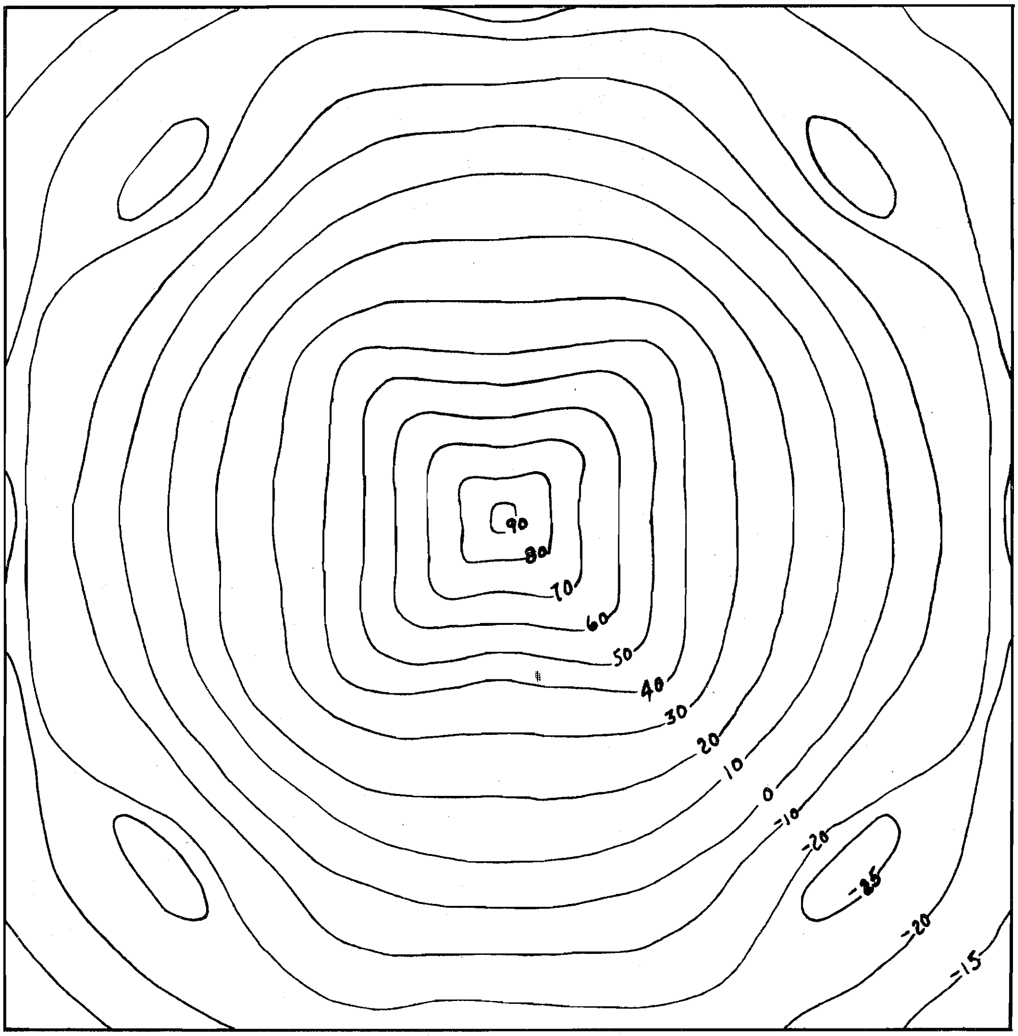

FIGURE 9
UNIFORM LOAD
MIN. PRINCIPAL STRESSES

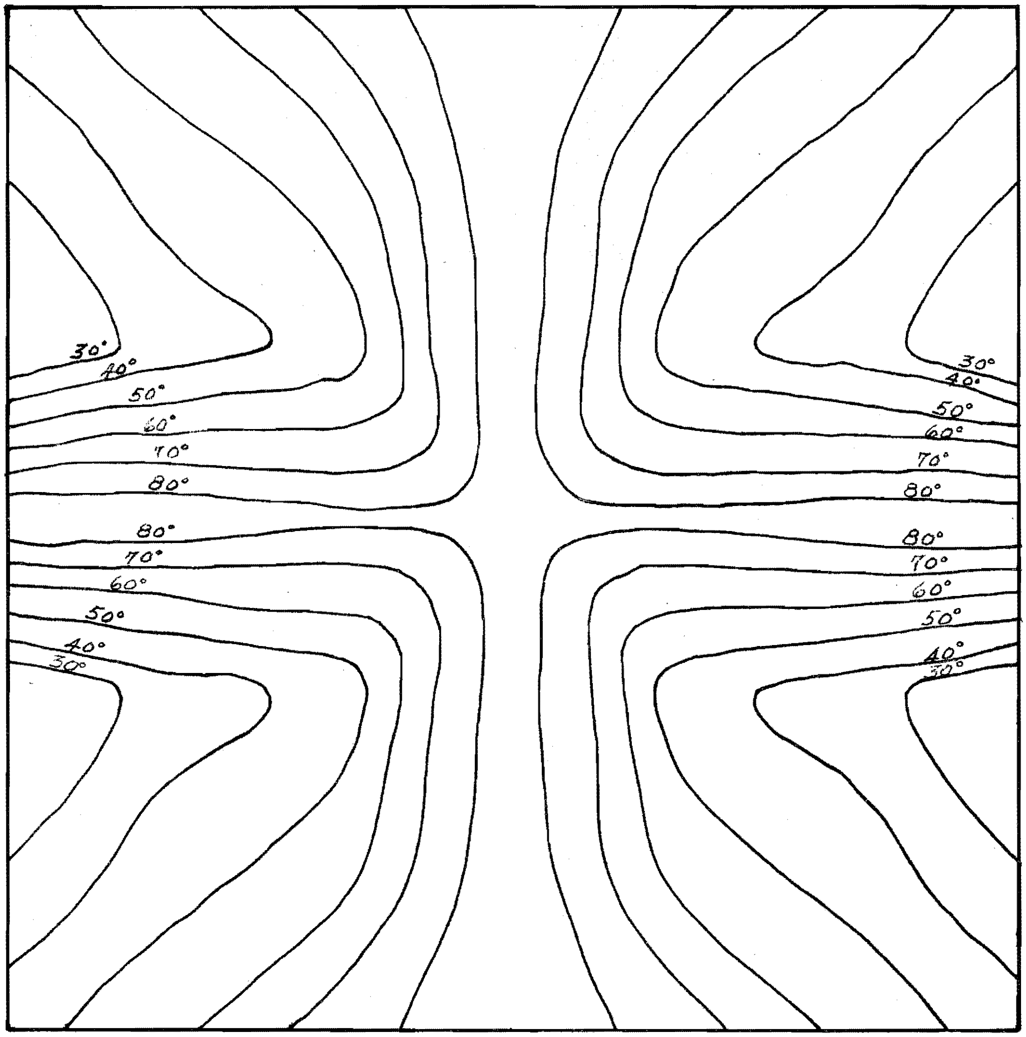

FIGURE 10
UNIFORM LOAD
CURVES OF ISOCLINICS

VII. CONCLUSIONS

Since this thesis is one of the first ones dealing with this subject, no attempt will be made to compare the results with other thesis pertaining to the subject. The conclusions given here are those that can be drawn from the results obtained from this investigation.

Several methods were employed to determine the amount of load carried to the columns by way of the beams, but none proved fruitful.

From the recorded strains a moment diagram was plotted for the beam. It was hoped that from the moment diagram the load on the beam could be deduced. This proved unsuccessful because of too many unknowns.

As was previously mentioned, an attempt was made to approach a pin end condition for the bottom of the columns. Referring to Table 2, it is seen that the stresses at the botton of the column are about 38% as compared to the stresses at the top of the column. The large amount of restrain developed at the base of the column can be attributed to the friction of the column end and socket. This action parallels the action of a normal building column where the designer considers the column end pinned.

This writer hopes that this thesis will be of help to other persons making investigations that deal with the distribution of the stresses in the slab and into the beams and columns, so that designers can arrive at a more economical design.

VIII. ACKNOWLEDGMENTS

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X. VITA

The writer was born at Oak Grove, Virginia (Westmoreland County), June 14, 1924, the second son of Lewis Charles Flemer and Nora Catherine Eaton Flemer. He was married June 10, 1950, to Jean Virginia DeShazo of Fredericksburg, Virginia.

He attended Oak Grove Elementary School and received his high school training and diploma from Christchurch School, Christchurch, Virginia, (Middlesex County) in 1943. At this time, he entered the Armed Services and served until 1946. He enrolled at Virginia Polytechnic Institute in the fall of 1946, majoring in Architectural Engineering. After receiving his Bachelor of Science Degree in 1950, he remained at Virginia Polytechnic Institute and began work for the fulfillment of the requirements for the Master of Science Degree in Architectural Engineering. It was necessary for him to leave school before completing the thesis. Since that time, he has been employed by Slaughter, Saville, & Blackburn, Inc., Consulting Engineers, Richmond, Virginia, 1951-1953; Baskervill & Sons, Architects & Engineers, Richmond, Virginia, 1953-1954. At present, he is employed by Reynolds Metals Company, Engineering Division, Richmond, Virginia.