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Investigation of Cold-formed Steel-deck-reinforced Concrete Floor Slabs

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INVESTIGATION OF COLD-FORMED STEEL-DECK-REINFORCED CONCRETE FLOOR SLABS

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Introduction

The cost of a separate forming operation for installing and removing forms (either wood or steel) has been a major part of the total cost of cast-in-place concrete floor construction. Elimination of this forming operation can be accomplished by using a corrugated cold-formed steel decking which remains permanently in place as an integral part of the floor system slab. This steel decking serves a second function as positive reinforcement during the service of the slab. Thus, the only additional steel necessary in the slab is that required to take care of temperature and shrinkage and, in the case of continuous spans, to resist negative bending. Secondary advantages also are inherent in this type of construction since it provides a ceiling surface, can be easily handled and placed, and contains pre-engineered ducting for electrification, communication, and air distribution.

The integral action between the cold-formed steel decking and the concrete slab is provided essentially by various shear transferring devices. These devices consist of rolled embossments (on the top flanges and/or webs of the corrugations), transverse wires (T-wires) spot welded to the top of the corrugations, or holes in the corrugations to allow concrete to fill the corrugations. In some instances, vertical interlocking between the steel and the concrete is provided by the geometry of the decking itself.

In 1966 research was initiated at Iowa State University under the sponsorship of the American Iron and Steel Institute to explore various aspects of the cold-formed steel-deck-reinforced floor slabs. The investigation began with a review of current construction projects and design procedures for floor systems reinforced with steel decking[1]. The major emphasis of the research has been on experimental laboratory testing to failure of various steel-deck-reinforced systems. This paper will present an overview of the experimental and analytical results of the current research investigation.

Types of Tests Conducted

A total of 256 various specimens have been tested to date. Most of these have consisted of static tests on one-way slab elements, mounted on simple supports, and subjected to a symmetrical pattern of concentrated loading. Tests on six other categories of specimens have also been conducted, as shown in Table 1.

Table 1. Summary of Types of Specimens Tested. (Note: All specimens subjected to static loading unless otherwise indicated.)

Number Tested	Type of Specimen
171	Slab elements — single span
56	Pushout specimens
14	Slab elements — single span — repeated loading
2	Slab elements with transverse corrugations — single span
5	Slab elements — multiple spans
6	Slab elements — single span — variable welded wire fabric
2	Full-scale two-way slabs — single span, simply supported on four edges
256	Total specimens tested

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A typical test setup for a slab element with a single span is shown in Fig. 1. The more significant variables include shear span, depth of beam, overall span length, gage thickness of steel decking, and concrete strength.

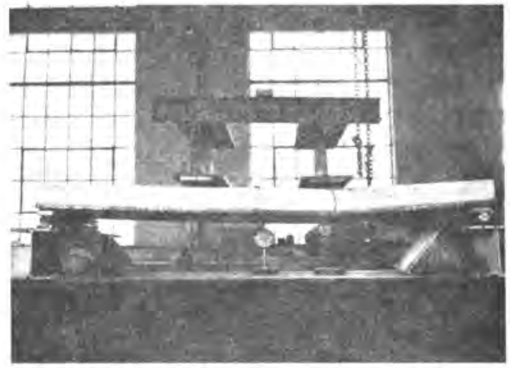


Fig. 1. Typical beam test arrangement.

Examples of some pushout specimens are shown in Fig. 2. The pushout tests were used to ascertain the relationship between bond capacity and embedment lengths for various types and gages of steel decking.

To determine fatigue characteristics, 14 simply supported beam elements were subjected to repeated loads in a fatigue machine. A photograph of a failed specimen is shown in Fig. 3.

Two simply supported slab elements were tested with the corrugations perpendicular to the direction of the span as shown in Fig. 4.

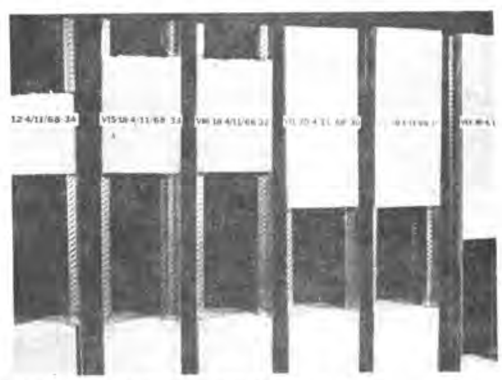


Fig. 2. Vertical pushout test specimens.

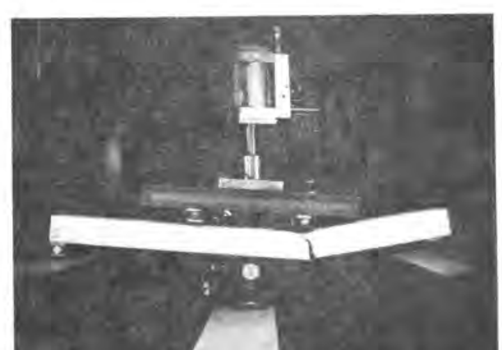


Fig. 3. Fatigue test specimen.

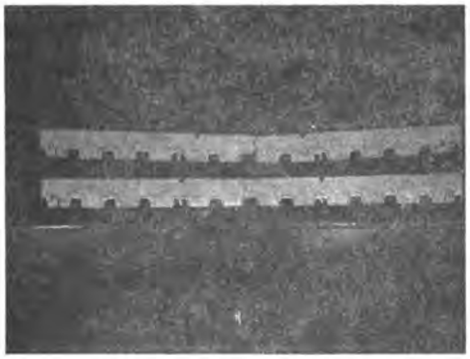


Fig. 4. Beams transverse to corrugations of decking.

The purpose of these tests was to ascertain what strength, if any, is achieved from the decking acting as reinforcement transverse to the direction of the corrugations.

A preliminary investigation of the use of steel decking as reinforcement for continuous one-way slab elements was conducted with five specimens. Four of these had three spans and one had two spans. A view of a typical test is shown in Fig. 5.

As a check on the influence of welded wire fabric, six simply supported beams were tested. In particular, the effect of welded wire fabric on crack width and overall integrity of the system was studied.

Now, research is being conducted on full-scale floor slabs reinforced with cold-formed steel decking. Figure 6 shows one of the slab tests viewed at the termination of testing. To date two such slabs have been tested, both of which rested on four simple supports and were subjected to four concentrated loads simulating fork-lift truck loading on a floor slab.

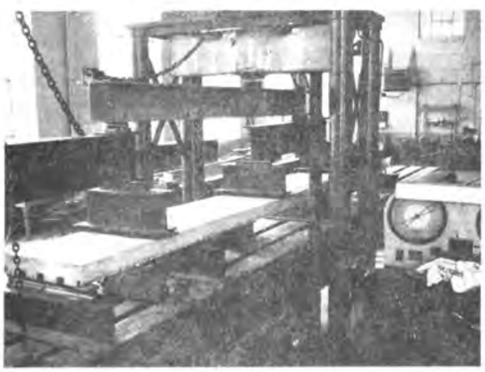


Fig. 5. Continuous beam test.

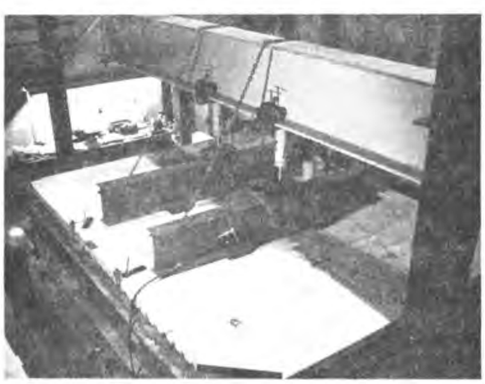


Fig. 6. View of full-scale two-way slab test.

One-Way Slab Elements

The main thrust of the research has been to develop an ultimate strength approach to the design of steel-deck-reinforced concrete floor slabs. As already mentioned, much of the experimental work up to this point has involved a study of the behavioral characteristics of one-way slab elements. Recently, however, the emphasis has been switched to full-scale slabs. The remainder of this paper will be devoted to ultimate strength considerations with respect to one-way and full-scale two-way slab systems.

Many steel-deck-reinforced concrete floor slabs are principally one-way systems in accordance with the direction of the corrugations.

There are basically two types of failures for these systems:

1. Bending or flexure failure
   a. Under-reinforced or tensile
   b. Over-reinforced or compressive
2. Shear-bond failure

Bending failures are quite similar to those found in conventionally reinforced concrete. The under-reinforced mode of failure, under high loads, is characterized by an excess of tensile strain in the steel accompanied by one or more large cracks in the concrete. Final collapse may be due to a tensile failure of the steel as indicated in Fig. 7. The magnitude of the ultimate load may be calculated with reasonable accuracy in accordance with the ACI Building Code[2]. In the over-reinforced case the concrete would crush on the top fiber without accompanying excessive strains in the steel. A view showing the top surface of a failed over-reinforced slab element is shown in Fig. 8.

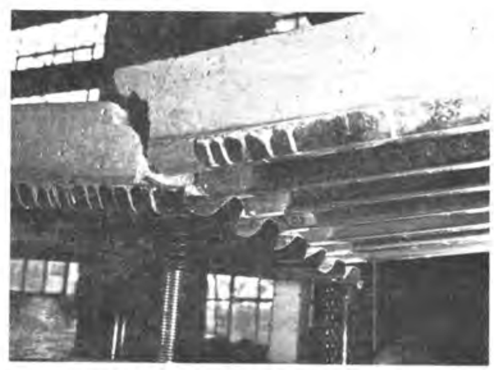


Fig. 7. Tensile steel failure of beam element.

Fig. 8. Crushing concrete failure of beam element.

The primary mode of failure for most steel-deck-reinforced floor elements is that of a so-called shear-bond failure. Typical shear-bond failed beams are shown in Fig. 9. This failure is characterized by a typical shear crack together with a horizontal slippage between the steel decking and the concrete. Figure 10 shows a typical crack pattern including the major shear failure crack. Figure 11 is a close-up view of a principal shear failure crack for a slab element. Shear-bond failure invariably is accompanied by "end slip" between the steel decking and the concrete, as can be visually observed at the end face of the slab. Figure 12 shows the end slip for a test slab.

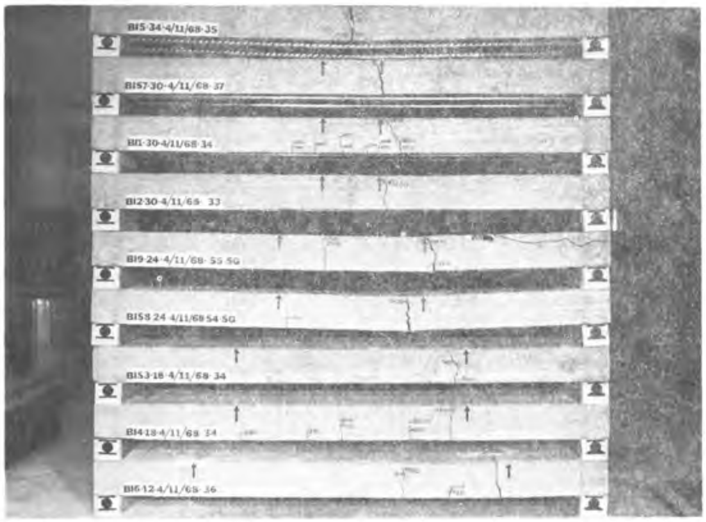


Fig. 9. Typical shear-bond failure of beams of various shear span lengths (arrows indicate loading point locations).

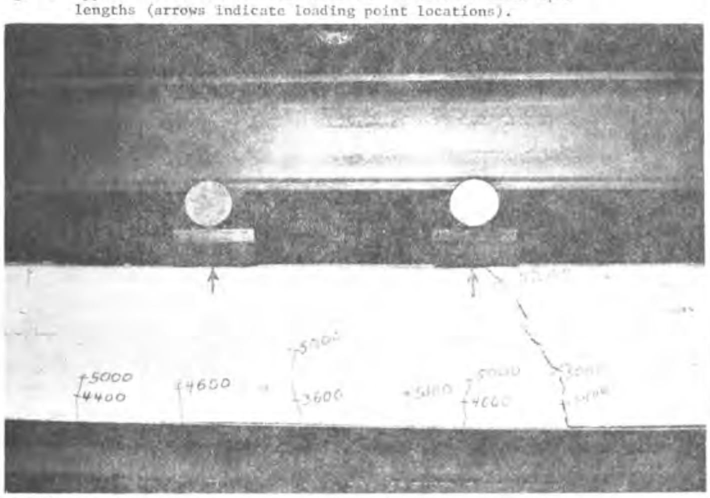


Fig. 10. Crack pattern of central portion of a one-way slab element with shear-bond crack near the load point.

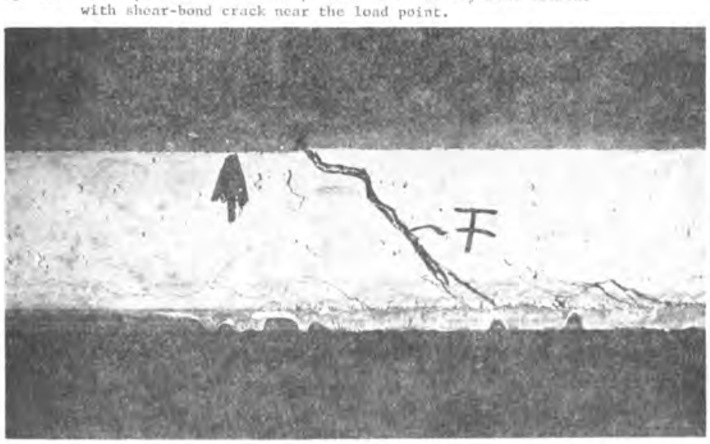


Fig. 11. Close-up view of a principal shear bond crack in a beam element.

Fig. 12. Typical end slip of a beam element failure in shear bond.

An expression for the ultimate experimental shear capacity, V_{ue}, (lb), of a slab element has been developed and is given as[3]

$$\frac{V_{ue} S}{bdp} = \frac{md\sqrt{f'_c}}{p\,L'} + k \qquad (1)$$

where

p = reinforcement ratio, $\frac{A_s}{bd}$

L' = shear span length, in. (Assume ¼ of the span length for uniformly loaded beams),

m = slope of regression curve,

k = intercept of regression curve,

S = center-to-center spacing of welded shear transfer devices, in. (in cases such as where embossments are employed in a fixed pattern, the value of S may be taken as unity),

b = width of cross section, in.

d = effective slab depth (distance from extreme concrete compression fiber to centraidal axis of steel deck), in.

A_s = cross-sectional area of steel deck per foot of width, in.^{2}

The use of Eq. (1) involves the determination of two constants, m and k. These constants are the slope and intercept, respectively, of the regression analysis of the linear relationship between $V_{ue}S/bdp$ and $d\sqrt{f'_c}/L'p$. The resulting total ultimate shear, V_u, (lbs) for slab elements of span length, L (ft), and dead load, W (psf), is given as

$$V_u = \frac{bd}{S}\left[\frac{md}{L'}\sqrt{f'_c} + kp\right] + \frac{WLb}{24} \qquad (2)$$

The load-deflection relationship for a slab element reinforced with steel decking (Type 1) is shown in Fig. 13. This applies to a member which had dimensions 6 ft long by 1 ft wide by 5 in. deep and subjected to two concentrated line loads at a distance of 30 in. from each reaction. Indicated on Fig. 13 is the ultimate predicted load, P_L, based on Eq. (1) as applied to Type I decking. Indicated also is the design load, P_{Des}, using the ∅ factor and load factors as recommended by the ACI Building Code[2]. The value of P_{Des} is observed to fall at the upper limit of the straight-line portion of the load-deflection curve, and well below the point of initial cracking. The ultimate experimental live load, P_e, was found to be 4000 lb with a midspan deflection of only 0.130 in. It is significant that the deflection was well within the L/360 criteria which is often used as an upper limit for normal construction. The shear-bond failure which occurred resulted in a sudden drop of beam strength. It seems evident, for this particular test at least, that the expression

$$P_{Des} = \frac{U - 1.5D}{1.8} \qquad (3)$$

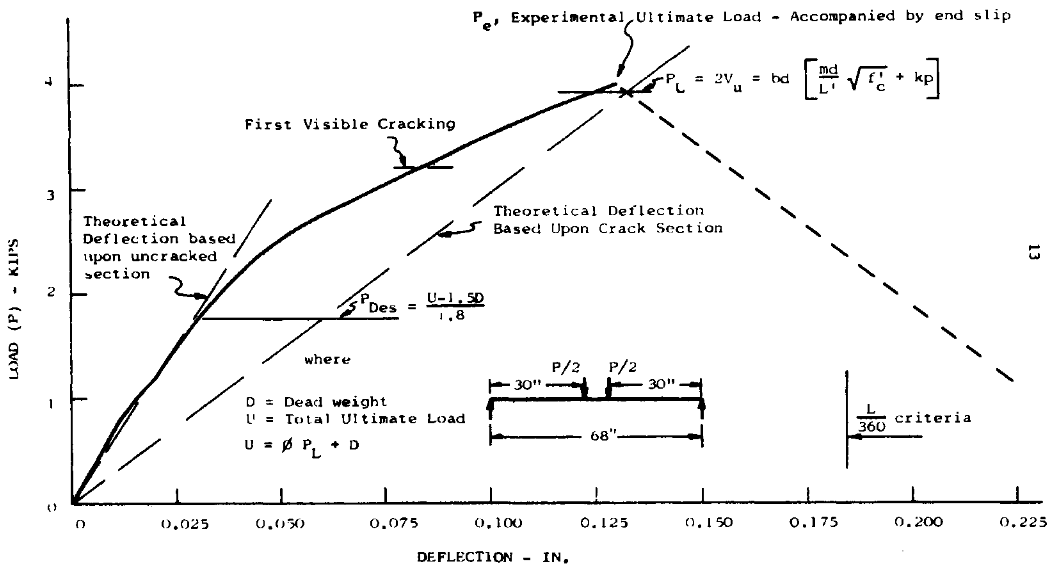


Fig. 13. Typical load vs. deflection curve for steel-deck-reinforces beam elements.

is quite reasonable. It remains to be seen, however, whether the ϕ and load factors will apply for the full range of possible steel-deck-reinforced concrete floor systems. There is admittedly some scatter on the basis of the limited number of ultimate strength tests which have been conducted to date. All of the systems also display a somewhat brittle, rather than ductile, behavior under high load, which might necessitate a more conservative selection of these factors. In all likelihood future testing alone can supply the answers.

Continuous Steel Deck Reinforced One-Way Slabs

The prediction of the predominate shear-bond type of failure which occurs in simple beams is even more difficult in continuous one-way slab elements. Figure 14 indicates how a continuous system may be broken down into equivalent simple beam segments by considering an effective distance L'' as that length occurring between the inflection points as determined from the usual indeterminate elastic analysis. In effect each segment subjected to positive bending with a length of L'' has been used for shear-bond analysis; thus, for continuous systems L'' is substituted in place of L in Eq. (2) and L' becomes the shear span corresponding to the reduced length L''.

Four three-span continuous beams (Fig. 15) have been tested as a part of a preliminary investigation involving the design and the ultimate load prediction of continuously reinforced steel deck slab elements. Two of the beams had conventional reinforcing bars as negative reinforcement over interior supports. In order to determine the effect of steel thickness, 16- and 22-gage decks were used. The basic behavioral characteristics of the four beams are illustrated in Fig. 16 showing applied load versus centerline deflection for the center span only. As would be expected the heavier gage steel-deck-reinforced beams had much stiffer characteristics. The method of analysis described above for shear-bond computations for continuous systems appeared to be quite adequate for the beams containing the 22-gage thickness of steel decking. However, the method did not give good results for systems with the much heavier 16-gage thickness of decking.

Full-Scale Floor Slab Tests

An analysis of the two-way action involved in floor slabs reinforced with cold-formed steel decking is not yet complete. For those cases involving flexure as the controlling design criteria, perhaps the yield-line theory is applicable; however, this has not yet been proven. The failure mode in the two slabs tested was that of shear-bond.

Figure 17 shows the dimensions and loading arrangement for two full-scale slab tests. 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. The following evidences of behavior were observed for each slab test:

1. Crack pattern development,
2. End slip data along the two opposite edges that are perpendicular to the corrugations,
3. Vertical deflections of slab at various points,

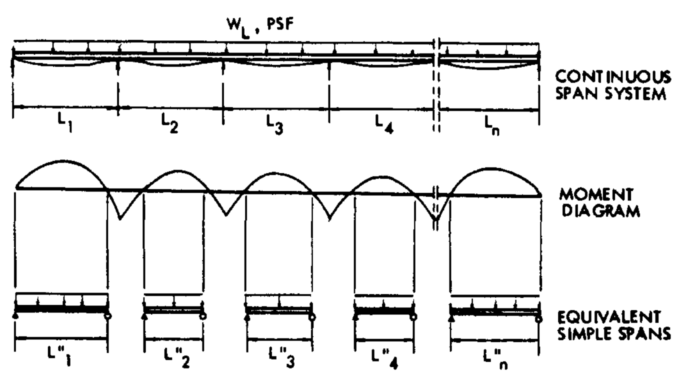


Fig. 14. Diagram of continuous beam analysis.

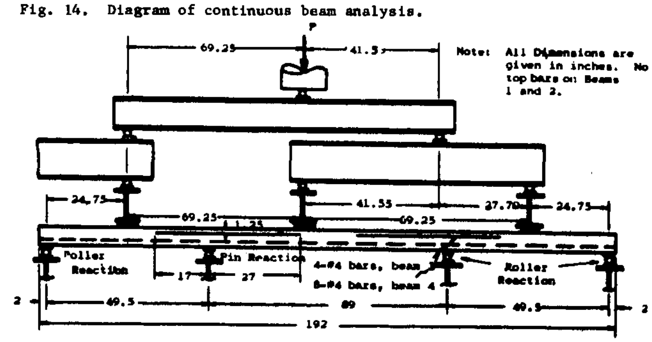


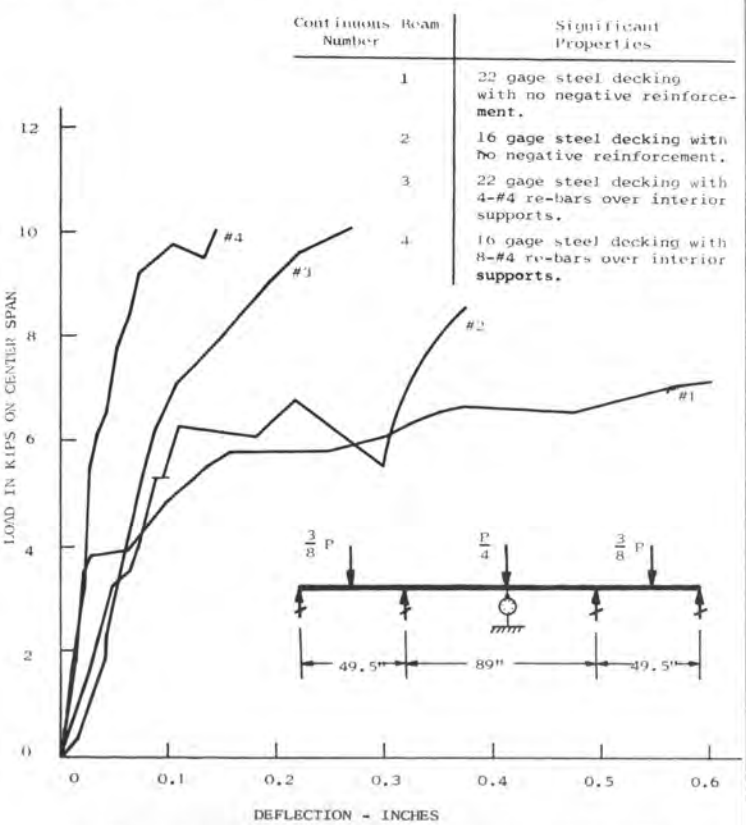
Fig. 15. Diagram of continuous beam test arrangement.

4. Bending strains at various points on the surfaces of the concrete and steel, and
5. Vertical reactions at edge of slab.

The first slab tested had all four corners tied down and all four support edges as simple supports. Figure 18 shows the instrumentation at one corner of the first slab involving the corner tie-down and the two types of reactive supports along two of the edges. Figure 19 shows all supports and the deflection dial grillage. Supplementary reinforcement placed directly on top of the decking consisted of 6 X 6 X 6/6 welded wire fabric for the first slab. Figure 20 shows the resulting crack pattern on top of the surface of the first slab at the time of test termination, after ultimate load had been achieved. The contours of the deflected surface at ultimate load of the slab are shown in Fig. 21.

The second test slab was exactly like the first except that no corner tie-downs were used, and the supplementary reinforcement was changed to 6 X 12 - 0/4 welded wire fabric. The heavier wire, about 0.30 in. diameter, was positioned at right angles to the deck corrugations. The jack load for the first slab had been increased continuously from zero to ultimate, whereas the second slab was loaded to about 60% of the expected ultimate load, then reduced at that point to zero. This loading and unloading procedure was repeated 10 times before making the final run to ultimate. The ultimate load for Slab 1 was 13.4 kips per load point, and for Slab 2 was 15.4 kips per load point. A comparison of the behavior of the two slabs is indicated in Fig. 22 showing the load versus deflection for the center point of the two slabs.

The crack patterns of the top surfaces of the two slabs were very similar except in the region of the corners (due to the absence of corner tie-downs on Slab 2). Figure 23 shows the top surface crack



Continuous Beam Number	Significant Properties
1	22 gage steel decking with no negative reinforcement.
2	16 gage steel decking with no negative reinforcement.
3	22 gage steel decking with 4-#4 re-bars over interior supports.
4	16 gage steel decking with 8-#4 re-bars over interior supports.

Fig. 16. Load vs. centerline deflection of various continuous beams.

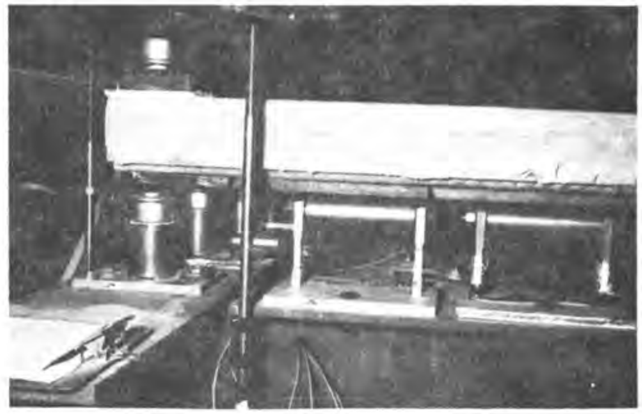


Fig. 18. Instrumentation to measure corner uplift force and vertical downward forces along edges of slab one.

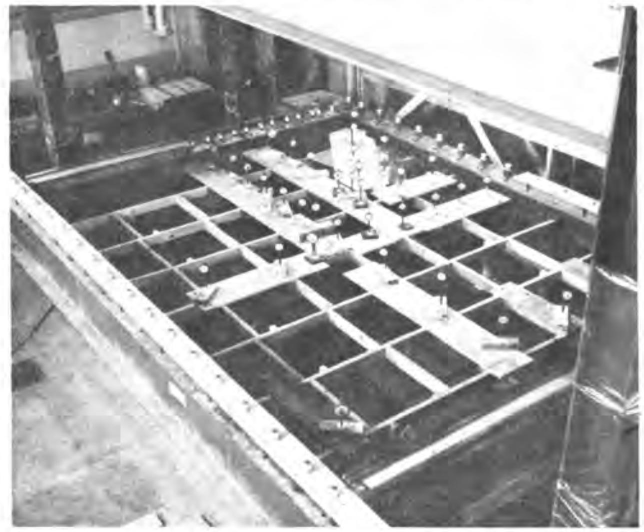


Fig. 19. Support and deflection dial arrangement for full-scale slab tests.

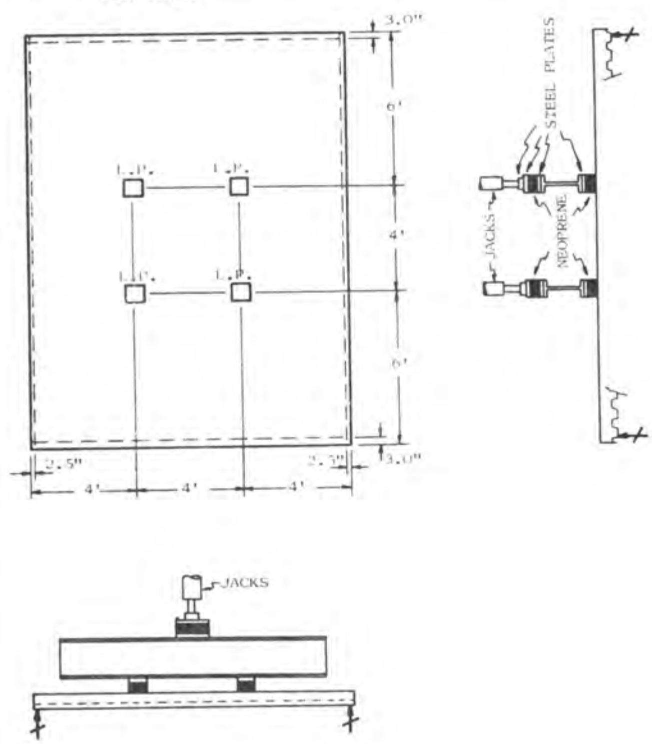
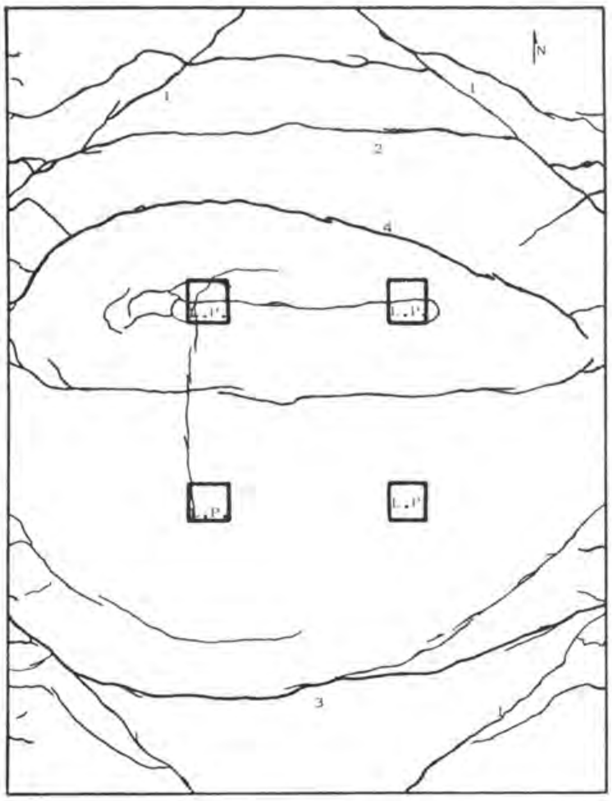


Fig. 17. Diagram of loading for slabs.



Scale: 1" = 2' - 0   Note: Numbers indicate approximate order of occurrance.

Fig. 20. Diagram of cracking of top surface of slab one at the termination of the test.

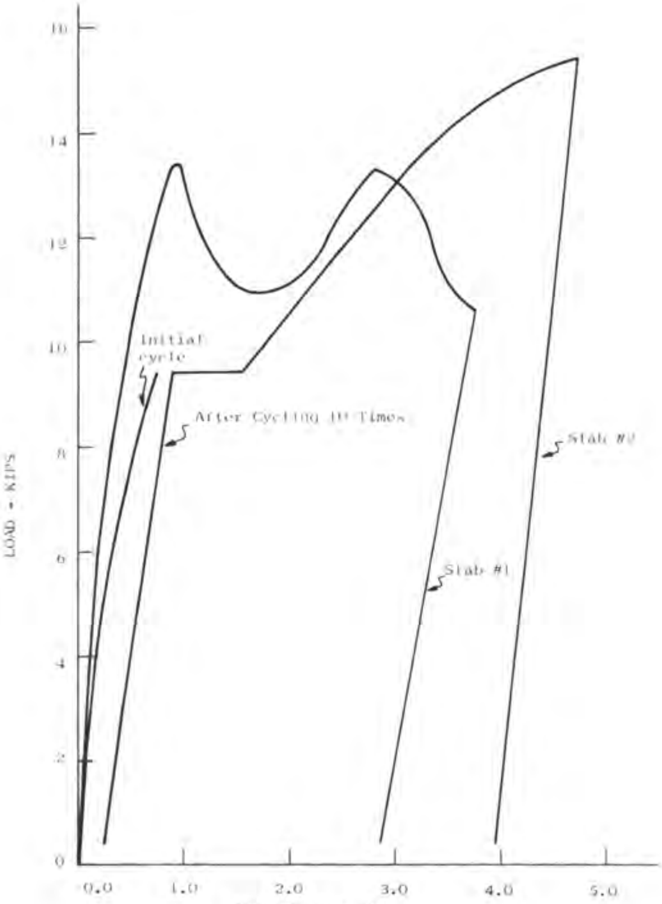
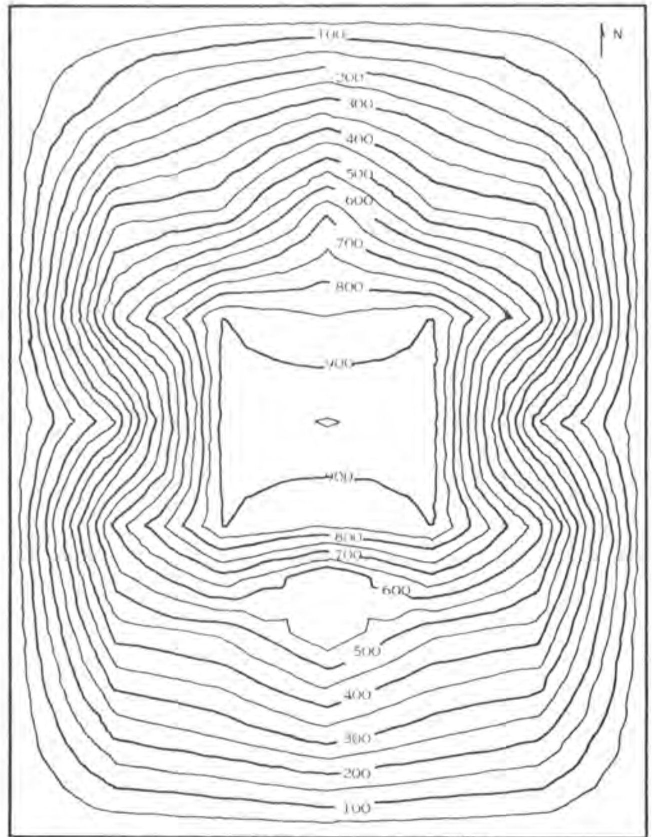


Fig. 22. Load vs. deflection curves for the center point of two full-scale slab tests.



Note: Contour numbers are deflections x 10^{-3} in.
Scale: 1" = 2' - 0

Fig. 21. Contours of points of equal deflection for Slab 1 at ultimate load (13.4 kips on each load point).

Fig. 23. Top surface of Slab 2 indicating crack pattern at termination of testing.

patterns of Slab 2 after test termination. More information about the behavior of full-scale floor slabs is expected to be available after completion of additional tests.

Summary

An attempt has been made to present the highlights of a continuing research project at Iowa State University concerning the design and use of steel-deck-reinforced concrete floor slabs. This work has primarily involved one-way slab elements and full-scale slabs, and has emphasized the development of an ultimate strength design concept. Much has yet to be undertaken on such problems as composite interaction of slab and supporting beam elements, continuous systems, slabs with cutouts, and effect on bond of surface coatings on steel decks. It must also be admitted that further refinement will be needed before any of the relationships for shear-bond presented herein may be considered valid.

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