optional plan to monitor potentials with an internal carbon probe will also be examined.

It is probable that extensive use will be made of the $Cu/CuSO_4$ half-cell as a means of external evaluation. Horizontal potential gradients are anticipated at the structure from the trench system because of the trench's relatively shallow burial. There could well be a tendency for the steel in the pavement near the trench to polarize first and for that farther away to polarize later. This being the case, as polarization occurs, a back electromotive force would develop in the closer steel and redistribution of current would result. A less noticeable horizontal gradient is expected with the posthole method. Because the anodes are buried deeper, they "see" the structure from a better angle, and potentials applied to the structure should be more uniform. $Cu/CuSO_4$ surface testing will again be used to measure applied potentials (absolute) and to check for possible gradients.

It should be obvious by now that there are still many unknowns in relation to the functional aspects of this system. If the situation had permitted, a more comprehensive approach to evaluating cathodic protection for application to CRCP would have been pursued. No doubt this would have consisted of conducting first a laboratory simulation, then a limited field trial, and finally a full-scale experimental test installation. Such an effort would have required 24-36 months to complete. Our present approach is to use the fast-track method, which is not unlike the approach used at the advent of the bridge-deck-spalling repair programs, a technique called "research by crisis".

CONCLUSIONS

In the past three years, more than $100 000/year has gone to patching or other ways of trying to maintain approximately 4 miles of four-lane Interstate highway that is now 10 years old. The distressed pavement is an 8-in slab reinforced with deformed wire mesh that was built by using the two-course construction technique. Since the corrosion phenomenon is for the most part irreversible, cathodic protection is now being examined as one possible solution to serious and rapidly advancing pavement deterioration problems.

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Study of Adhesive-Bonded Composite Concrete-Metal Deck Slabs

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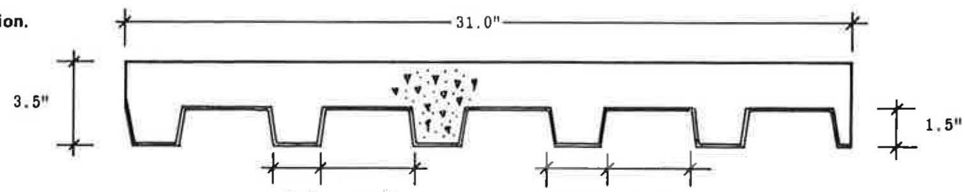
The results of a study conducted to determine the effectiveness of an epoxy resin as a shear connector in composite systems are presented. Composite concrete-metal deck slabs were constructed by using an epoxy resin to bond the concrete to the metal deck. Three composite specimens and three noncomposite control specimens were used in the test program. The concrete was plant mixed and trucked to the site by a local concrete supplier. No special additives were used in the concrete. All specimens were loaded to failure on a simply supported span of 3.66 m (12 ft). A four-point loading system was used. The loads were applied slowly, and impact loading was not considered. The noncomposite control specimens showed a fairly high percentage of partial composite action. Two of the three composite specimens failed by excessive deflection without reaching a definitive value of ultimate load. The adhesive-bonded composite specimens, based on serviceability criteria, carried more than twice the load carried by the noncomposite control specimens. The test results indicated that the epoxy bonder performed well as a shear connector and allowed the composite concrete-metal deck slabs to achieve full composite action. Additional studies are required to extend the results to both other composite systems and other types of loading.

There is a considerable attraction to be found in the use of adhesives as shear connectors for composite beams. Mechanical fasteners, while quite effective, furnish a horizontal shear connection only at a set of discrete points. There are also high local stress concentrations in the shear connectors and in the surrounding concrete.

On the other hand, the adhesive furnishes a continuous bonding plane at the point where the two dissimilar materials meet. Several references in the literature (1-5) show the feasibility of the

Figure 1. Specimen cross section.



Note: 1 in = 2.54 cm.

Figure 2. Location of strain gages.

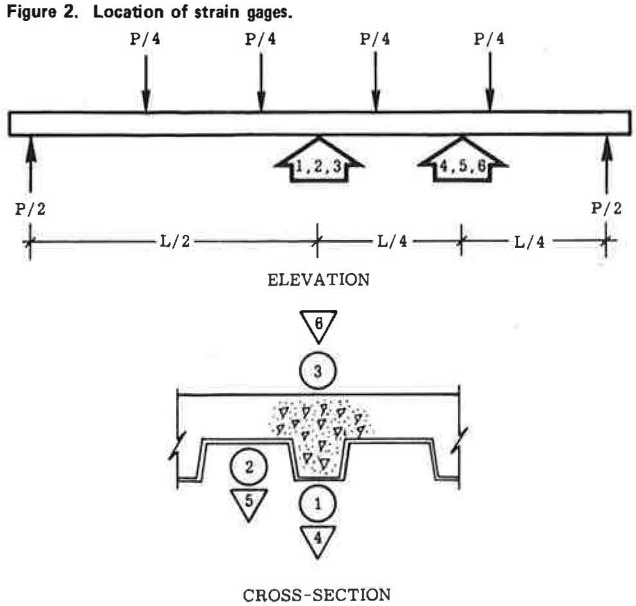
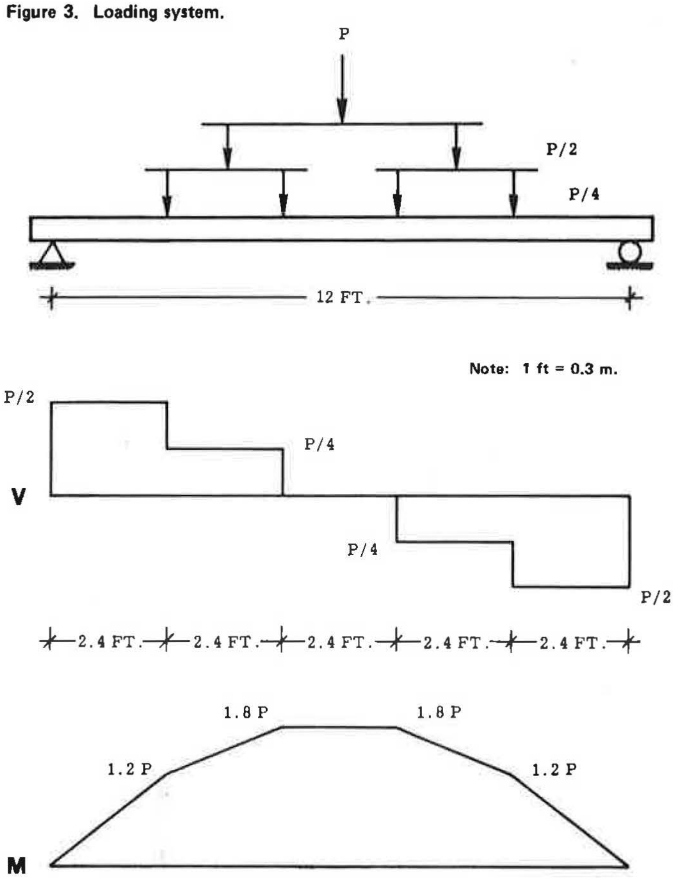


Figure 3. Loading system.



Note: 1 ft = 0.3 m.

Table 1. Summary of testing program.

Test	Type of Specimen	Method of Failure	Load (kN) Failure	Load (kN) Ultimate
1	Noncomposite control	Horizontal shear	11.1	23.1
2	Noncomposite control	Horizontal shear	11.6	26.7
3	Noncomposite control	Horizontal shear	10.7	23.1
4	Adhesive-bonded composite	Yield failure in metal deck, no loss of bond between slab and deck	26.7	Not reached
5	Adhesive-bonded composite	Horizontal shear	26.7	28.9
6	Adhesive-bonded composite	Yield failure in metal deck, no loss of bond between slab and deck	28.9	Not reached

Note: 1 kN = 224.8 lbf.

adhesive as a shear connector.

Epoxy bonding compounds are now widely accepted as a construction material in heavy construction, although their use in composite construction has been quite limited. Up to this time, adhesives have been used in composite bridge construction in connection with the use of externally bonded thin steel plates to strengthen existing reinforced concrete bridges. In lighter construction, such as residential and light commercial buildings, elastomeric adhesives have been used successfully in composite members. Concrete slab on metal deck would appear to be the ideal type of construction for adhesive bonding because of the large bond area available betwen the metal deck and the slab.

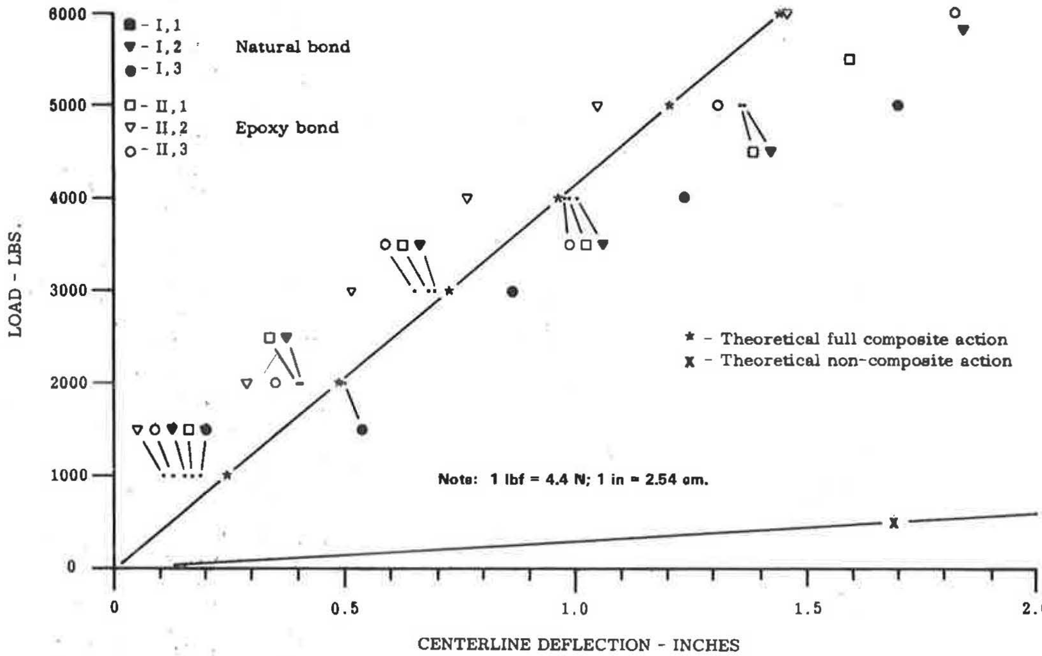
Steel deck composite beams are usually associated with building construction, where the steel decking can provide raceways for electrical conduit and telephone services. However, "stay-in-place" metal forms are gaining wider acceptance in bridge construction. If these metal forms can act to form a composite slab, considerable economy can be achieved in construction.

TESTING PROGRAM

The specimens used in the testing program were 3.96 m (13 ft) long and 0.76 m (2.5 ft) wide. The specimens were formed on 22-gage metal deck. Concrete was placed to a depth of 51 mm (2 in) above the high flutes of the deck. Figure 1 shows a typical cross section of a specimen. Non-air-entrained concrete with a strength of 34.5 MPa (5000 lbf/in^2) was used in the project; it was plant mixed and trucked to the site by a local concrete supplier.

The metal deck sections were sandblasted clean. Then a brush coat of a water-compatible epoxy was applied to the deck. The epoxy bonding agent consisted of an epoxy resin and an epoxy hardener; the two were blended together, mixed with water, and applied to the surface of the metal deck at a rate of 4.9 m^2/L (200 ft^2/gal). Because the length

Figure 4. Load-deflection curves.



of time from mixing to initial set of the glue (pot life) was only 1.75 h, its application before the concrete was cast had to be carefully timed. The epoxy used met the requirements of ASTM C882-78. Finally, the concrete was cast on the freshly applied epoxy coat. The control specimens were constructed by casting the concrete directly on the cleaned metal deck sections.

The specimens were loaded as simply supported beams with a span of 3.66 m (12 ft). The loading was applied as four equal line-load concentrations, equally spaced along the span. This loading simulated a uniform load condition and provided a very small shear-free region at midspan.

Figure 2 shows how sets of strain gages were mounted at the quarter point and centerline of the 3.66-m span. At both locations, three gages were applied: two on the bottom of the metal deck on the upper and lower flutes and one directly above on the top of the slab. Dial gages to measure deflection were placed under the deck at midspan.

Loading was applied to the specimens in 890-N (200-lbf) increments at a slow, uniform rate of approximately 45-60 s. No impact loading was used. Figure 3 shows the loading system applied to one of the slabs.

The strength of the noncomposite control specimens was calculated based on the strength of the concrete and the metal deck acting independently. No natural bond was assumed between the slab and the deck. No interior reinforcing was used in any of the slabs.

TEST RESULTS

The types of specimens used, their methods of failure, and their failure loadings are summarized in Table 1.

The specimen in test 1 carried much more than its design load, which indicates the presence of enough natural bond to give partial composite action. No load-deflection curve is available for specimen 1 because the gage was damaged in the course of the test. The first cracking and local bond failure occurred at a very low load--3.3 kN (750 lbf). At 11.1 kN (2500 lbf), horizontal shear failure was noted at both ends of the specimen. The specimen continued to carry load with increased cracking, up to ultimate load, when there was a sudden collapse of the specimen.

The specimen in test 2 also showed the presence of some natural bond and partial composite action. The first cracks that appeared in the specimen appeared at 6.7 kN (1500 lbf) and were horizontal cracks in the region of high flexure and zero shear. Local bond failure occurred near midspan at 11.6 kN (2600 lbf). Specimen 2 showed behavior similar to that of specimen 1 up to ultimate load.

In test 3, the first cracks appeared at 8.9 kN (2000 lbf). Horizontal shear cracks were noted at both ends at 10.7 kN (2400 lbf). The specimen continued to carry load, with increased cracking, up to ultimate load.

In test 4, the first crack in the specimen was a vertical crack near midspan, which formed at 22.2 kN (5000 lbf). As a result of continued deflection, there was local crushing adjacent to this crack. At 26.7 kN (6000 lbf), significant yield in the metal deck at midspan was noted. The specimen continued to deflect excessively, with only minor increase in load, up to 31.1 kN (7000 lbf). The bond remained intact between the metal deck and the concrete. At this point, the specimen had deflected 152 mm (6 in) and exceeded the stroke of the testing machine.

In test 5, the first vertical crack appeared at the centerline at 17.8 kN (4000 lbf). Under continued loading, the specimen showed horizontal shear failure at 26.7 kN (6000 lbf). At 28.9 kN (6500 lbf), the specimen would accept no more load.

In test 6, the first vertical crack appeared at one interior load point at 13.3 kN (3000 lbf). Another vertical crack near the interior load point appeared at 20 kN (4500 lbf). The specimen continued to carry load with no loss of bond. At the failure load, the deck yielded significantly but continued to carry load with no other distress until it exceeded the stroke of the testing machine.

Theoretically, it is impossible to attain 100 percent composite action because any amount of elasticity in the shear connection causes at least a negligible amount of slip. It is also theoretically impossible to obtain absolute noncomposite action because of friction and the presence of some natural bond between the components of the member.

However, a member may be considered to have full composite action if one of the components of the member fails without rupturing the shear connection. This type of failure was achieved in two of the three composite specimens. The third composite specimen did show a horizontal shear failure but only after the load equaled the failure load of the other composite specimens.

INTERPRETATION OF RESULTS

The expected mode of failure in flexural members without adhesive bonding is a horizontal shear failure at the interface between the two dissimilar materials. All of the noncomposite specimens did fail in horizontal shear between the concrete slab and the metal deck. The dimensions of the slab unit were selected so that each specimen could resist a uniform loading of 5 kPa (100 lbf/ft^2). No natural bond between the concrete and metal deck was assumed for these calculations. However, the unbonded members all carried load well in excess of their computed capacity, which indicated the presence of a relatively high percentage of partial composite action.

In this program, failure was considered as a lack of serviceability of the unit. In the case of the noncomposite control specimens, this lack of serviceability was defined by horizontal shear and separation of the decking from the slab. In two of the three composite specimens, no horizontal shear failure occurred and lack of serviceability was defined by excessive deflection. The load-deflection curves (see Figure 4) show that the adhesive-bonded composite specimens deflected less than the noncomposite control specimens by an average of 15 percent.

CONCLUSIONS AND RECOMMENDATIONS

As defined by serviceability criteria, the composite specimens carried more than twice the load of their unbonded counterparts. These results demonstrate that the adhesive-bonded concrete-metal deck member can achieve full composite action (3,6).

It is difficult to form a comparison based on ultimate load because two of the three composite specimens never actually reached an ultimate load but continued to deflect with increasing load. These tests were terminated because the deflection exceeded the stroke of the testing machine. The one composite specimen that demonstrated horizontal shear failure showed only an 8 percent increase in ultimate load capacity compared with its unbonded counterparts.

Some aspects of adhesive bonding that might be recommended for study include the following:

1. Long-term creep effects on the effectiveness of the adhesive shear connection,
2. Determination of the amount of natural bond that exists between the slab and the metal deck,
3. The effects of cyclic and impact loading on the adhesive shear connection, and
4. Definition of the amount of permissible slip between the slab and the metal deck, which should be correlated with a study of the properties of various adhesives.

The results presented here are not expected to have any immediate impact on current practice, but they are one more contribution to the growing history of adhesive-bonded composite members.

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Evaluation of the Durability of Metal Drainage Pipe

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Preliminary results are presented of a 10-year field study undertaken in Louisiana to determine the ability of aluminum and galvanized-steel culverts to resist corrosion in moderate, acidic, and low-electrical-resistivity environments. In 1973, 10 types of aluminum and galvanized-steel culverts were installed, generally as side drains, at 10 test locations. One pair of each type of culvert was installed at each site. Every two years, investigators are removing one designated culvert of each of the pairs and subjectively rating the condition of the metal and protective coating. Field samples of the culverts are evaluated in the laboratory, and the test culverts are installed again after each inspection. The undisturbed mate of each pair remains buried for the duration of the study to analyze the impact of the periodic inspections. After 6 years of field exposure, the asbestos-bonded, bituminous-coated, galvanized-steel culvert is resisting corrosion quite well even in brackish water, an environment characterized by low electrical resistivity. Several aluminum and coated galvanized-steel culverts appear to be well suited for the normal range of acidic environments encountered in Louisiana. All of the test culverts appear to provide satisfactory service life in the test environment designated as moderate.