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Summary of Full-Scale Laboratory Tests of Concrete Slabs Reinforced With Cold-Formed Steel Decking

Résumé d'essais en laboratoire sur des dalles en béton armé avec des tôles façonnées à froid

Eine Zusammenfassung von Laborversuchen an mit kaltverformten Stahlplatten armierten Betondecken

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INTRODUCTION

Reference is made by C. F. McDevitt and I. M. Viest[1] to the development and use of cold-formed steel decking as a form and as composite reinforcement for reinforced concrete floor slabs. The increased use of cold-formed steel-deck-reinforced concrete floor slabs stems primarily from the economic advantages including elimination of the need to install and remove formwork, ease in handling and placing the deck sheets, convenience of a working platform prior to casting, and providing the steel reinforcement for the floor slab through various shear connecting devices to give positive interaction between the steel and the concrete.

Particular reference is made by McDevitt and Viest[1] to an extensive theoretical and experimental investigation of steel decking as reinforcement for concrete slabs at Iowa State University. This investigation was initiated in 1966 under the sponsorship of the American Iron and Steel Institute to explore various aspects of the cold-formed steel-deck-reinforced floor slabs. To date 273 tests using decks from five different manufacturers have been conducted including the following types (all subjected to static loading unless otherwise indicated):

1. single span, simply supported one-way slab elements;

2. pushout specimens;

3. single span, simply supported one-way slab elements subjected to repeated loading;

4. single span, simply supported slab elements with the deck corrugations transverse to the span;

5. multiple span one-way slab elements;

6. single span slab elements containing variable welded wire fabric; and

7. full-scale two-way slabs consisting of a single span on four simply supported edges.

Extensive laboratory tests have shown that most of the one-way steel-deck-reinforced concrete slab systems exhibit a shear-bond type of failure. This is characterized by the development of a diagonal crack just before failure, followed by an observable end-slip between steel decking and concrete. An experimental linear regression relationship which permits calculation of the ultimate shear capacity of a one-way slab element was developed by Dr. R. M. Schuster[2] at Iowa State University in 1969.

This paper will focus primarily on the investigation of four full-scale slab tests, item No. 7 on the above list. The objective of this research was to develop information that would lead to improved criteria for the design of such systems. Since testing of the fourth slab of the series was only recently completed, it was not possible to reduce all the data in time for presentation. It was possible, however, to include a variety of information which describes the behavioral characteristics of each slab as it was loaded to failure in the laboratory. These characteristics include crack patterns, vertical deflections, reaction distributions, and end-slip.

DESCRIPTION OF FULL-SCALE SLAB TESTS

Figure 1 shows the general layout of the four full-scale slab tests. All four slabs had nominal out-to-out dimensions of 12 X 16 ft and were loaded with four concentrated loads located as shown in Fig. 1. The four concentrated loads were chosen to approximate the effect of a fork-lift truck, and to ascertain the load distributions encountered with concentrated loads on steel-deck-reinforced floor slabs. All slabs had the steel decking placed with its corrugations parallel to the 12 ft length as indicated in Fig. 1.

The first three slabs were constructed with steel decking, which had rolled embossments to provide a positive shear transfer between decking and concrete, whereas the fourth slab had steel decking with transverse wires spaced 3 in. apart spot-welded to the tops of the corrugations for shear transfer. Figure 2 shows a typical cross section for each deck type.

Table 1 provides data on various properties of the material for each of the four test slabs. The out-to-out slab thickness varied somewhat in each slab due to deflection of the deck under the dead weight of the wet concrete. The values of average thickness were determined from measurements taken after each slab was sawed into sections following completion of tests. The concrete strengths were obtained from tests on standard 6 X 12 in. cylinders, and the modulus of rupture values refer to 6 X 6 X 36 in. plain concrete beams.

Supplementary reinforcement in the form of welded wire fabric (WWF) was placed directly on top of the steel decking in the first and second slabs. Slab 1 contained 6 X 6 X 6/6 WWF and slab 2 contained 6 X 12 X 0/4 WWF; however, slab 3 contained no welded wire fabric. The fabric in slab 2 was placed so that the zero gage wire (on 6-in. centers) was transverse to the corrugations of the decking. Table 1 shows the steel areas and yield strengths of the steel decking and the welded wire fabric.

The four slabs were instrumented with strain gages on both the concrete and steel decking at about 17 locations with rosette gage arrangements. Deflection dials to measure vertical deflections were located at about 28 locations. Six roller transducers were placed along the south edge and 11 ball bearing transducers were placed along the west edge of the slab. These transducers were instrumented to measure only vertical reactions along the edges of the slabs. Slab 1 contained corner restraints which were instrumented to determine the vertical uplift reactions of the corners. The remaining slabs did not

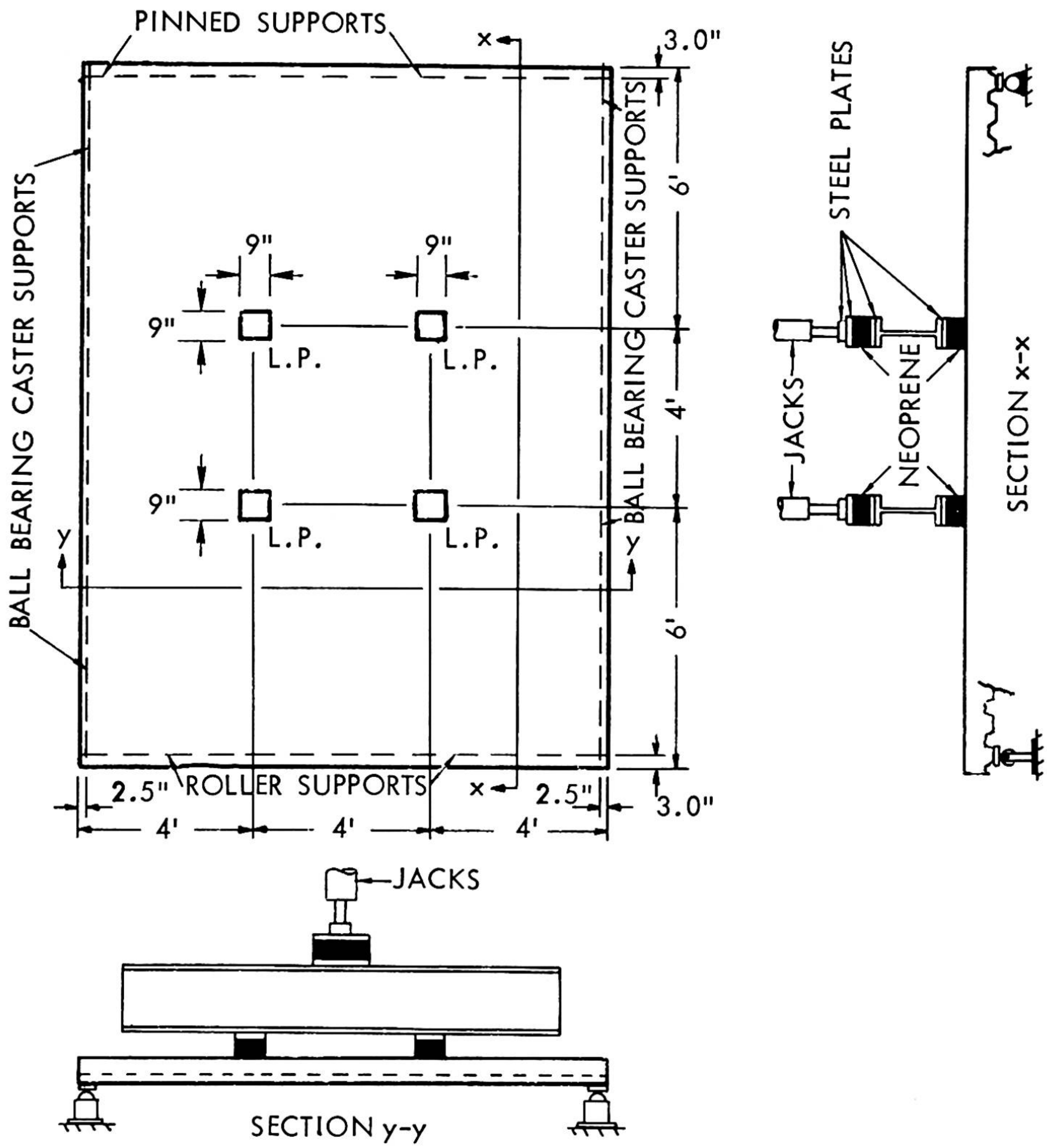

Fig. 1. General layout of full-scale slab tests indicating positions of the load points and positions of the support reactions.

contain the corner restraints and the corners were free to lift upward. The amount of the corner uplift was measured for those slabs containing no corner restraints. Figure 3 shows the transducer reaction assemblies, including the corner restraint instrumentation used in slab 1.

The overall support assemblage for the slabs is shown in Fig. 4. Also seen in Fig. 4 are the deflection dials which were supported on an independent grillage. As can be seen by the location of the deflection dials, basically only the southwest quadrant of the slab was heavily instrumented.

TEST PROCEDURE AND RESULTS OF SLAB TESTS

Loading for all four slabs was applied through the use of hydraulic jacks as shown in Fig. 5 which depicts an overall view of slab 1 immediately after

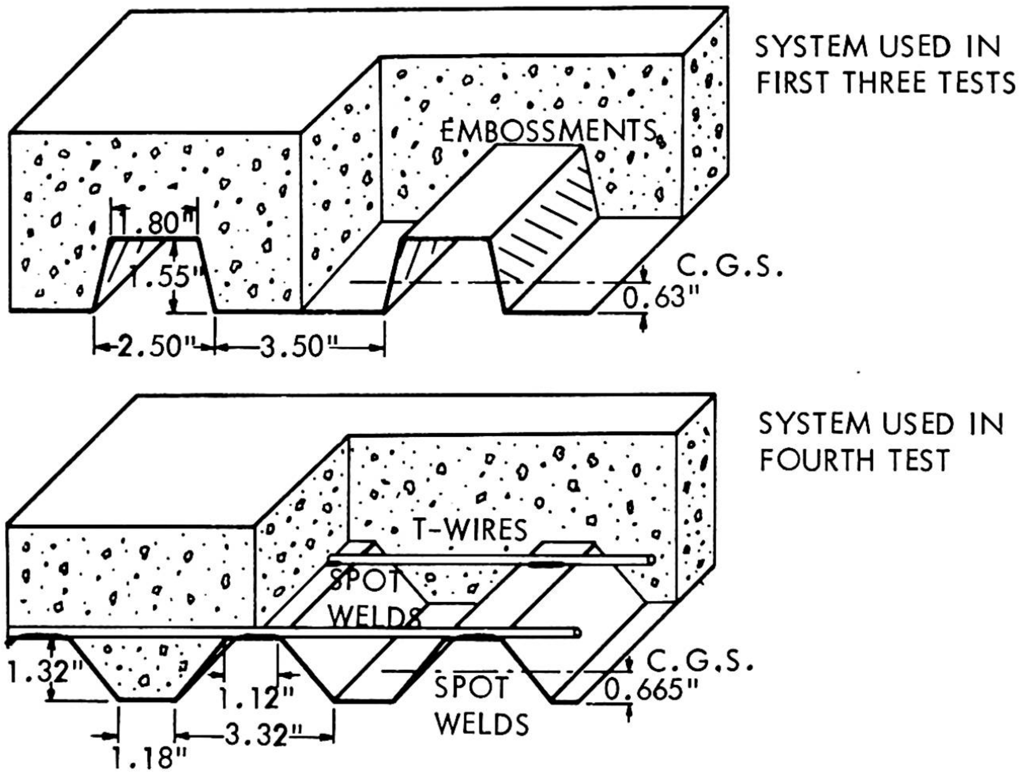

Fig. 2. Views showing types of decking used.

testing. Loading for slab 1 was applied continuously from zero to ultimate except for the time needed to take instrumentation readings.

The other three slabs were loaded continuously from zero to a designated cycling load. At this stage unloading and reloading to the cycling load occurred 10 times. After cycling, a final loading was made from zero to ultimate failure of the slab. Table 1 contains the ultimate load and cycling load for each of the four slabs. A complete view of the total crack pattern which occurred in slab 2 is shown in Fig. 6.

Figure 7 provides diagrams of the crack patterns that occurred on the top surface of the four slab tests. Indicated is the load at which essentially no top surface cracking had occurred. In each case note that the top surface cracking occurred at a fairly high percentage of the ultimate load. Except for slab 1, in which the first cracking was that across each of the corners, the first cracking of the slabs occurred as diagonal cracks along the edges of the slabs.

Figure 8 shows the edge cracking that occurred for the east edges of all the slabs. This edge cracking was typical of that which occurred on both edges of the slabs. The crack numbers in Fig. 8 indicate the order of crack development. After these edge cracks had formed, subsequent loading results in a widening of the primary cracks in each case as shown by the heavier crack lines for each edge. Ultimate failure of all four slabs was precipitated by a horizontal slippage between the steel decking and the concrete in the region between the primary edge cracks. This slippage was first detected by dial displacement gages during or just after completion of the cyclic loading for the last three slabs, but did not become significant until failure was imminent.

Table 1. Properties of component materials and test data.

	Slab 1	Slab 2	Slab 3	Slab 4
Concrete Properties				
Average cylinder strength, psi	4355	3538	3951	3816
Modulus of rupture, psi	485	470	—	521
Age of cylinder beams and slab, days	15	17	17	16
Steel Decking				
Cross-sectional area in.^{2}/ft	0.625	0.625	0.625	0.376
Yield strength at 0.2% offset, ksi	45.1	45.1	45.1	101.6
Supplementary Reinforcing (Welded wire fabric, or T-wires)				
Area of WWF parallel to deck corrugations in.^{2}/ft	0.0525	0.034	None	None
Area of WWF or T-wires transverse to deck corrugation, in.^{2}/ft	0.0575	0.144	None	0.0150
Yield strength, (0.005 strain), ksi	79.0	82.6 (#0 ga.) 84.6 (#4 ga.)	None	92.1
Composite Test Slab				
Average out-to-out thickness, in.	4.98	4.75	4.75	4.87
Corner support condition	Restrained	Free	Free	Free
Cycling load, kips/load point	None	9.4	6.4	9.4
Ultimate load, kips/load point	13.5	15.5	8.8	14.4
Equiv. ultimate uniform load, psf	300	345	196	321

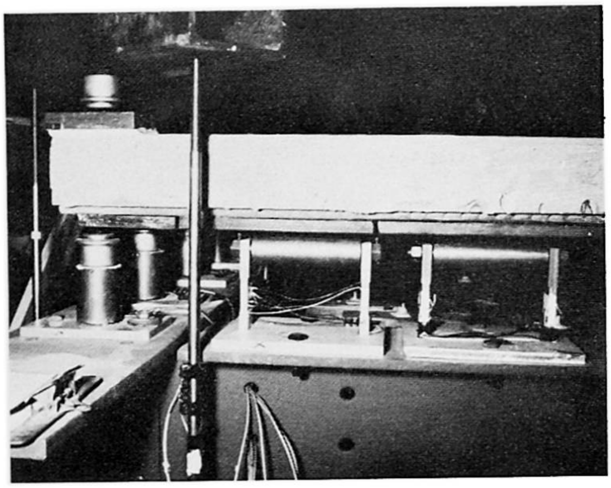

Fig. 3. Instrumentation to measure corner uplift force and vertical downward forces along the edges of slab 1.

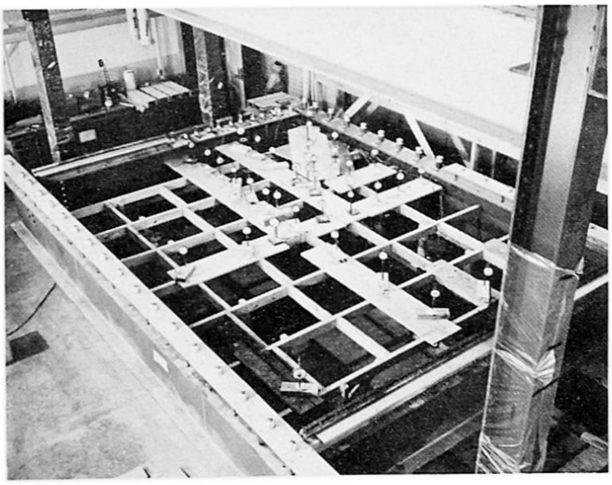

Fig. 4. Support and deflection dial arrangement for full-scale slab tests.

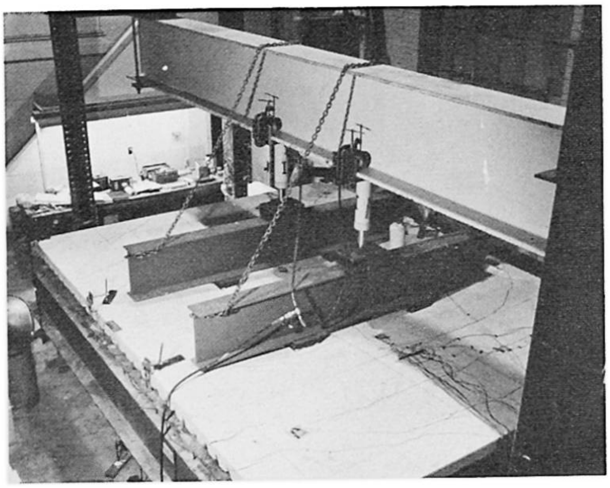

Fig. 5. View of full-scale two-way slab test.

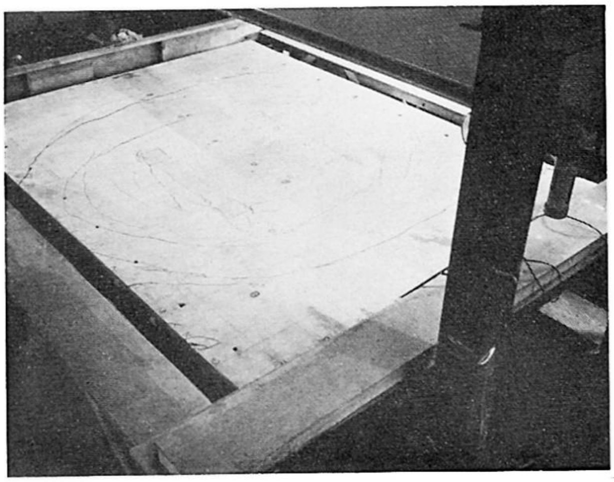

Fig. 6. Top surface of slab 2 indicating fixed crack pattern at termination of testing.

The stiffness characteristics of the four slabs can be ascertained by examining Fig. 9, which shows the load-deflection relationships for the first application of load. In the case of slab 1, the load was increased in continuously applied increments up to ultimate load. The second, third, and fourth slabs were loaded to 9.4, 6.4, and 9.4 kips per load point, respectively, during the first cycle. As previously explained, the loading on these latter three slabs was reduced to zero, and re-applied nine more times. The residual deflections following the first cycle of load, is shown to be 0.182 in., 0.101 in., and 0.265 in. for slab 2, 3, and 4, respectively. It is of interest that slab 1, with the corner tie-downs, exhibits significantly stiffer characteristics than any of the other slabs. Slab 2, which was heavily reinforced with 6 × 12 × 0/4 welded wire fabric, was somewhat stiffer than slabs 3 and 4. Slabs 3 and 4 were of about equal stiffness.

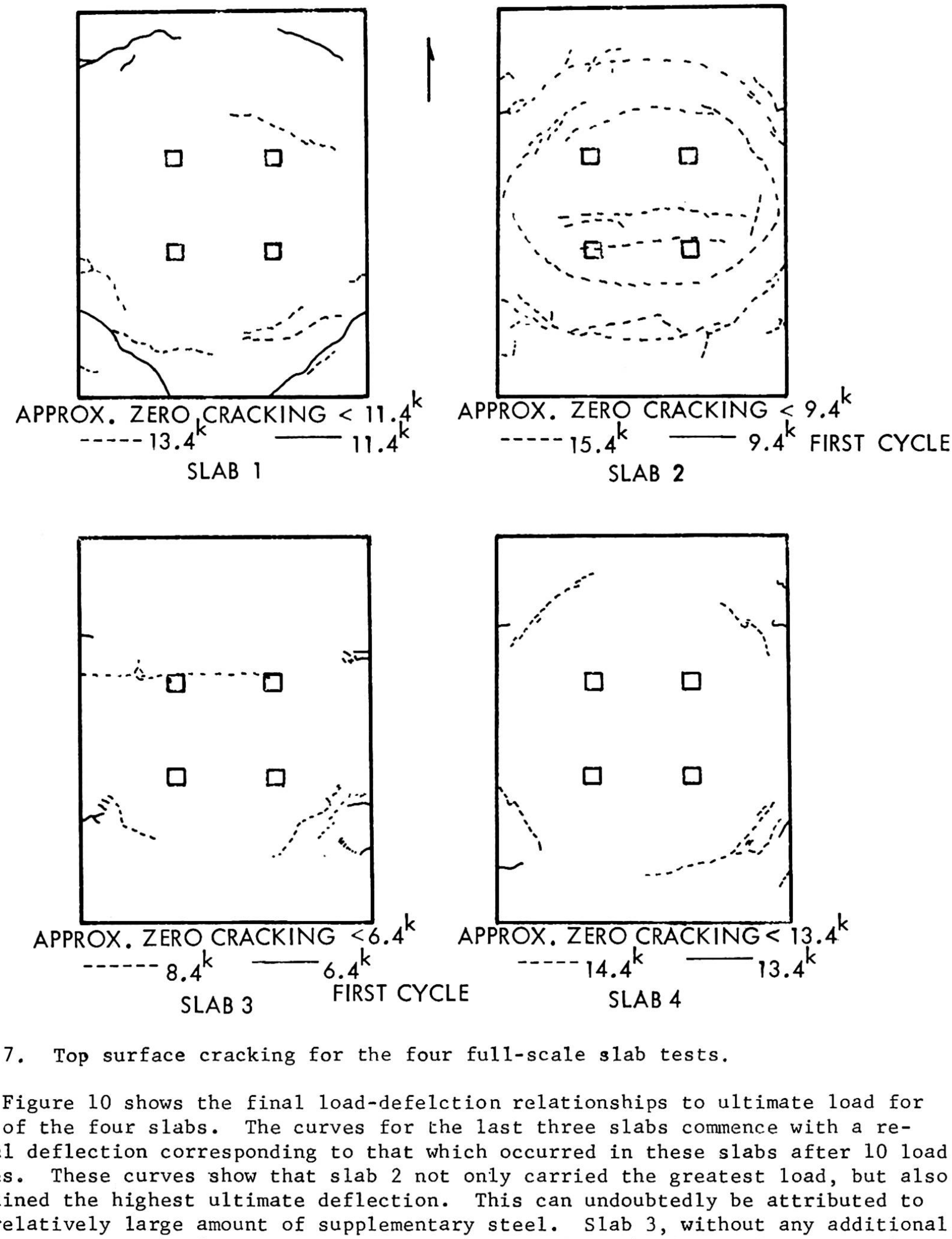

Fig. 7. Top surface cracking for the four full-scale slab tests.

Figure 10 shows the final load-defelction relationships to ultimate load for each of the four slabs. The curves for the last three slabs commence with a residual deflection corresponding to that which occurred in these slabs after 10 load cycles. These curves show that slab 2 not only carried the greatest load, but also sustained the highest ultimate deflection. This can undoubtedly be attributed to the relatively large amount of supplementary steel. Slab 3, without any additional steel, indicated the least ultimate strength as well as the least ultimate deflection. Slab 1 with the corner tie-downs and rather nominal 6 x 6 x 6/6 welded wire fabric closely approximated the behavior of slab 4, with the closely spaced welded transverse wires, neglecting the permanent deformation due to cycling.

Typical results of the measured vertical reactions along the west side may be summarized by looking at Fig. 11 for slab 1. The amount of load transmitted

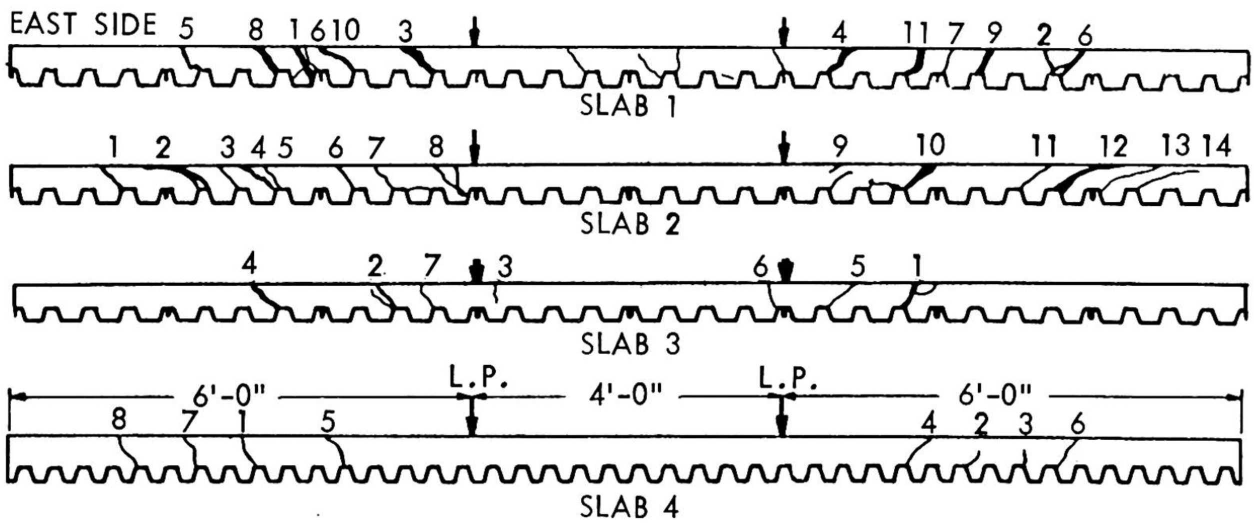

Fig. 8. Crack pattern development along the east edges of all slabs.

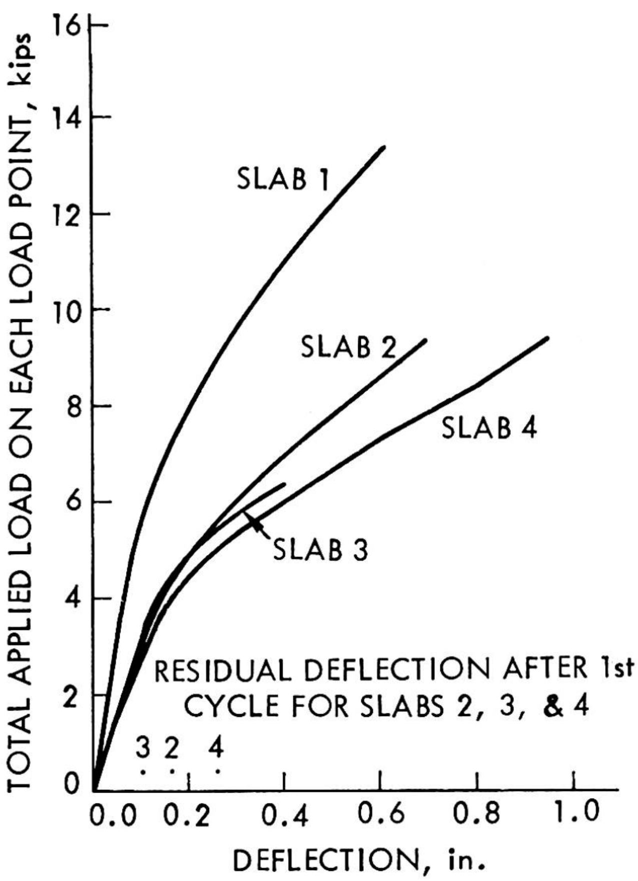

Fig. 9. Load deflection characteristics for the initial loading stages.

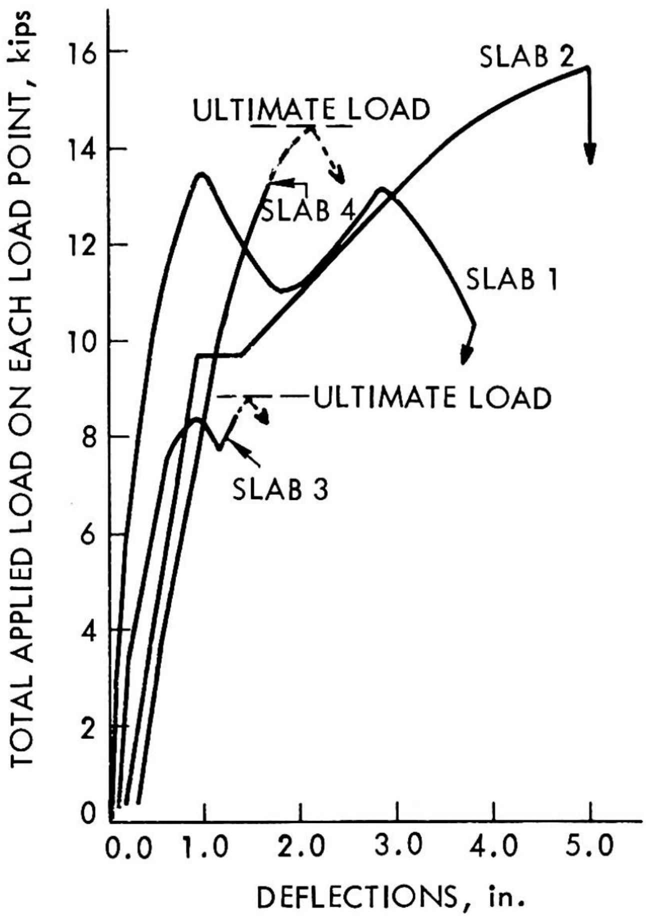

Fig. 10. Load deflection characteristics after cycling, for the final loading stages.

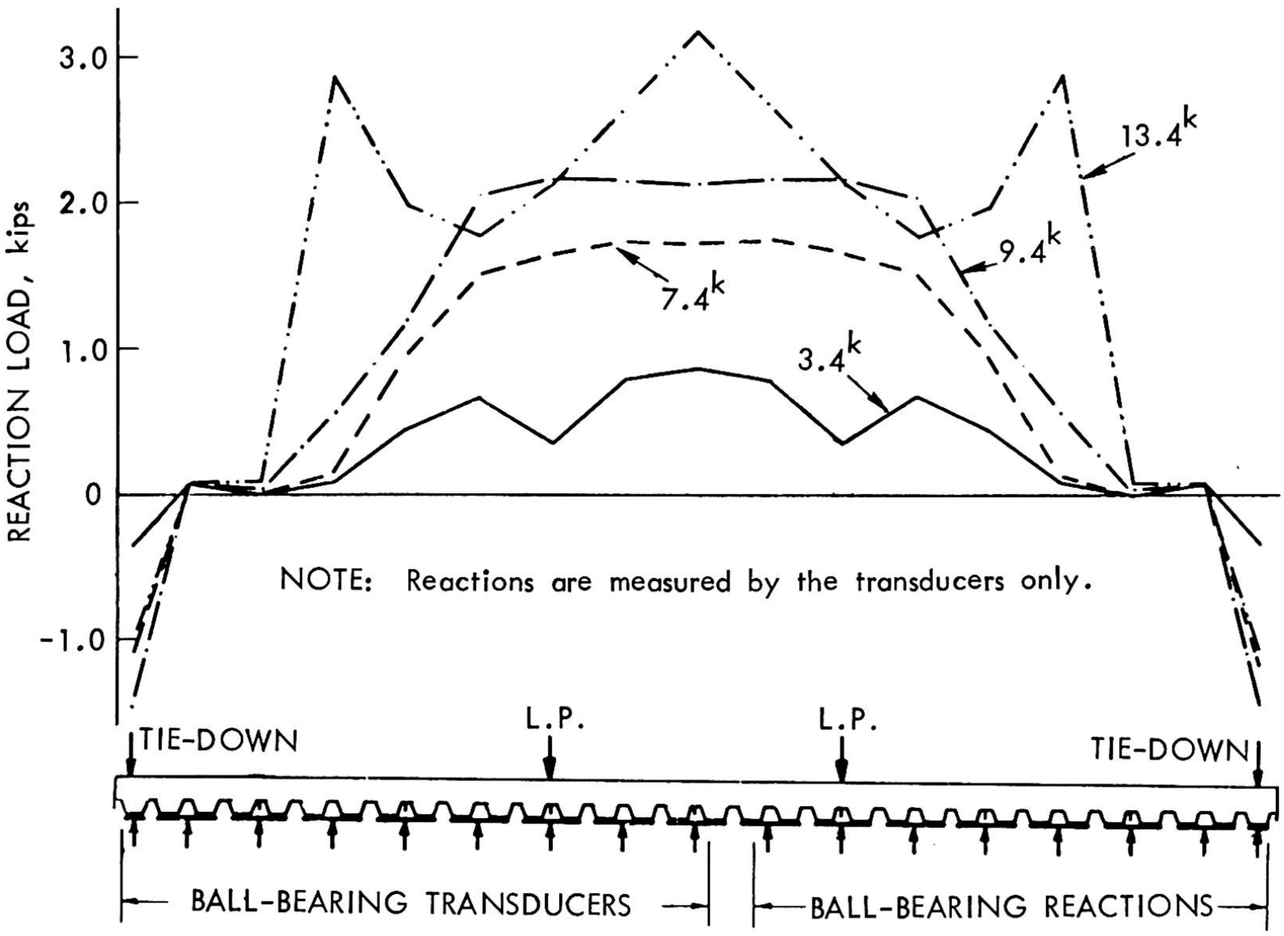

Fig. 11. Distribution of reactive forces due to the applied load along the west edge of slab 1.

in the weak direction to the south support is illustrated by the reaction distributions in Fig. 12. An indication of the percentage of load transmitted to each side as loading progressed is shown in Fig. 13. Note that the greater distribution in the direction transverse to the corrugations occurs at lower loads while the slab is essentially still elastic. The amount of distribution to south support for the other three slabs without corner tie-downs was less than that indicated by Figs. 12 and 13. In fact, for these slabs 90% or more load was transmitted to the east and west supports for all loads over 50% of ultimate.

Limitations of space prohibit presentation of reaction distributions for the other slabs. However, it was noted during testing and supported by the reactive measurements and crack patterns that for the three slabs without corner tie-downs, there appeared to be an approximate effective width of 8 ft over the central regions of the east and west support edges.

The analysis of data for each of the four slabs is still underway. Coupled with the analysis is an effort to develop a theory of failure which will provide a basis for an ultimate strength approach to design. A first attempt was to try the yield line method for predicting the ultimate strength. This method, unfortunately, gave results which were at least 22% higher than the experimental values of ultimate strength for all four slabs. This seems logical from the standpoint that the four slabs did not fail by a flexure

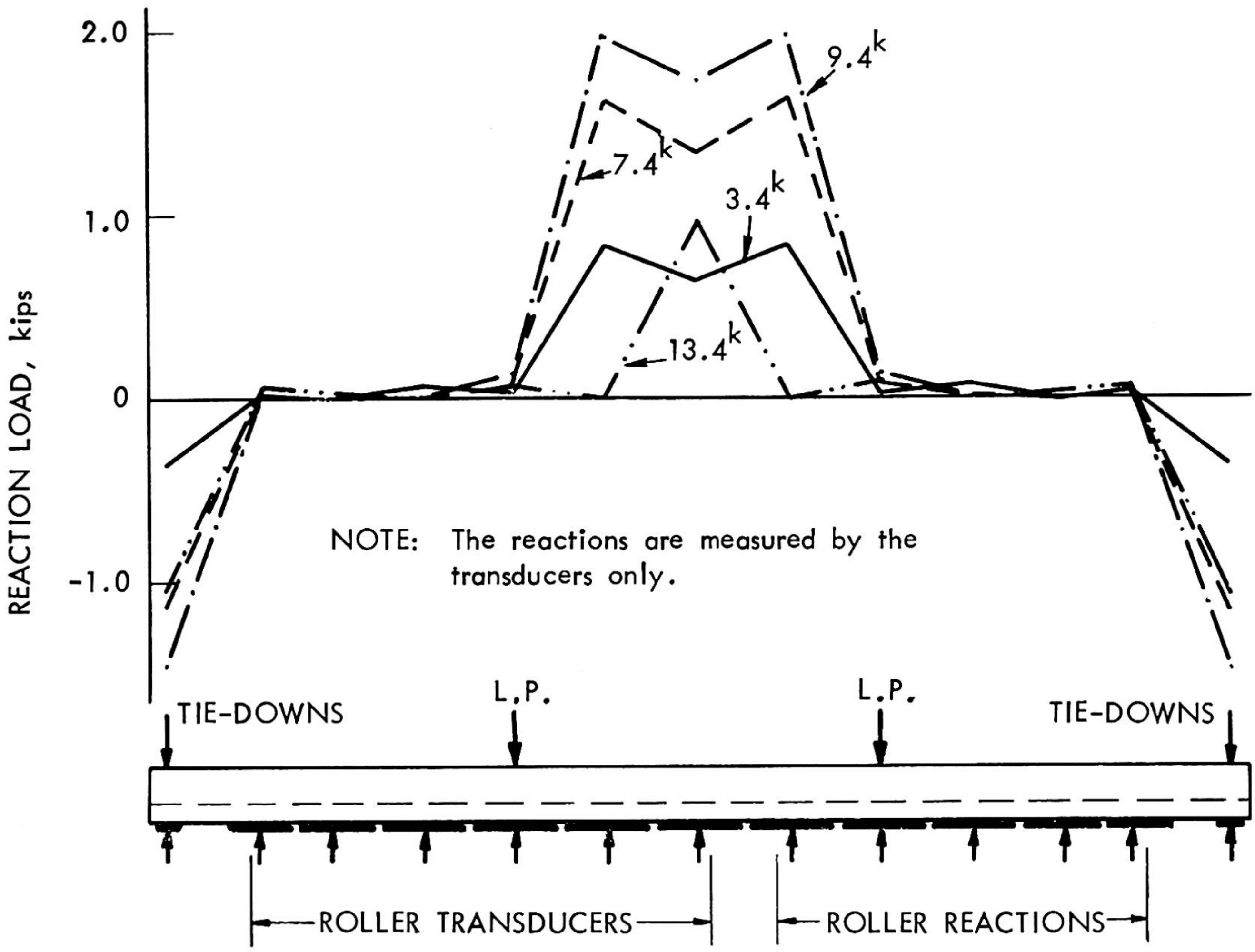

Fig. 12. Distribution of reactive forces due to the applied load along the south edge of slab 1.

action, as the yield-line method assumes, but rather by a shear-bond action involving a horizontal slippage between steel decking and the concrete.

CONCLUSIONS

1. Four full-scale slabs, of approximately identical dimensions, were loaded to failure in the laboratory. All four slabs failed by a shear-bond action which was precipitated by a horizontal slippage between steel decking and concrete.

2. Slab 2 with the largest amount of supplementary steel, in the form of welded wire fabric, sustained the largest test load. This slab also sustained the largest ultimate deflection of the four slabs.

3. Slab 1 with all four corners tied down, and with only a nominal amount of supplementary reinforcing, exhibited a significantly stiffer behavior than any of the other slabs.

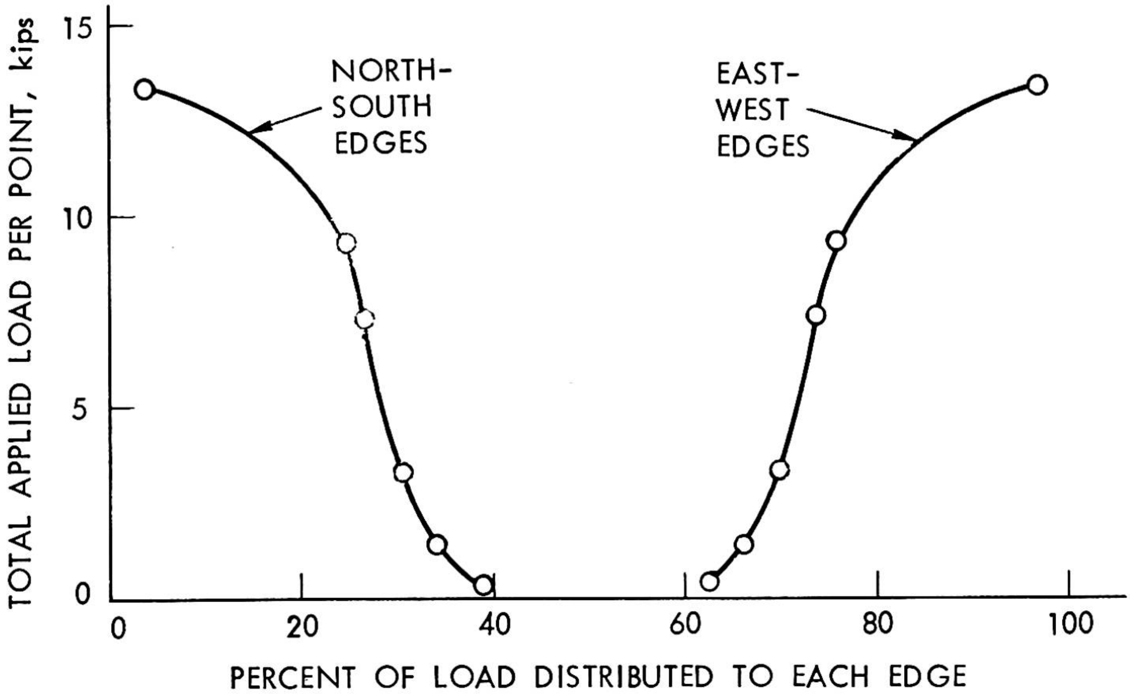

Fig. 13. Illustration of the percentage of applied load transmitted to each reaction support as loading increases for slab 1.

4. Slab 3 without any supplementary reinforcing could carry an ultimate load which was only about 57% that of slab 2. The ultimate deflection of slab was much less than that of slab 2.

5. In general, the top surface cracking behavior of all slabs was excellent. Excessive cracking did not occur until ultimate load was imminent, and even then the cracks were quite small.

REFERENCES

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SUMMARY

Four composite slabs were constructed with nominal out-to-out dimensions of 12 ft X 16 ft X 4-7/8 in. and incorporating corrugated, cold-formed steel decking as reinforcement. Two slabs had supplementary reinforcing in the form of welded wire fabric.

Laboratory tests to ultimate load on the simply supported slabs were conducted using four concentrated load points. The mode of failure was a shear-bond action precipitated by slippage between steel decking and concrete.

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