CONCLUSIONS

1. Thin PC overlays that provide low permeability and high skid resistance can be installed on bridge decks with a minimum of disruption to traffic and at about one-half of the cost of alternative service-life-extending measures such as portland cement concrete overlays.

2. Laboratory tests indicate that the temperature changes to which bridge decks in Virginia are typically subjected are sufficient to cause deterioration and eventual failure of the overlays. The deterioration results from the development of shear stress in the vicinity of the bond between the concrete and overlay because of differences in the moduli of elasticity and the coefficients of thermal expansion of the materials. The Van Vlack equation provides a reasonable indication of the magnitude of the stress.

3. The thermally induced stress can be held to a minimum by placing the overlay at 60°F to 70°F, by applying an excess of aggregate to the resin, and by specifying a resin with a low modulus of elasticity and a low coefficient of thermal expansion.

4. Thermally induced cracks have been noted in the overlay, the base concrete, and the bond interface--a majority of them in the medium least able to withstand the stress. Cracks in the overlay increase its permeability and cracks in the base concrete or the bond interface lead to delamination of the overlays.

5. The base concrete is least likely to fail when its shear strength is high, its surface is prepared by shotblasting, and an excess of aggregate is applied to the resin.

6. It is estimated that a properly installed overlay prepared with either of the two polyester resins tested in this study will have a useful service life of at least five years that, considering its ease of application, may be acceptable for bridges where it is difficult to close a lane to make a more permanent repair.

ACKNOWLEDGMENT

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REFERENCES

1. Applications of Polymer Concrete. American Concrete Institute, Detroit, MI, Publ. SP-69, 1981, pp. 21-43.
2. Polymer Concrete Overlays: Interim User's Manual, Method B. U.S. Department of Transportation, Rept. FHWA-TS-78-225, Feb. 1978.
3. L.H. Van Vlack. Elements of Materials Science. Addison-Wesley Publishing Company, Inc., Reading, MA, 1967.
4. Special Provision for Polymer Concrete Overlay. Virginia Department of Highways and Transportation, Richmond, April 1981.
5. D. Whiting. Rapid Determination of the Chloride Permeability of Concrete. FHWA, Rept. FHWA-RD-81/119, Aug. 1981.

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Effects of Creep on Adhesive-Bonded Composite Concrete-Metal Deck Slabs

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In this study, the long-term effect of creep on the effectiveness of adhesives as shear connectors between concrete and corrugated steel decking in composite slabs was analyzed. Approximately three years ago, eight specimens were constructed: four control specimens with no adhesive bonding the concrete to the steel deck and four specimens that used an adhesive as a shear connector. Six of these specimens (three control and three adhesive) were tested three years ago. The remaining two slabs were loaded with a uniform load of 150 lb/ft^2 for 2.5 years. These remaining two slabs were then loaded to failure in the identical manner that the earlier six slabs were and the same parameters were measured. Direct comparison of the parameters provided insight as to the effects of this prolonged creep loading on the effectiveness of the adhesive as a shear connector. Instead of the strength loss, which might have been expected, the creep-loaded specimens actually showed a very slight gain in strength and stiffness.

The use of composite deck slabs in transportation structures is not new. Steel-concrete slabs have been used for parking structures, and, in the western United States, timber-concrete composite decks have been used successfully for short span bridges. In commercial buildings, of course, the steel-concrete composite slab is in very wide use (1).

Structurally, the steel-concrete composite slab is a very advantageous combination, since the concrete accepts the flexural compressive stress and the steel deck accepts the flexural tension stress. The capacity of the composite slab is greatly influenced by the strength of the connector used at the interface of the concrete and the steel deck. When the composite slab is subjected to flexure, a horizontal shear stress develops at the interface. The connector must be able to accept this shear stress and prevent any slippage, which would destroy the structural integrity of the composite slab. The most common type of connector is the welded stud that projects up into the concrete slab. Adhesives as shear connectors in composite construction (2) can be advantageous from two points: (a) the adhesive furnishes a continuous bonding plane, thereby eliminating shear concentrations caused by the discrete shear connectors, and (b) the adhesive eliminates the installation of separate connectors, which

Figure 1. Typical cross section.

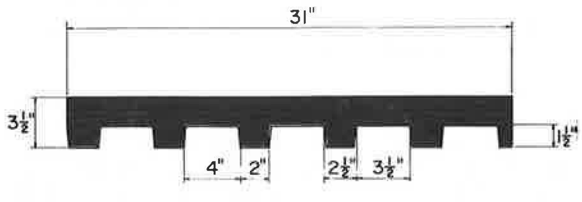


Figure 2. Schematic of loading system with shear and moment diagrams.

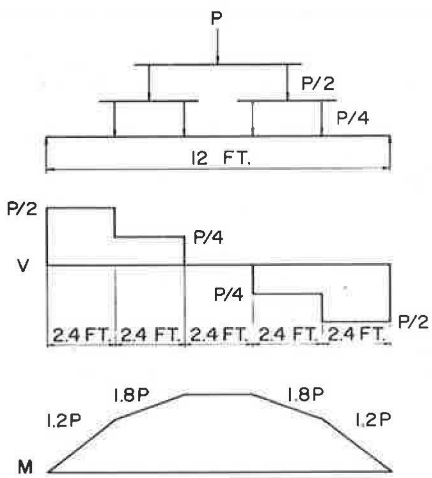
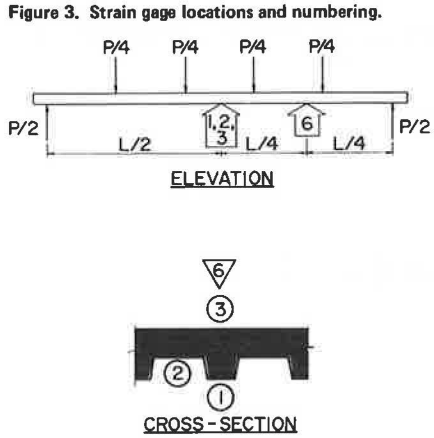


Figure 3. Strain gage locations and numbering.



reduces construction hazards and construction time. The problem, therefore, is to prove that the adhesive provides enough strength and elasticity to fully develop the capacity of the concrete and steel. Proof, without question, must include tests to determine the performance of the adhesive when subjected to long-term loading.

Approximately three years ago work was done on the effectiveness of adhesives as shear connectors for composite concrete-steel decks by McKee and Cook (3) at the University of Cincinnati. Four slabs that used an epoxy resin as a shear connector between concrete and steel and four slabs that used no shear connector were constructed. Three of each were tested, and the investigators found that the adhesive-bonded specimens, based on serviceability criteria, carried more than twice the load of the noncomposite control specimens.

The two remaining slabs (one each) were set aside and strain gages were placed on the steel deck and on top of the concrete. Both slabs were then uniformly loaded with 150 lb/ft² and left for 2.5 years, which allowed natural creep due to load to occur. After 2.5 years, the creep strain was measured and the slabs were unloaded.

These remaining two slabs were then tested in the identical manner that the previous six slabs were and the same parameters were measured. Direct comparison of the results with the results obtained from the previous tests provided the necessary evidence to evaluate the performance of the adhesive slabs when loaded for an extended period of time.

TEST SPECIMENS AND LOADING

The test specimens were constructed of 22-gage corrugated steel deck and non-air-entrained concrete of f'_c = 5000 psi. The adhesive used was a water-soluble epoxy applied to the sandblasted top side of the steel deck. The adhesive used was an epoxy that meets the requirements of ASTM C-881. The material was furnished by Protex Industries of Denver, Colorado. The specimens were 13 ft long and 2.5 ft wide. A typical cross section is shown in Figure 1. The overall thickness of the slab was 3.5 in with 2 in of concrete above the upper flute. The lower flutes were 3.5 in wide. In the control specimen, the top side of the steel deck was also sandblasted, but no epoxy bonder was used.

The slabs were loaded on a simply supported span of 12 ft. Four equally spaced concentrated line loads were used. The four-point loading setup was used because the moment diagram that results from this loading closely resembles the moment diagram that results from a uniformly distributed load. The loading system and the resulting shear and moment diagrams are shown in Figure 2. As is evident in the shear diagram, the maximum shear was developed at the supports. Therefore, if a horizontal shear failure occurred, it would do so at or near the supports. Also of significance was that the maximum moment occurred at the center of the span, and any flexural failure would occur in the center portion of the span.

MEASUREMENTS

Two parameters were measured during the testing of the specimens: strains and centerline deflections. In order to measure the strain, four type SR4 A-1 120-ohm strain gages were mounted at various points on each specimen. Three of the gages were applied to the specimen at the centerline, one gage on top of the concrete slab and two gages on the bottom of the steel deck on the upper and lower flutes. The fourth strain gage was located at the quarter point on the top of the concrete slab. A manual strain indicator and switch and balance unit were used to determine the strain measurements. The gage factor for the strain gages was 2.03 ± 1 percent. Figure 3 shows the strain gage locations and their numbering. All of the strain gages were applied to smooth, clean, sanded surfaces by means of a thin coat of epoxy. The other parameter that was measured was the centerline deflection. This measurement gives an indication of the stiffness of the slab and also determines the serviceability of the slabs. Two dial gages located at the edges of the slab at the centerline and supported by a separate frame were used to determine the deflections at various loads.

Loading of the control specimen was faster than intended. Nevertheless, it did yield the necessary information for purposes of comparison. The adhesive specimen was loaded at 500-lb increments up to the failure load. The loads were applied at a uni-

Figure 4. Load versus strain, gage 1.

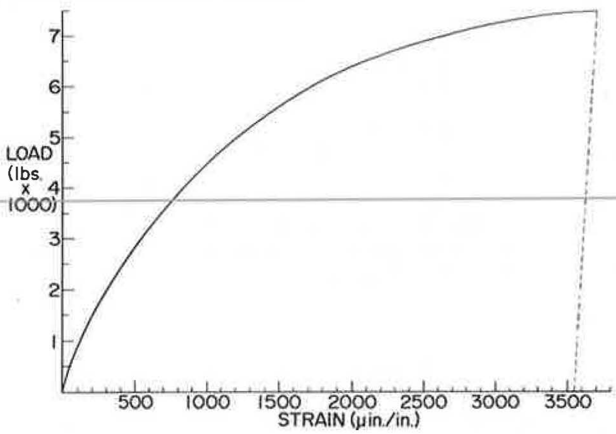
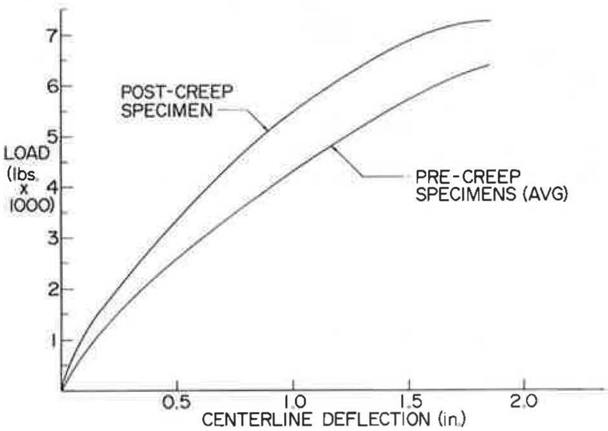


Figure 5. Load versus deflection.



form rate such that the full increment was applied after 30-45 s. Strain readings and deflections were taken after they had reached a constant value. In the adhesive slab test, this was approximately 10-15 min after the load had been applied. Each specimen was loaded to failure or excessive deflection. The specimen was considered failed when the structural integrity of the composite member was lost.

The permanent sag of the control slab before loading was 0.875 in. The following observations were made about the control specimen. An inspection of the end of the failed specimen showed a shear slip of 0.75 in between the concrete and steel deck. Both ends of the failed specimen exhibited this characteristic. Besides the large amount of slip, the concrete cracked due to flexural tension directly under the applied load nearest midspan. The corrugated steel deck also buckled directly under the transverse tension crack in the concrete. Inspection of the failed slab also indicated that very little bond remained between the concrete and metal deck.

The permanent sag of the composite slab prior to testing was 0.375 in. The specimen was successfully loaded at 500-lb increments and strain and deflection data were recorded. The specimen was loaded to a failure load (excessive deflection) of 7500 lb and then to an ultimate load of 8100 lb. The first vertical flexural tension hairline crack appeared under one of the line loads at 4500 lb. Several more hairline flexural tension cracks appeared at various points at 5500 lb. At 6500 lb, several new cracks appeared and widening of the existing cracks occurred. At 7000 lb there was a proliferation of cracks. Failure occurred at 7500 lb when the corrugated steel deck yielded in tension at the centerline. In addition to the yielding of the steel deck, large vertical tension cracks occurred in the concrete at midspan. The load was increased to 8120 lb before the slab failed completely. No slippage or horizontal shear failure occurred anywhere on the slab. The data collected from the adhesive specimen tests are given in tables below where the strain gage numbering is the same as that shown in Figure 3 (note, + = tension and - = compression):

Total Load, P (lb)	Strain Gage Reading (μin/in) No. 1 (+)	No. 2 (+)	No. 3 (-)	No. 6 (-)
0	0	0	0	0
500	52	20	32	28
1000	110	40	59	52
1500	197	76	98	82
2000	-	-	-	-
2500	418	190	198	150
3000	572	268	250	192
3500	712	328	298	230
4000	864	386	344	270
4500	1016	470	396	310
5000	1204	574	456	350
5500	1414	678	510	390
6000	1684	810	578	430
6500	2088	1014	670	475
7000	2626	1286	763	516
7500	3690	1670	904	562
8120	-	-	-	-
0	3545	1165	390	70

Total Load, P (lb)	Centerline Deflection (in) Pre-creep	Post-creep
400	0.044	-
500	0.057	0.041
1000	0.130	0.086
1500	0.230	0.147
2000	0.345	0.224
2500	0.473	0.316
3000	0.616	0.429
3500	0.760	0.530
4000	0.906	0.638
4500	1.064	0.746
5000	1.239	0.864
5500	1.437	1.006
6000	1.644	1.150
6500	-	1.346
7000	-	1.580
7500	-	2.900

Figure 4 shows a plot of total load (P) versus strain for gage no. 1. Gage no. 1 was located on the lower flute of the corrugated steel deck at the centerline. This was the point of highest tensile stress, and hence the point where yield first occurred. Inspection of the curve shows that the steel did yield. This is evident because (a) the curve approaches tangency to a horizontal line at the failure load and (b) when the load was removed the strain remained at 3545 μin/in. This information confirms the fact that the steel deck yielded in tension, which caused failure of the adhesive slab.

To determine whether the prolonged loading reduced the strength of the adhesive as a shear connector, two things were considered: (a) the type of failure exhibited by the postcreep specimen and (b) the stiffness of the specimen. As discussed earlier, the adhesive specimen failed due to yielding of the steel deck. The second point is best considered when one views Figure 5. This is a plot of total load (P) versus centerline deflection for the epoxy specimens tested. The lower curve is comprised of the average of the deflections of the three adhesive specimens tested 2.5 years ago. If the prolonged loading had reduced the strength of the adhesive, one would expect to see the postcreep curve to be lower than the precreep curve. The lower the curve, the more deflection occurs per unit load, which implies a smaller relative stiffness. Inspection of Figure 5 indicates that the opposite has occurred: The postcreep curve is higher than the precreep curve. This verifies that the creep did not reduce the strength of the adhesive. Two things that would increase the strength of the post-creep slab are (a) curing of the concrete and (b) curing of the epoxy. However, these effects comprise only a very small percentage of the total strength of the slab and could not increase a post-creep curve that lies below the precreep curve to a position above the precreep curve. These two factors--mode of failure and stiffness--provide sufficient evidence to conclude that the prolonged loading and resulting creep did not reduce the effectiveness of the adhesive as a shear connector.

CONCLUSIONS

Testing of the control specimen revealed several things. It was concluded that the prolonged loading and resulting creep destroyed much of the natural bond between the concrete and deck exhibited in the earlier slabs. Therefore, the concrete slab and metal deck were acting independently, and the failure load, although not measured, was probably little more than the modulus of rupture of the concrete slab itself. This suggests the necessity of some type of shear connector in order to provide composite action.

As mentioned before, the obvious conclusion that resulted from testing of the adhesive specimen was that the creep had no effect on the effectiveness of the adhesive as a shear connector.

Some recommendations for future research include

1. Studying the effects of cyclic and impact loading on the adhesive shear connection,
2. Determining the amount of permissible slip between the concrete slab and the steel deck, and
3. Subjecting the composite slabs to the same type of weathering test that is being used by Calder (4) in the United Kingdom.

REFERENCES

1. J.P. Cook. Composite Construction Methods. Wiley, New York, 1977.
2. H.A. Miklofsky, M.R. Brown, and M.J. Gonsior. Epoxy Bonding Compounds as Shear Connectors in Composite Beams. New York State Department of Public Works, Albany, Physical Res. Project 13, Interim Rept., 1962.
3. D.J. McKee and J.P. Cook. Study of Adhesive-Bonded Composite Concrete-Metal Deck Slabs. TRB, Transportation Research Record 762, 1981, pp. 22-25.
4. A.J.J. Calder. Exposure Tests on Externally Reinforced Concrete Beams: First Two Years. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, TRRL Supplementary Rept. 529, 1979.

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