by 19 feet 1 inch, and was forty days old when tested. Fig. 5 shows the manner of reinforcing.

The deformations in the steel and the concrete at the critical points were obtained by means of extensometers, and from the data thus obtained the various stresses were computed. Mr. Lord claims that the precision attained in his measurements was such that the stresses in the steel were obtained with a maximum error of 1,000 pounds per square inch, and in the concrete with a maximum error of 50 pounds per square inch.

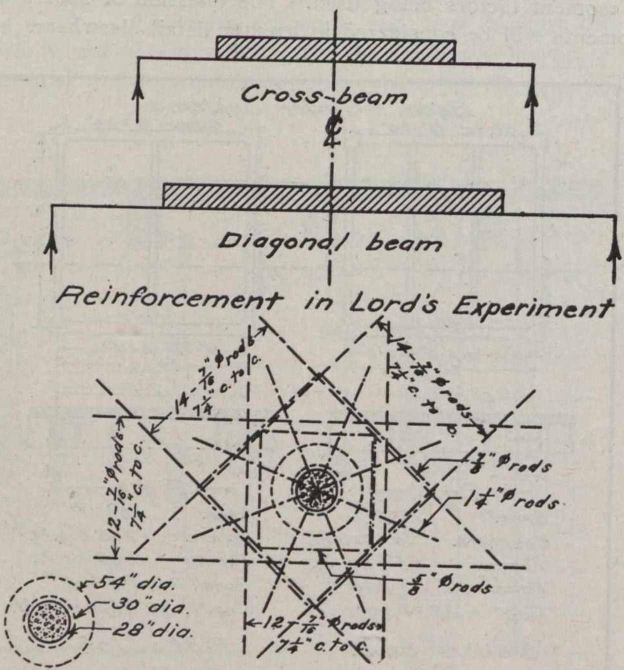

Fig. 5.—Column Used in Lord's Test, Showing Reinforcing.

With regard to deflection it was found that the presence of joints where new concrete is joined to old materially increases the deflection. For eight panels loaded the deflection in the centre panel was 1/1400 of the span, and for one panel only loaded was 1/1200 of the span. Deflections in the central panel were uniformly less than those in the outer panels. The maximum deflection found was at a bulkhead in an outer panel and amounted to 1/1000 of the span.

The stresses in the steel at the centre of the panel were found to be low, running from 2,000 to 4,000 pounds per square inch, and indicating a large amount of arch action reducing the computed stresses. The measurements indicated that the steel in the centre of the panel is most severely stressed for one panel only loaded. The diagonal and cross-band reinforcing were found to be taking practically the same stress, and this stress was approximately equally divided among the individual members of a band. In the steel over the columns the stress reached 16,000 pounds per square inch, so it is clear that this is the critical point in the design of such structures.

For the determination of the stresses in the concrete Young's Modulus was taken as 1,875,000 pounds per square inch. This gave compressive stresses at the edge of the capital of 650 to 675 pounds per square inch. While in this case these values are not excessive, it indicates that care must be taken to provide compressive reinforcement at this point if necessaryy. It is quite common practice to run the diagonal reinforcing straight through on the bottom, thus providing this compressive reinforcement and at the same time avoiding the need of placing four layers of steel in the upper part of the slab near the column which decreases to a considerable extent the effective depth of the slab.

The floor was carefully cleaned and closely examined for indications of cracks. With a load of 265 pounds per square foot faint cracks appeared at a bulkhead. Other such cracks appeared when the maximum load of 350 pounds per square foot was put on. One set ran along the centre of the cross-band, dying out at each end, thus indicating the tension in the upper part of the slab along the side of the panel previously mentioned. Another set appeared around the columns, and 2 inches to 3 inches outside the edge of the capital, furnishing another indication that the maximum moment occurs over the column, the critical section being at or near the edge of the capital.

With regard to the distribution of stress among the several bars of a band of reinforcing, some experiments made on broad, shallow beams by Prof. A. N. Talbot, of the University of Illinois, are of interest. The beams were 36 inches broad, 4 feet 10 inches long, and 3 inches deep, the ratio of depth to width being one-twelfth. They were loaded across their full width and freely supported at the ends for various proportions of their width. The beams were made and tested induplicate at an age of about sixty days.

The following table, from the National Association of Cement Users proceedings, 1911, shows a few of his results:

		Total load carried, in pounds.		
Beam No.	Supported.	Beam No. 1.	Beam No. 2.	Average.
711.1-2	Full width	15,550	15,800	15,675
713.1-2	Half width	15,000	17,000	16,000
715.1-2	Fifth width	14,900	12,250	13,500
717.1-2	Fifth width	14,550	16,000	15,300

All the beams had ten ⅜ inch round rods 3 inches from the face, and beams 717.1-2 had also ¼ inch round rods on 4-inch centres crosswise of the beam.

From these and similar experiments he concludes that for a beam supported over one-half its width there is no appreciable falling off in the load carried. For a beam supported only one-fifth of the width the decrease in strength is slight. Other tests made by Prof. Talbot on footings, as well as Mr. Lord's extensometer tests, bear out this result.

Prof. Talbot states his conclusion thus: "The width of the resisting section, as governing the stress in the steel, is composed of the width of the pier, plus the depth to the steel on each side of this, plus one-half the remaining width of the footing." In ordinary flat slab designs this would include all the steel in both diagonal and cross-bands, and we may therefore conclude that it is all nearly equally stressed regardless of whether it passes directly over the column head or not. The above discussion refers, of course, to stresses produced by moments only, as the resisting section to shear stresses will obviously only be equal in width to the column.

Mr. C. A. P. Turner's method of design* is based on experimental data obtained from tests made on full-size single panels. Details of particular tests are not to hand, but the general method may be described. Tests were first made on the steel reinforcing and its yield point determined. The slab was then built, using this grade of steel, and after being allowed to season a reasonable time was loaded until the steel reached this yield point, which Mr. Turner claims to be able to recognize from the behavior of the slab. Then, by simple proportion, an exact working load for a given tension in the metal may be determined.

*"Concrete Steel Construction," by C. A. P. Turner, Chap. 3.