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Foreword

The modern structural engineer is no longer confined to the so-called traditional methods and materials used for so many years in bridge construction. In concert with the materials engineer, the structural engineer has developed uses for epoxy and other plastic compounds for special design applications. In addition, he is using new forms of old materials as well as highly improved generations of the old materials. The five papers in this RECORD illustrate the continuing efforts that are under way in developing new materials and improving the old.

The first report deals with a series of tests on the use of epoxy compounds as shear connectors for composite T-beams. In addition to simple span tests, one of the T-beams was tested as a continuous structure and another in the low temperature range. This series of tests indicated that it is practical to utilize epoxy aggregates to develop composite action between a reinforced concrete deck and a structural steel supporting member. The authors feel that the main advantage in this application would lie in the reduction of bridge deck cracking and the elimination of the construction hazard which is created by the raised mechanical connectors now being used.

The next report is especially timely insofar as the bridge construction engineer is concerned. It reports on the effect of vibrations created by pile driving on adjacent existing structures. It describes actual tests involving the several types of soils as well as different methods of pile driving. In addition, tables and graphs are presented showing comparisons of damage criteria for residential structures. Of special interest for application during the construction of concrete structures is a table showing the limiting safe distance versus curing time of concrete for pile-driver operation.

The third paper deals with the surfacing of orthotropic steel deck bridges. This is an area that greatly concerns the structural engineer and this report, while it does not develop any new ideas, is an excellent state of the art summary. The present status of such wearing surfaces both in Europe and North America is discussed. European practice relies primarily on the layered paving system consisting of a prime coat and insulation layer, a leveling surface and a surface course on the steel deck plate, whereas American practice seems to be equally divided between the layered system using an epoxy prime coat and one or two layers of either sand asphalt or asphaltic-concrete. The alternate to this is a single-layer sand resin mixture. It appears that most of the systems now in use in the United States are experimental and so far not considered to be completely successful. European practice, because of the availability of different types of materials than are economically available in the United States, does not seem to be entirely applicable to our conditions.

The fourth paper supplements the information given in the above state of the art report in that it outlines the testing program being followed in the development of an appropriate wearing surface for the new orthotropic bridge across San Francisco Bay from San Mateo to Hayward. This paper discusses the various factors that must be considered in the design of a layered system and outlines the method of preparation of test specimens as well as the methods of test. Materials involved in the test up to the time of the report are also given; however, the final design of the paving materials had not yet been determined.

The last paper is of special interest to the structural designer in that it develops a mathematical solution based on experimental results for determining stresses in a three material beam. The results of the testing program were practically identical with the analytical solution. The theory developed will be found to be well worth study by all structural designers in that it removes some of the guesswork now necessary in the assumptions utilized during the conventional approach to this problem.

Contents

Use of Epoxy Compounds as a Shear Connector in Composite T-Beams

JAMES D. KRIEGH, Associate Professor of Civil Engineering, University of Arizona,
GENE M. NORDBY, Professor of Civil Engineering and Dean, College of Engineering, University of Oklahoma, and
ELTON G. ENDEBROCK, Assistant Professor of Civil Engineering, New Mexico State University

•APPROXIMATELY twenty years ago, epoxy resin compounds were introduced in this country. Since that time new developments and improvements of the compounds have been made, and practical applications in the highway field have been established. With the exception of the aircraft industry, the one area which has remained largely unaffected by these compounds is that of structural connections for the building and highway construction industries. The work of the aircraft industry in bonding plastic or metal cores to metallic skins for forming sandwich panels (1) and the reports of high tensile and shear strengths both in bond and in material have suggested that further practical applications of the epoxy compounds as a structural connector are warranted. One of the first such possible uses in the highway field subject to investigation was the use of epoxy resin compounds as a shear connector for composite T-beams.

The New York State Department of Public Works reports (2) that the first research conducted for the specific use of epoxy resin compounds as a shear connector was carried on in the laboratories of the Sika Chemical Corporation, Passaic, N.J. The Civil Engineering Department of the University of Arizona began similar investigations in 1959, and on July 1, 1960, they continued research under a formal contract with the Arizona Highway Department and the U.S. Bureau of Public Roads. Similar efforts were proposed by the senior bridge engineer of the California Division of Highways in 1961 (3), and in October 1962 the Civil Engineering Department of Rensselaer Polytechnic Institute, under contract to the Bureau of Physical Research, Department of Public Works, State of New York, reported the first half of its investigations (4).

ADVANTAGES OF EPOXY SHEAR CONNECTOR

Three highway departments, two at opposite boundaries of the continental United States, independently gave consideration to this proposed use of epoxy resin compounds and considered such studies worthy of support. This consideration was, and still is, supported by the U.S. Bureau of Public Roads. The possible benefits to be gained by this type of construction include the following.

1. Reduction of Bridge Deck Cracking.—The deterioration of bridge deck slabs, where mechanical connectors have been used, may be explained in part by the presence of high thermal and shrinkage stresses (this point has been subjected to much controversy among authorities). These stresses, when combined with vibration and impact stresses, may cause excessive strains and stresses resulting in cracking which may later lead to the spalling of the bridge deck. In support of this theory it is reported that visible shrinkage cracks did not develop in the slabs of epoxy-glued T-beams,

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whereas similar mechanically connected T-beams developed large visible shrinkage cracks (5). This difference apparently existed because the epoxy compound was able to adjust during its setting process to the shrinkage of the concrete during its curing process. Likewise, the elastic properties and low modulus of elasticity of the epoxy compound probably alleviate thermal stresses to a great degree by providing a membrane which can adjust between the slab and stem later while the beam is in service.

2. Provision of a Continuous Connector.—Mechanical connectors, by the very nature of the function provided, develop stress concentrations in the bridge decks along the points of contact. An epoxy compound used in this manner provides a continuous connection between the deck and beam or girder, eliminating stress concentrations and resulting in more integral beam action.

3. Elimination of Construction Hazards.—Mechanical connectors, which the epoxy compounds would replace, are generally applied to the beams or stringers before the deck form work has begun. Hence, workmen must continually face the hazard of tripping over the connectors and falling. Some safety and ease of construction would be achieved by removing these connectors from the working surfaces.

4. Economy.—One of the most important and most common questions arising from the topic of an epoxy shear connector is that concerning the economy of its use. Aside from the economic factors previously discussed, and considering that successful application and use can be made with an epoxy compound, it is unlikely that any significant differences in initial construction costs would exist between the two joinery methods. Because deck cracking is presumably a major cause of deck deterioration, sometimes requiring deck replacement within a few years, this system may provide a major economic advantage. Costs of deck replacement on a typical bridge are approximately one-fourth of its original cost. Therefore, the designer could afford to make other concessions in the design and still have a major economic advantage. It is apparent from tests that such concessions would be minor, if necessary at all. These potential savings make the studies worthwhile.

APPLICATION OF EPOXY SHEAR CONNECTOR

Probably the greatest disadvantage of using an epoxy resin compound as a shear connector is the surface preparation required before applying these materials. All surfaces must be free from dust, dirt, rust, mill scale, latency in the case of concrete, and other surface contaminates. It is not difficult to achieve good clean surfaces, but it may be difficult to maintain this degree of cleanliness before application of the epoxy material.

In the case of the T-beams, all steel I-beams, wide flange, or built-up girder section surfaces must be sandblasted just before the pouring of the deck. After the steel surfaces are thoroughly cleaned by sandblasting, and after the excess blasting grit has been blown from the deck, the steel surface is cleaned with methyl-ethyl-ketone. This final cleaning must be accomplished with lint-, dust-, and grease-free rags or other suitable cleansing materials. The surface is then ready to receive the epoxy compound.

By a suitable applicator (which still needs to be developed) the epoxy compound, which has just been mixed, is applied to a depth of $\frac{1}{4}$ in. to the steel surface. The $\frac{1}{4}$-in. thickness was selected only after extensive tests showed this thickness to be a minimum requirement if moisture from the fresh concrete is not to penetrate the epoxy and cause rusting of the steel surfaces (5).

After the epoxy compound has been uniformly spread over the steel surface, and washed clean, angular aggregate is placed randomly over the epoxy compound layer. The aggregate should have a maximum size of $1\frac{1}{2}$ in. and a minimum size of 1 in. This material should be placed so as to offer approximately 20 percent coverage of the epoxy surface. No additional effort to force the aggregate into the epoxy layer is necessary after initial placement. The purpose of the aggregate is to increase composite action and to provide the necessary resistance to vertical separation of the slab from the stem. The epoxy compound aggregate surface is now ready to receive the fresh concrete.

The aggregate acts as more than a mechanical shear and vertical tension connector. It actually integrates the concrete with the bonding material so that the mortar matrix

in the regular concrete gradually changes from a cement mortar to an epoxy mortar, culminating in a surface in which strong bond exists between the steel and the epoxy concrete boundary. Thus, the aggregate probably never acts as a mechanical connector in the sense that a steel stud welded to the beam does.

In placing the concrete, the epoxy compound should be covered with concrete as soon as possible after it has been placed. Therefore, no more compound should be applied than can quickly be covered by a layer of concrete. The compound can receive a quick unfinished coverage of concrete; then the normal placement and finishing of the concrete can take place. The steel surface and the fresh concrete on the epoxy will sufficiently retard its curing. No more epoxy compound and concrete should be placed in advance of the normal pouring and finishing process than can be covered before the concrete has taken an initial set.

The epoxy adheres to concrete even though the wet concrete is applied to the wet epoxy compound. But water should not be allowed to penetrate the epoxy compound layer because it forms a film on the steel surface resulting in a rusted area and poor bond between the compound and steel. This water penetration is prevented by using at least ¼-in. thickness of epoxy compound.

EPOXY COMPOUND FORMULATIONS

The epoxy compound used for the shear connector is a definite, precise formulation of an epoxy, flexibilizer-hardener, hardening agent, and inert fillers. Extensive material and adhesive property tests of the formulation and variations of this formulation have been made. Many similar materials have been studied. The formulation described herein is the only one which can be recommended by the authors as suitable for this use, although some uninvestigated formulations may also be adequate. Formulation variation studies (6) have shown that considerable field errors in mixing appropriate proportional amounts can be tolerated without greatly affecting the material properties. However, improper mixing and blending of the ingredients cannot be allowed. For this reason the authors recommend a three-component compound rather than the usual two. To maintain conformity to the usual practices, the formulation is described first as the two-component system.

Two-Component System

Part A		Part B	
Constituents	Parts by wt	Constituents	Parts by wt
Epi-Rez 510	100	Epi-Cure 855	20
Alumina T-60	20	Epi-Cure 87	10
Asbestos 7-TF-1	20	Alumina T-60	20
		Asbestos 7-TF-1	20

Constants.

Wt-gal, resin portion (lb)	11.77
Wt 1 gal, converter portion (1b)	12.93
Wt 1 gal, total (lb)	12.16
Viscosity of total formula at 1 rpm, 77 F (cps)	84,000
Pot lift at 77 F, for a 1-pint batch (min)	35
Gel time in thin film at 77 f (hr)	2.0

Mixing Ratio. —The mixing ratios may be based on the parts by weight of epoxy as listed in the foregoing formulation, or if volume is preferred, the 1-quart resin portion is thoroughly mixed with 1 pint of the converter portion.

Liquid Portions. —Epi-Rez 510 is a standard epoxy resin composed of commercial diglycidal ether of bisphenol A containing no added diluents, solvents or other contaminants. Epi-Cure 855 is a flexibilizer-hardener, and is an aliphatic amide polyamine consisting of the reaction product of a long chain monobasic acid with polyamine. Epi-Cure 87 is a hardening agent, and it is a modified aliphatic polyamine.

Cautions. —The use of rubber gloves, goggles, and protective clothing is necessary where this formulation is handled and used. Areas of the body which come in contact with uncured resin and/or converters should be cleaned with denatured alcohol, then well scrubbed with soap and water.

Three-Component System

Part A		Part B		Part C	
Constituents	Part by wt	Constituents	Part by wt	Constituents	Part by wt
Epi-Rez 510	100	Epi-Cure 855	25	Alumina T-60	40
		Epi-Cure 87	10	Asbestos 7-TF-1	40

Parts A and B are thoroughly mixed by mechanical or manual means. After the mix is complete, Part C is added and thoroughly stirred into the mixture. By this method better chemical reaction is assured, particularly on days with temperatures ranging below the recommended minimum temperature of 60 to 70 F. This three-component system is recommended for use as a T-beam shear connector.

MATERIAL PROPERTIES OF EPOXY COMPOUND

As with any structural material, strength properties are a very important consideration in design. To determine the properties of the epoxy compound (Table 1), various tests were performed according to ASTM standards for plastics, and other tests were developed for determining the properties not covered by the standards. In any case, the methods used for determining the properties of epoxy resins need some revision, as duplication of results is not within close tolerances.

The results of some of these tests follow. The method used is not described in detail, but the proper reference is given for obtaining the test method.

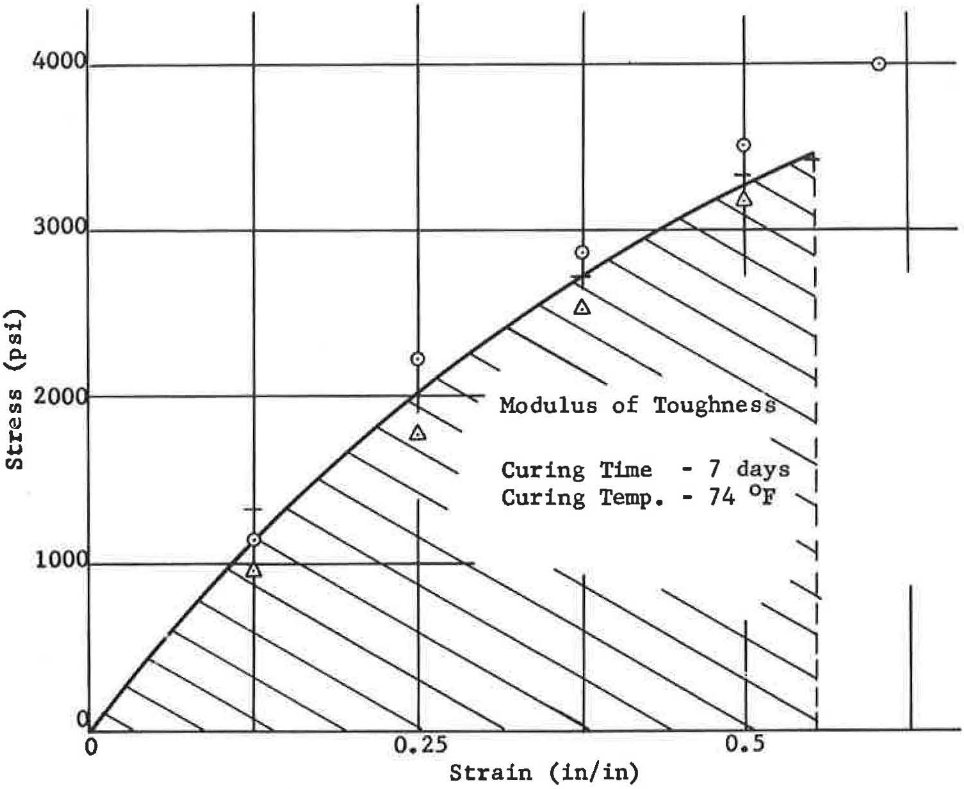
1. Material Tensile Strength. —The material tensile strength was obtained by using a test method similar to ASTM Standard D638-64T. Difficulties in preparing specimens occurred because of the high viscosity of the system being tested. Large air bubbles occurred in the specimens, resulting in stress concentrations and an ultimate strength lower than that obtained for the adhesive tensile strength. The specimens were cast in

TABLE 1

PROPERTIES OF EPOXY COMPOUND SHEAR CONNECTOR MATERIAL

Property	Range of Values[a]
Material tensile strength	3,000 to 4,000 psi
Modulus of elasticity, tension	400,000 to 700,000 psi
Modulus of toughness, tension	10.2 to 20.0 in.-lb/in.^{3}
Percent elongation	0.5 to 1.2 percent
Material compressive strength	7,000 to 8,000 psi
Proportional limit, compression	5,000 to 6,000 psi
Modulus of elasticity, compression 3	325,000 to 350,000 psi
Adhesive tensile strength	3,000 to 7,500 psi
Adhesive shear strength	1,200 to 2,400 psi
Coefficient of thermal expansion	27.3 × 10 to 32.0 × 10 in./in./deg F
Arizona composite cylinder rating	92 to 102 percent

[a]These values represent a minimum of eight test specimens.



3 Specimens	Ave.
Maximum Stress	3573 psi
Proportional Limit	---
Modulus of Elasticity	572,000 psi
Modulus of Toughness	11.75 in-lb/in^3
% Elongation	0.55%

Figure 1. Epoxy compound tensile strength test.

silicone rubber molds and machined to a smooth finish, with a cross-section $^5/_{16}$ in. thick and $^1/_2$ in. wide. Figure 1 shows the stress-strain curve for the material with the appropriate values listed as obtained from the curve.

2. Material Compressive Strength. —The material compressive strength was run similar to ASTM Standard 695-63T. The specimens, cast in silicon rubber molds were 1 in. in diameter and 1.312 in. high. Figure 2 shows the stress-strain curve with the material properties listed.

3. Adhesive Tensile Strength. —This test was run similar to ASTM Standard D897-49. Two solid steel cylinders with 1-sq in. cross-sectional areas were joined end to end with the epoxy compound. A special jig was used to align the specimens and assure a proper bond-line thickness. The specimens were tested by applying tension on the cylinders, thus placing the bond line in tension (Table 1).

4. Adhesive Shear Strength. —This test was run similar to ASTM Standard D1002-64. Steel strips, 5 in. long by 1 in. wide by $^5/_{16}$ in. thick, were overlapped $^1/_2$ in. and bonded together with a thin coat of epoxy compound. As was the case for the adhesive tensile strength, a special jig was made which assured proper bond-line thickness and proper

alignment of the specimens. The specimen was placed in longitudinal tension resulting in a shear failure at the overlap (Table 1).

5. Arizona Composite Cylinder Test.—This test method was developed in the civil engineering laboratories of the University of Arizona. The purpose of the test is to evaluate effectively materials which will bond wet concrete to hardened concrete or hardened concrete to hardened concrete.

The epoxy compound used as a shear connector must bond in the presence of wet concrete, and if the composite T-beam stem is to be of reinforced concrete, this test method is directly applicable. For the cases where a steel stem is utilized, the test still proves the feasibility of epoxy compound to bond in the presence of wet concrete.

To prepare specimens, half cylinders of concrete which had been cured for at least 30 days were cast in standard 6- by 12-in. cylinder molds in which an elliptical plate had been placed at a 30-deg angle to the vertical axis. The resulting slanted flat surface was treated with a 10 percent solution of hydrochloric acid to remove latent materials and was then thoroughly washed. The epoxy compound was spread on the surface, the cylinder replaced in the mold, and the remaining cavity filled. This resulted in a standard cylinder with an epoxy-compound bond line diagonally cutting it. The results are expressed as a percentage of the strength of a regular homogeneous cylinder cast at the same time. A strength of 90 percent was set as the minimum in which acceptable bond was achieved.

Although specific results are not presented here, this test, now widely used, is an excellent one to use for sorting the many epoxy compounds and selecting the most appropriate. The various compounds divide into two clear-cut groups, those which are clearly inadequate (with a rating of 30 to 40%) and those which have a rating of 90 percent and above (Table 1).

6. Coefficient of Thermal Expansion.—The coefficient of thermal expansion of the epoxy compound was measured by making electrical strain gage readings on a free epoxy specimen undergoing temperature changes. The strain measuring bridge was completed by a dummy gage on silica glass which has a negligible coefficient of thermal expansion (Table 1).

7. Creep, Fatigue and Aging.—Tests were also made to determine whether creep, fatigue and aging properties of the epoxy compound would adversely affect the life of the composite T-beams. The creep and fatigue studies were made on full scale T-beams by subjecting them to sustained loads and repeating loads, respectively. At normal working load conditions no measurable creep was noted for loads maintained for as long as one week. The epoxy-joined T-beams did not suffer from fatigue loading (8).

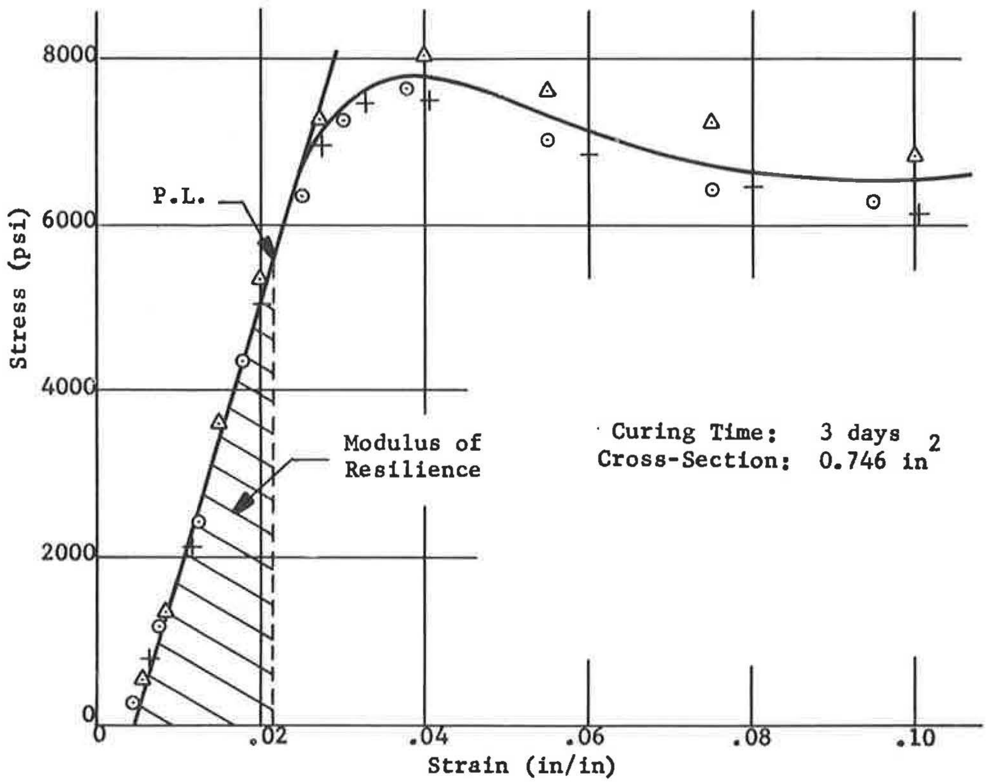
Studies of the epoxy are very complex and far from being conclusive. Nevertheless, those performed to date have shown that the particular epoxy system being used as a shear connector is quite stable with time.

DESIGN AND PROPERTIES OF T-BEAMS

Thirty-one T-beams, 26 of which used an epoxy compound shear connector, were tested statically. Test results of 10 of these beams are presented in this paper. Of these, 6 beams utilized the most successful of the epoxy compound shear connectors tried, and were prepared according to the aforementioned procedures. Four of the beams utilized an equivalent mechanical shear connection (studs) for comparison purposes. The T-beam dimensions and properties are given in Table 2.

All of the T-beams were "biased" in design to produce high shear stresses in the epoxy compound bond line. This was done by sizing the slab and steel beam so that the neutral axis occurred close to the shear connection, and also by selecting steel sections with a relatively narrow flange width. Both of these factors tended to maximize the shear stresses at the joint.

Property tests of many types of epoxy resin compounds have shown that one must be very selective to achieve good resistance to shear when bonding to steel. This is particularly true where impact loading is applied. The compound selected has sufficiently high tensile strength and meets the minimum requirements of shear strength. The steel-to-steel shear strength is approximately 2,200 psi with occasional specimens



3 Specimens **Ave.**

Maximum Stress	7740 psi
Proportional Limit	5570 psi
Modulus of Elasticity	340,000 psi
Modulus of Resilience	46.50 in-lb/in^3

Figure 2. Epoxy compound compression test.

TABLE 2
T-BEAM DIMENSIONS AND PROPERTIES[a]

T-Beam	Length (ft)	Steel Beams Size	Slab Dim. Size (in.)	Moment of Inertia, I	Modular Ratio, n	Yield Strength (psi)	f'_c (psi)	Neutral Axis, N. A. (in.)	Shear Span, a (ft)
A2-1	21	8I23	4 × 29	245	7.02	43,000	5,470	3.97	6.5
A4-1	29	15I50	4 × 40	1,327	6.73	38,400	5,580	5.63	10.0
C1-1	11	8WF17	3 × 24	160	8.10	46,000	4,670	3.48	3.0
C1-1S[b]	11	8WF17	3 × 24	170	6.35	46,100	4,470	3.18	3.0
C2-1	21	8I23	4 × 24	245	7.00	37,900	4,740	3.97	6.25
C2-1S	21	8I23	4 × 24	268	5.06	39,400	4,720	3.57	6.25
C3-1	21	12I50	4 × 40	849	8.64	39,300	4,230	5.52	6.25
C3-1S	21	12I50	4 × 40	902	7.00	35,900	4,690	5.11	6.25
C4-1	29	15I50	4 × 40	1,255	8.38	36,900	4,370	6.10	10.5
C4-1S	29	15I50	4 × 40	1,340	6.50	39,300	4,410	5.55	10.5

[a]I = moment of inertia of cross-section, computed from transformed section;
n = ratio of modulus of elasticities of steel and concrete;
f'_c = compressive strength of concrete at time tests;
N. A. = computed neutral axis of cross-section measured from top of beam; and
a = shear span, measured from support to location of load (two symmetrically placed jacks provided load).

[b]S following T-Beam identification indicates a mechanically connected T-beam; other T-beams employed epoxy shear connector.

failing as low as 1,240 psi. These strength values were also obtained using other epoxy compound systems. However, the selected system was the only one which showed sufficient resistance to impact loading.

The data for several preliminary T-beams are not presented here. These had varying bond-line thickness, epoxy compounds, and surface treatments used in the shear connection. In general, these beams failed at the shear joint due to poor bond between the steel and the epoxy compound. These were largely pilot tests to investigate various combinations of materials, and to study the general behavior of the beams. A2-1 and A4-1 were the first beams made which performed adequately at loads beyond the yield point of the steel. These beams used the recommended epoxy compound formulation with a 1/4-in. thick joint and the scattered aggregate treatment previously described.

There was considerable difference in the performance of the pilot beams, depending on the thickness of the shear joint. A special investigation was initiated to study the thickness effect, but the results were of qualitative value only. A round steel specimen was machined to a 4-in. diameter and a 1-in. thickness with one surface ground highly smooth. The plate was placed inside a standard 6- by 12-in. cylinder mold in such a way that a half cylinder (6 × 6) could be cast on top of it, and was coated to a uniform depth of either 1/16, 1/8, or 3/16 in. with epoxy compound. Either no aggregate was applied to the surface, or a 20 percent coverage of 3/8, 3/4, or 1 1/4-in. aggregate was applied. The plates were then pulled off in a testing machine, and the tensile strength of the joint recorded.

The 3/16-in. epoxy joint with the 1 1/4-in. aggregate coverage gave the best result. The failure was always by rupture of the concrete in this case at an average stress of over 4,000 psi for all joints with the 3/16-in. thickness regardless of the aggregate size used. The 1/16-in. coating gave results averaging about 2,500 psi. The thinner coatings showed water penetration, rusted areas on the steel plate, and the resulting poor bond. The aggregate size seemed to have a less pronounced effect. Figures 3 and 4 show the difference in types of failure for different thicknesses. As a result, a 1/4-in. coating and

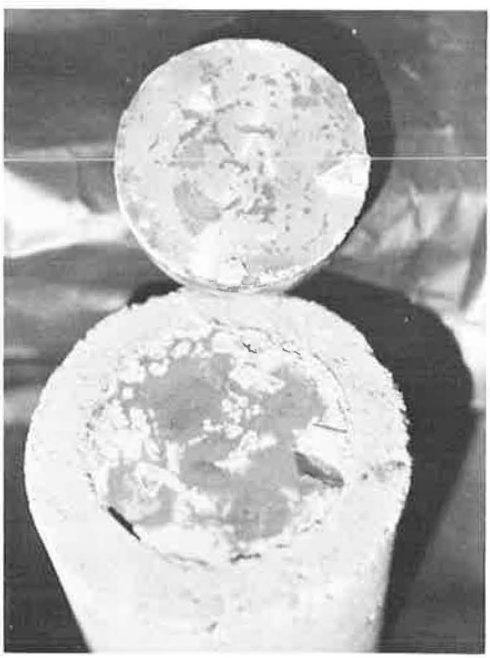


Figure 3. Concrete-to-steel tension test showing effect of moisture penetration through epoxy compound: rusting of steel, and areas of compound turned white by moisture.

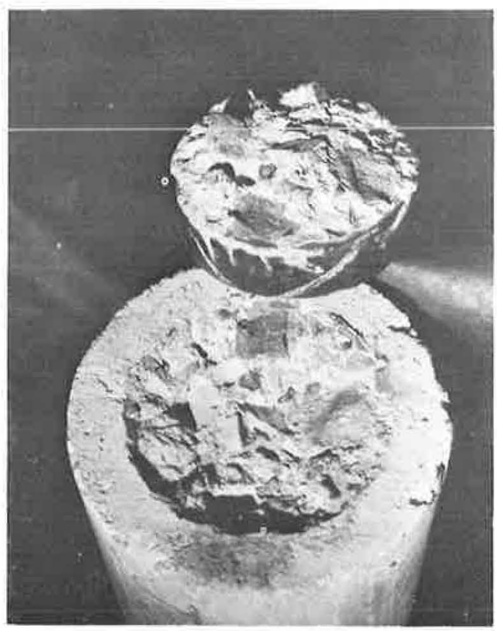


Figure 4. Concrete-to-steel tension test showing 3/16-in. bond line resulting in concrete failure, and unruptured white line between epoxy compound and concrete.

TABLE 3

T-BEAM TEST DATA

T-Beam Connector	Ultimate Load			Shearing Stresses			Maximum Bond-Line Slip (10^{-4} in.)	Type of Failure
	Computed Kips	Test Kips	Ratio: Test/Computed	Theoretical Yield (psi)	Theoretical Ultimate (psi)	Avg. Experimental Ultimate (psi)		
A2-1 Epoxy	21.8	22.9	1.05	390	770	925	35	Flexure-shear
A4-1 Epoxy	42.7	39.6	0.93	368	790	600	9	Flexure-shear
C1-1 Epoxy	37.0	35.3	0.95	494	1,215	1,160	36	Shear
C1-1S Mechanical (stud)	36.8	39.2	1.06	594	1,220	—	290	Flexure
C2-1 Epoxy	22.6	22.2	0.98	319	814	799	23	Flexure-shear
C2-1S Mechanical (stud)	23.3	23.5	1.01	452	842	—	570	Flexure
C3-1 Epoxy	61.2	46.9	0.77	657	1,393	1,068	8	Shear
C3-1S Mechanical (stud)	58.4	57.7	0.99	586	1,280	—	304	
C4-1 Epoxy	41.4	41.4	1.00	334	757	757	10	Flexure-shear
C4-1S Mechanical (stud)	43.6	41.4	0.95	366	807	—	290	Flexure

20 percent coverage with 1¼-in. aggregate were selected for the shear joint for the rest of the tests (C series). The T-beams of groups C1 and C3 were designed to induce shear stresses approaching the minimum adhesive shear value of 1,200 psi at ultimate load. The T-beam of groups C2 and C3 were designed to have shearing stresses in the neighborhood of one-half the former value. Variance in concrete and steel properties led to the values given under "theoretical ultimate shearing stresses" (Table 3). These values are, at best, only a conservative estimate, because there is no theory which is accurate for calculating them.

TESTING PROCEDURE

The T-beams were loaded by two hydraulic rams placed symmetrically about the midspan of the beams. A small load was first applied to assure complete initial settlement of the T-beam and the supports. The supports were rollers placed 6 in. from the

Figure 5. T-beam test setup.

ends of the beam. The shear span, a, the horizontal distance from the support to a loading jack, is given in Table 2. After removal of all but a small "holding" load, zero readings were made on all gages. All of the beams were instrumented to measure deflection and strain distribution at midspan and end slip of the slab in relation to the steel beam. Some beams were instrumented to measure bond-line slip along the length of the beam-strain distribution across the top and bottom of the slab, and to measure strains in the shear span.

The load was applied in increments, and data readings were taken after each increment. The loads were continually increased until complete failure of the specimen occurred. The test setup is shown in Figure 5.

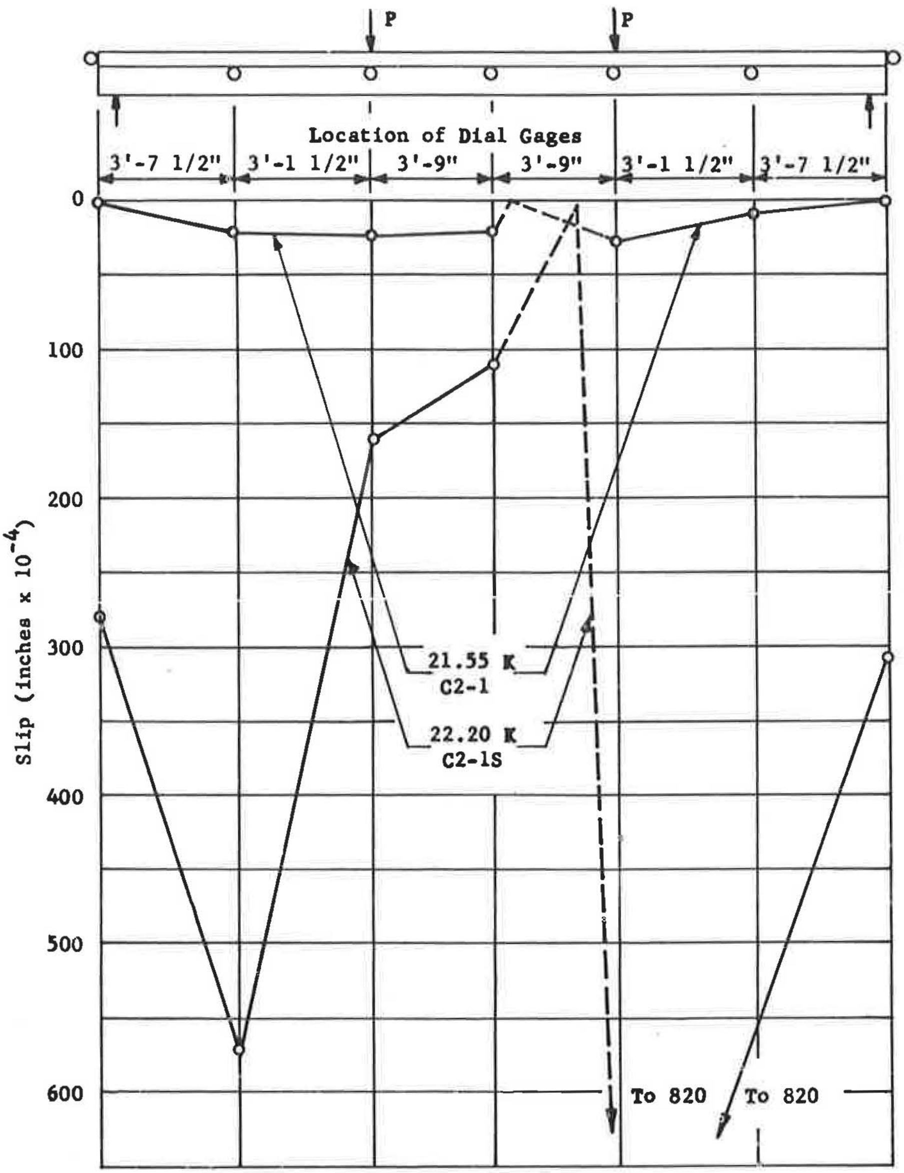


Figure 6. Comparison of load-slip curves at ultimate load for beams C2-1 and C2-1S.

TEST RESULTS AND CONCLUSIONS

The test results are given in Table 3, with data for each epoxy-compound joined T-beam followed by data for the equivalent mechanically connected T-beam. A direct comparison can be made of ultimate loads, shearing stresses, and bond-line slip. All of the T-beams behaved under load as expected. The epoxy T-beam with the highest ultimate shear stress, C3-1, failed at the lowest load. This is indicated by the 0.77 ultimate load ratio. This T-beam initially failed by shear of the epoxy from the steel throughout the shear span. The mechanically connected T-beam C3-1S behaved normally under loading, and the theoretical ultimate load was reached before complete failure occurred. This set of T-beams indicated that for high shear stresses, mechanically connected T-beams are best as far as developing the ultimate strength of the steel beam is concerned, at least for the "biased" design beams tested. It also demonstrated that shear stress is an important criterion when designing an epoxy-joined T-beam.

Epoxy-joined T-beam C1-1 had the second highest theoretical ultimate shear stress. The ratio of the ultimate loads for this T-beam was 0.95 which indicates excellent performance. The corresponding mechanically connected T-beam performed adequately as is indicated by the 1.06 load ratio. The T-beam C1-1 actually had a higher average experimental shear stress. Realizing that these values are only an estimate based on conventional theories and considering the range of values obtained by the bonding shear strength tests, it can be surmised that both T-beams C1-1 and C3-1 were in a critical load range. Complete failure could occur suddenly within a fairly wide range of load values.

Of the other two groups, C2 and C4, the epoxy-joined T-beams carried as great a portion of the theoretical ultimate load as was carried by the mechanically connected T-beams. The shearing stresses at ultimate load were well below the critical range. The two epoxy-joined T-beams of these groups failed by a combination of shear and flexure action. A longitudinal crack developed in the slabs of each of these T-beams near the time of failure. It is not known which occurred first, the crack or shear failure. This longitudinal crack is common when testing mechanically connected T-beams,

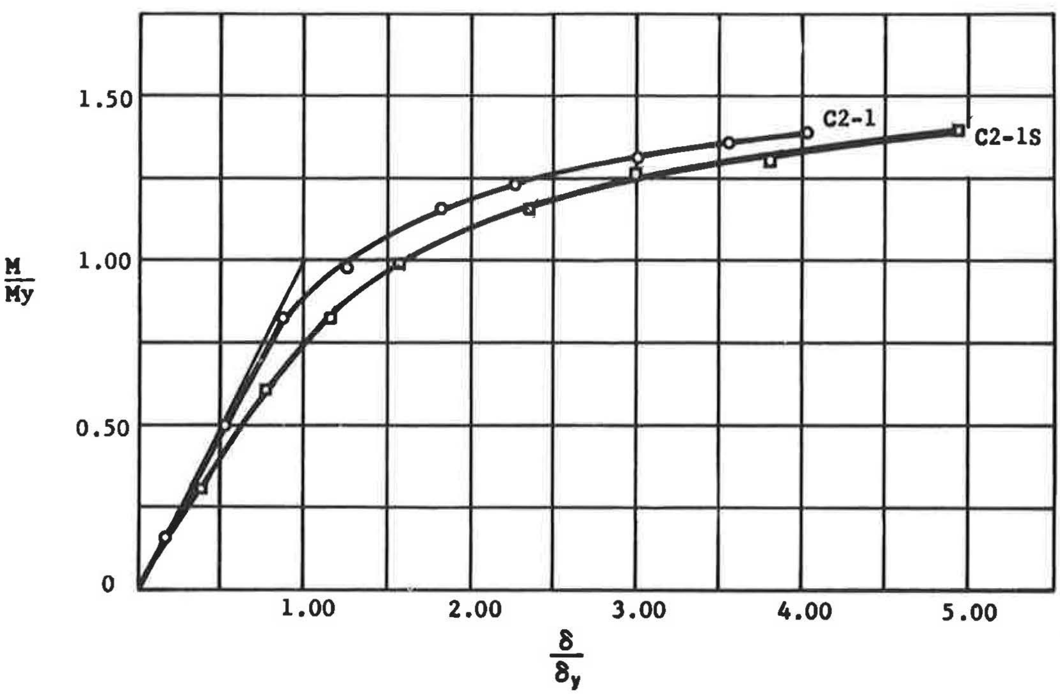


Figure 7. Comparison between deflection curves, beams C2-1 and C2-1S.

and is the first sign of failure of the concrete slab when ultimate load has been reached. In this case, however, studs do prevent the slab from coming completely free of the steel beam. The theoretical ultimate loads (based on the development of a complete plastic hinge in the steel) of the T-beams in the groups C2 and C4 were reached.

The effectiveness of the epoxy resin compound in developing more integral beam action than the mechanical connectors is probably best illustrated by the bond-line slip. All T-beams were instrumented so that the relative movement of the slab with respect to the stem could be measured. The maximum slip measured before failure in an epoxy T-beam was only 0.0036 in., whereas the highest slip for a mechanically connected T-beam was 0.0570 in. The slip in the mechanically connected T-beams was from 8 to 30 times as high as that in the epoxy-joined T-beams. A typical set of slip-load curves (beams C2-1 and C2-1S) is shown at ultimate load in Figure 6.

Figure 7 shows a typical dimensionless deflection curve comparison for beams C2-1 and C2-1S. The epoxy-joined beam is stiffer than the mechanically connected beam, and in every case followed the theoretical deflection curve in the elastic region, indicating more integral behavior between the beam and the slab. Also, the epoxy-joined T-beam always carried more load for a given deflection value. Thus the mechanically connected beam had more toughness, but the epoxy-connected beam acted more elastically.

A study of a wide variety of composite T-beam designs shows that shear stresses in the slab-beam joint seldom exceed 200 psi in normal applications. The ultimate experimental shearing stresses achieved in the test beams ranged between 600 and 1,160 psi. Thus the safety factor was no lower than 3 and ranged as high as 6. Inasmuch as these shear failures were closely associated with the yield of the steel and the development of a full plastic hinge, they would undoubtedly exceed these values in other cases.

Of prime importance is the ability of these T-beams to withstand fatigue and impact loading. As previously indicated, impact was one of the primary considerations in selecting the epoxy resin system. Eight composite T-beams were subjected to fatigue; six were epoxy-compound connected, and the other two were mechanically connected. They were of the same design as those previously mentioned. It is beyond the scope of this paper to report the complete dynamic testing and behavior of these beams. However, none failed under dynamic loading. Some of the epoxy-joined beams endured as many as 7,000,000 cycles without any significant change in bond-line slip or deflections. They underwent load cycling which varied from 200 to 250 cpm. The maximum and minimum load, expressed as shear stresses in the epoxy-compound joint, varied from 150 to 300 psi. The flexure stress in the steel ran as high as 41,500 psi during some of this testing.

CONCLUSION

An epoxy resin compound can serve as a reliable and safe shear connector for composite T-beams. The epoxy resin formulation and method of applying this formulation have been described. Test results indicate that very successful behavior of these epoxy-joined T-beams can be expected under long-term loading, either static or dynamic. The shear stress in the epoxy joint is a prime consideration when designing the T-beam. Economic considerations due to its minimizing shrinkage cracks and consequent bridge-deck deterioration may give it a great advantage over mechanical connections. It is not recommended that design specifications be written at this time. However, further work leading to design criteria should be carried out, and the contention that T-beams can be designed as monolithic units without regard to the shear connector when the recommended epoxy compound is used should be further substantiated.

REFERENCES

1. Adhesive Bonding. In Materials in Design Engineering, New York, Vol. 50, No. 3, Sept. 1959.
2. Further Studies of Epoxy Bonding Compounds. Physical Res. Proj. No. 13, Eng. Res. Series RR 63-5, New York State Dept. of Public Works, Dec. 1963.

3. Epoxy Bonded Aggregates as Shear Connectors in Composite Construction. Progress Rept., California Dept. of Public Works, Bridge Dept. June 1964.
4. Miklofsky, Brown, and Gousior. Epoxy Bonding Compounds as Shear Connectors in Composite T-Beams. Eng. Res. Series RR 62-2, Bureau of Physical Res., New York State Dept. of Public Works, 1962.
5. Kriegh, James D., and Endebrock, Elton G. The Use of Epoxy Resins in Reinforced Concrete—Static Load Tests, Part II. Final Rept. No. 2 to the Arizona Highway Dept., Eng. Res. Lab., Univ. of Arizona, Jan. 1963.
6. Kriegh, James D., Endebrock, Elton G., and Perry. A Study of the Adhesive and Material Properties of an Epoxy Resin Formulation as a Function of Proportioning the Ingredients, Part II. Final Rept No. 5 to the Arizona Highway Dept. Eng. Res. Lab., Univ. of Arizona, July 1963.
7. Nordby, Gene M., Kriegh, James D., and Endebrock, Elton G. The Use of Epoxy Resins in Reinforced Concrete—Static Load Tests, Part I. Final Rept No. 1 to the Arizona Highway Dept., Eng. Res. Lab., Univ. of Arizona, January 1962.
8. Kriegh, James D., and Endebrock, Elton G. The Use of Epoxy Resins in Reinforced Concrete—Dynamic Load Tests. Final Rept No. 3 to Arizona Highway Dept., Eng. Res. Lab., Univ. of Arizona, August 1963.

Damage Effects of Pile Driving Vibration

JOHN F. WISS, Wiss, Janney, Elstner and Assoc.

•PILE DRIVING, like dynamite blasts, nuclear blasts, and sonic booms, is a source of vibration which is frequently alleged to cause damage to structures. Unlike blasts, however, pile driving vibrations are produced by mechanical energy that is limited by the capabilities of the mechanical system. For example, a 5,000-lb ram falling freely from a height of 3 ft cannot deliver more than 15,000 ft-lb of energy on impact. Similarly, the maximum energy available from a double-acting steam hammer is limited by the steam pressure, the area of the piston, and the stroke.

On impact, the energy of the ram is imparted to the pile. It is distributed between rebound of the ram, elastic distortion of the pile, elastic and plastic deformation of the cushioning material, penetration of the pile, and elastic and plastic deformation of the earth surrounding the pile. The elastic deformation of the soil is propagated through the earth materials as elastic waves. The distribution of the available impact energy to the sources previously mentioned consists of interrelated functions, but the most important factor is the resistance of the soil to penetration by the pile. In a soft, easily penetrated soil, most of the energy is used in advancing the pile, and the least amount in the elastic deformation of the soil. In very hard, resistant soil the converse is true.

It is convenient to visualize the wave motion at the surface of the earth as being similar to the ripples produced on a smooth surface of water when a stone is thrown in. The wave length of the earth waves from pile driving is approximately 200 ft; this is the distance from the crest of one wave to the crest of the succeeding wave. Structures supported on the surface ride such waves in the same manner as a cork or box floating on the ripples of the water. Deeply embedded structures respond to a lesser degree in proportion to the orbital diameter of the earth particle motion which decreases exponentially with depth. For example, a structure embedded 200 ft below the surface would receive virtually no vibration. One at 100 ft would receive $\frac{1}{32}$th of the vibration experienced by a point on the surface. Regardless of depth, the magnitude of vibration intensity varies with the amount of energy transmitted to the soil, the physical properties of the soil, and the distance that the wave has traveled from the source.

Many instruments are capable of measuring the vibration intensities resulting from pile driving. Basically, such systems consist of a vibration sensor which converts the physical motion of the earth or structure into electrical signals. These in turn are sufficiently strengthened by an electronic amplifier to drive a galvaniometer and produce a recording of vibration vs time. It is essential to record the vibration, because the impulses are transient, and the response of meters is not fast enough to follow the vibrations accurately. It is also important to record simultaneously the vibratory motion in three mutually perpendicular directions. Although the impact force is generally in the vertical direction, the maximum earth or structural vibration is not necessarily vertical.

The instrument most commonly used for measurement of earth or structural vibration resulting from pile driving is the portable three-component seismograph. This unit is a mechanical optical system which utilizes seismic principles, is portable and battery operated, and produces a recording of displacement in three mutually perpendicular directions vs time. It is ideally suited for field recordings of the vibrations associated with pile driving.

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Figure 1. Sprengnether portable seismograph and equivalent complement of electronic equipment and transducers.

Figure 1 shows the Sprengnether portable seismograph which we have used extensively, and an equivalent complement of electronic equipment and transducers. Typical recordings of vibration from pile-driving operations are shown in Figure 2.

The damage potential of pile-driving vibrations depends on the displacement and the frequency of the vibration. Neither of these two characteristics alone will damage a structure. Concerning displacement, it is common knowledge that a structure can be uniformly jacked through several feet without causing damage. Likewise, with regard to frequency, normal sound, in passing through a wall, can vibrate the wall at high frequencies (several thousand cycles per second) without

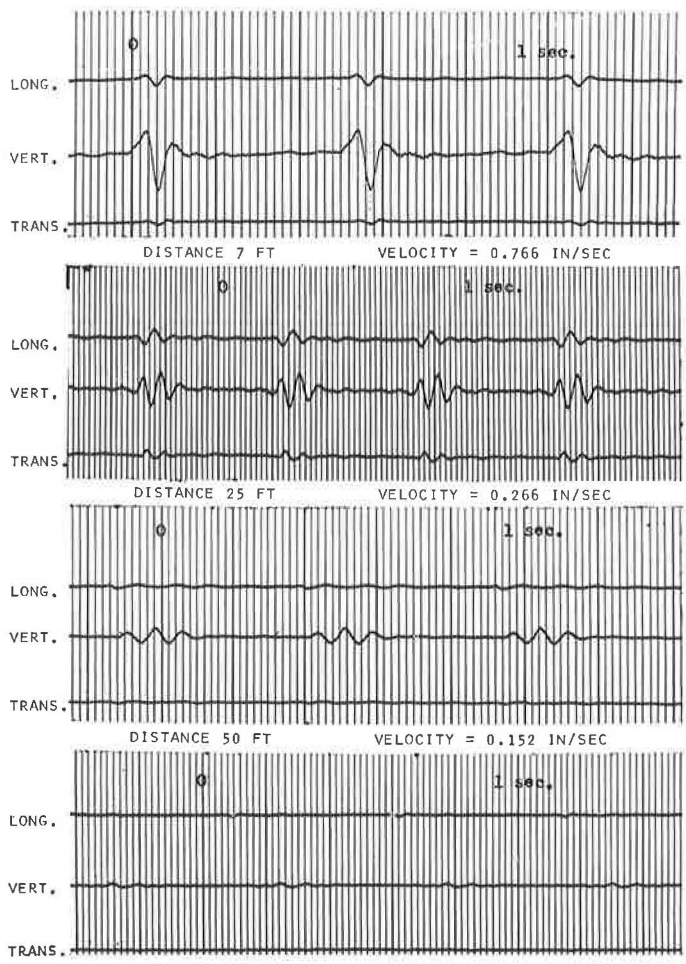
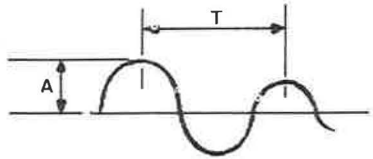


Figure 2. Typical earth vibrations from pile driving.



A = AMPLITUDE OR DISPLACEMENT (INCHES)

T = TIME (SECONDS) FOR ONE CYCLE

f = FREQUENCY (CPS) = 1/T

V = PARTICLE VELOCITY (INCHES PER SECOND)

$$V = 2\pi f A$$

TYPICAL SOIL FREQUENCIES

(PILE DRIVING)

ALLUVIAL FILL	5-10 CPS
CLAY	15-25 CPS
SAND	30-40 CPS

Figure 3. Particle velocity in alluvial fill, clay, and sand.

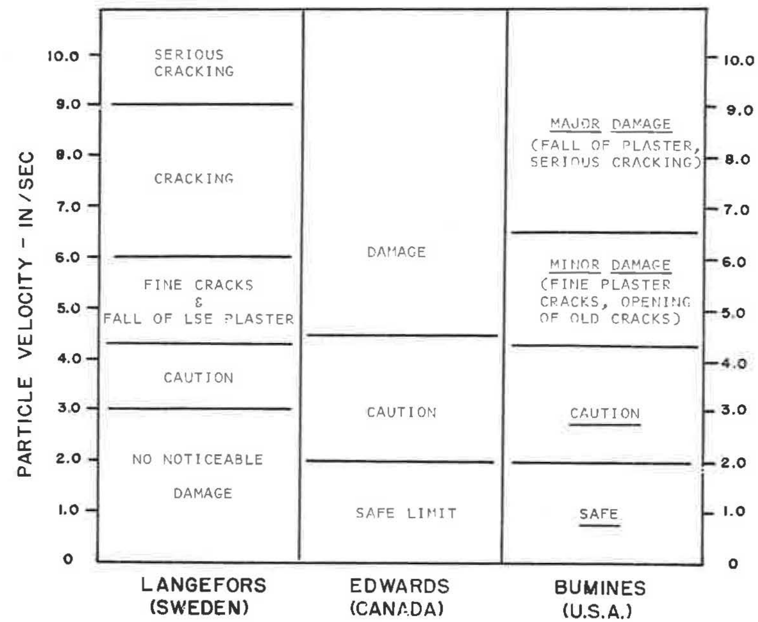


Figure 4. Comparison of damage criteria, residential-type structures.

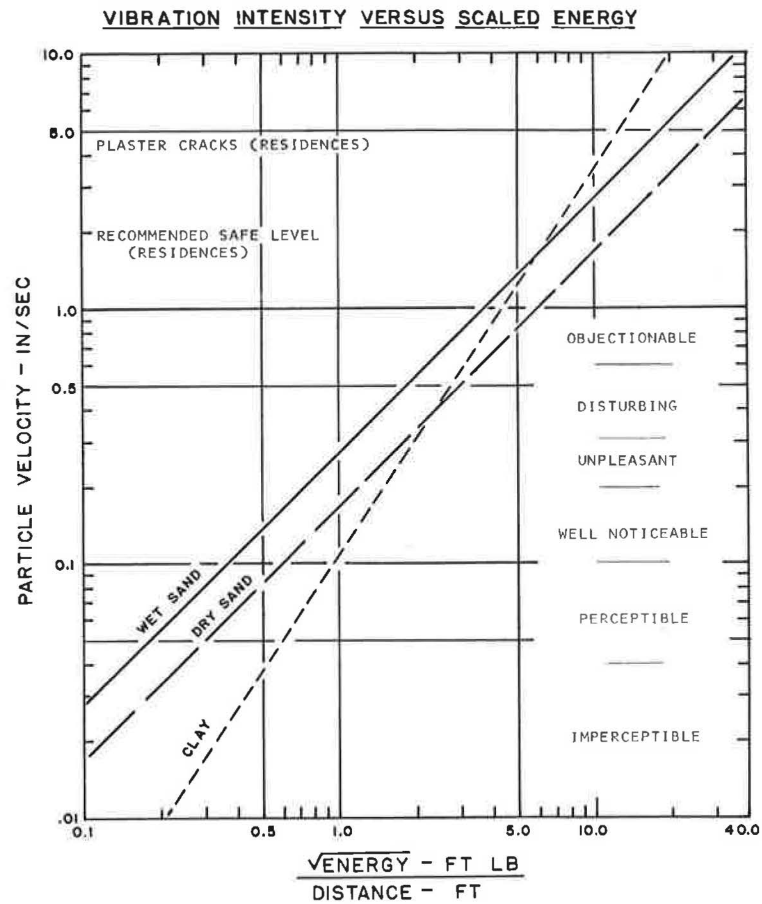


Figure 5. Maximum vibration intensities expected from pile driving on wet sand, dry sand, and clay.

causing damage. It is a combination of displacement (amount of motion) and frequency which causes damage. The particle velocity of earthborne vibration is the best measure of damage potential because it combines displacement and frequency in the most significant manner. Particle velocity (Fig. 3) can be expressed as $2\pi f A$, in which f is frequency (cps) and A is amplitude (displacement). Impact vibrations produced by pile driving have characteristic frequencies depending on the type of soil. A loose alluvial fill has natural frequencies of about 5 to 10 cps, clay soils vary between 15 and 25 cps, sand between 30 and 40 cps.

Several investigators in this country and abroad (including the U.S. Bureau of Mines) have found that particle velocities in excess of 4.0 in./sec are required to cause plaster cracks in dwellings. Figure 4 shows a comparison of the results of several of the investigations. With appropriate conservatism, the investigators agree that a vibration level of 2.0 in./sec (particle velocity) is safe with regard to plaster cracks in residential-type structures.

The effect of ground motion on an engineered structure can be computed by commonly used methods in the earthquake engineering field. The structure is considered a lumped mass-spring dashpot system, and its response to a series of impacts can be calculated. Based on observation and experience, it can be stated that ground motion particle velocities below 4.0 in./sec are well within the safe range for engineer structures.

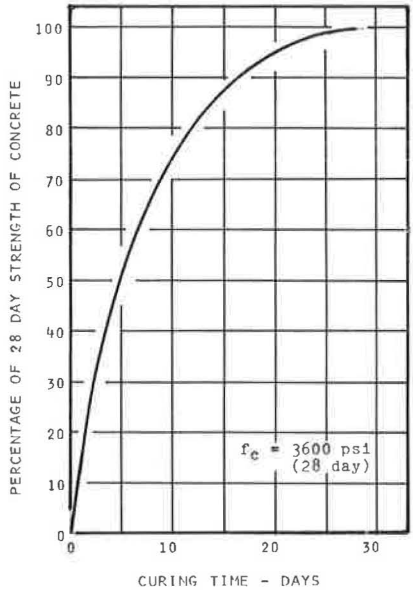


Figure 6. Strength of concrete vs curing time.

Figure 5 shows the maximum vibration intensities to be expected from pile driving in several soils on which extensive data have been obtained by the author. The data are plotted on log-log paper in which the abscissa is $\sqrt{E/D}$. This scaled energy factor permits use of the graphs with any size of pile driver; E is the foot-pounds of energy delivered by the hammer, and D is the seismic distance, in feet, from the pile tip to the location of interest. The vibration intensity (particle velocity) varies as the square root of the energy of the hammer. Figure 5 also indicates the levels at which vibration damage may be expected and the normal human evaluation of pile driving vibration. In several investigations, vibrations resulting from the driving of sheet piling, wood piles, and H piles were measured. For all practical purposes there is no difference in the vibration produced, all other variables being constant.

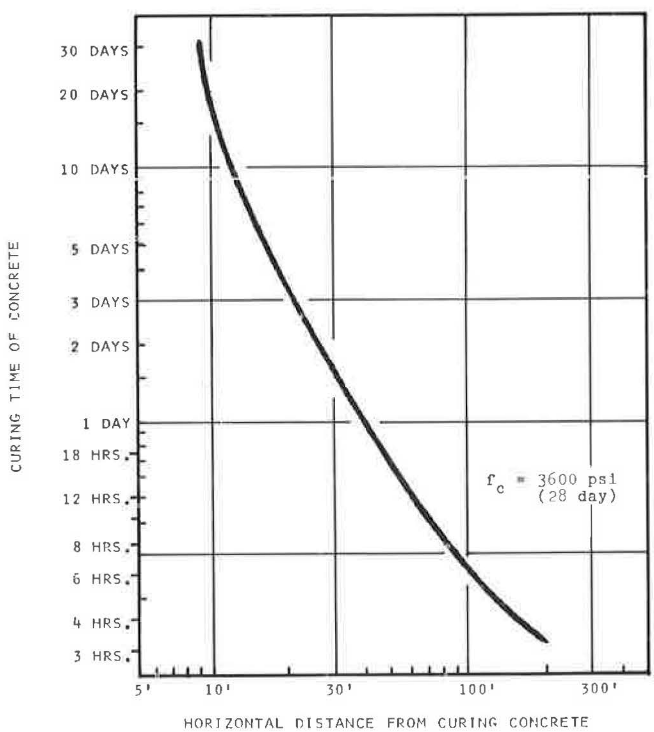


Figure 7. Limiting safe distance vs curing time of concrete for pile driver rated at 15,000 ft-lb of energy.

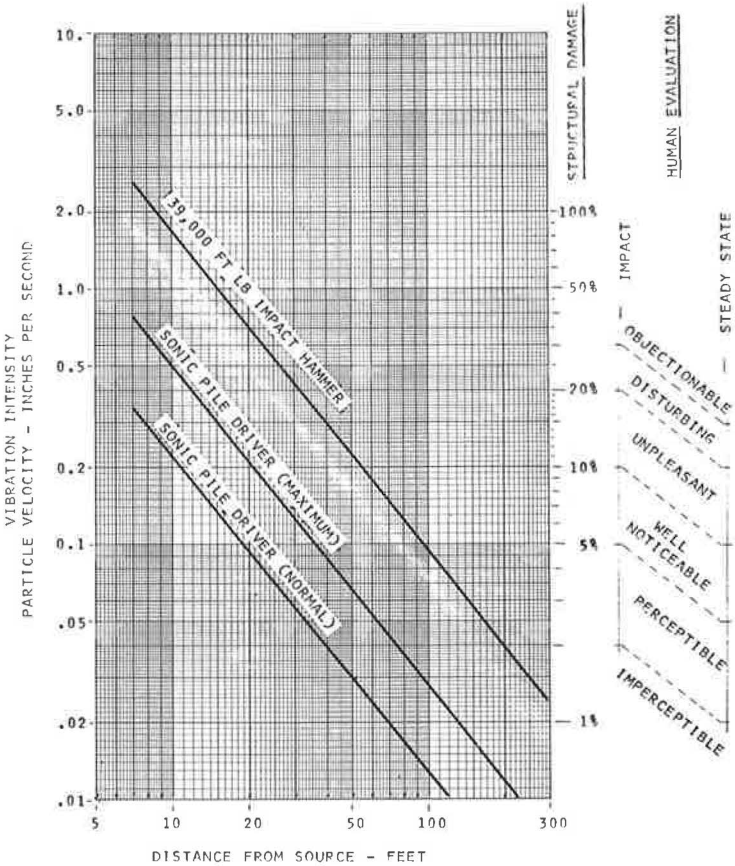


Figure 8. Vibration intensity vs distance.

Another problem of common interest on a construction project involves the case in which piles are to be driven at the same time that concrete is being placed. The question has frequently been raised as to whether the pile driving might have a detrimental effect on "green" concrete. Until more is known about concrete technology, it is doubtful that a rigorous analysis of such effects can be made.

As a practical matter, the following reasoning has been used by the author in the past, and although the magnitude of the safety factor has not been determined, there has been no evidence to indicate that the approach is not conservative.

Assuming that 5.0 in./sec (particle velocity) is conservatively a safe vibration level for cured concrete, and recognizing the rate at which green concrete attains strength, it is then possible to relate the permissible safe vibration level to the time since placing the concrete. When the decrease of vibration with distance has been evaluated for a particular site, the distance at which pile driving may be permitted (for a certain size hammer) can then be determined as a function of curing time.

Assume, for purposes of illustration, that a 3,600-psi concrete (28-day strength) has been specified on a particular project. The percentage of 28-day strength of concrete vs curing time is shown in Figure 6. This curve shows that the concrete has approximately 5 percent of its strength in 12 hr (one-half day), or 10 percent in 24 hr (one day). Thus, the vibration intensity should not exceed this same percentage of a particle velocity of 5.0 in./sec; limiting values of vibration are, therefore, 0.25 and 0.5 in./sec, respectively. If the soil is basically a clay, these vibration particle velocities correspond to an $\sqrt{E/D}$ of 2 and 3, respectively (Fig. 5). If a 15,000-ft-lb

pile driver is used, the closest permissible distances are 61.5 and 41.0 ft, respectively.

By the foregoing method a curve can be developed in which the limiting safe distance for pile driving vs curing time of concrete can be determined (Fig. 7). This curve is represented as a typical evaluation. For a specific site, pile driver, and concrete, limiting distances and curing time should be based on measured vibration intensities and concrete strength determinations, especially where short curing times are involved.

In closing, some brief comments on the vibration produced by the sonic pile driver and by other vibratory pile drivers are pertinent. In contrast to the impact hammer, which excites the soil at its natural frequency and the vibrations die out before the next blow, the sonic and vibratory pile drivers force the soil to vibrate at the continuous frequency (rpm) of the driver. These units can be speed controlled over a limited frequency range. Investigations of a sonic pile driver driven at frequencies between 90 and 120 cps, and a vibratory pile driver adjustable between 16 and 21 cps resulted in the following observations.

The normal vibration levels from the sonic driver may by one order of magnitude lower than those of an impact pile driver. However, the vibration varies continuously and occasionally attains intensities approximately one-half of the levels produced by a comparable impact hammer. Further, because the vibration is of a steady-state rather than a transient character, the human evaluation is usually more pronounced—by a factor of 2 (Fig. 8). The other vibratory pile driver investigated produced vibration levels of the same order of magnitude as a comparable impact pile driver.

With a steady-state excitation the possibility of resonance response in building components (especially panels) may become of significant importance. In the case of the transient vibrations produced by an impact pile driver, the duration of the transient is sufficiently short (0.2-0.3 sec) that a resonance buildup of structural components is not likely. The safe level of intensity for a steady-state vibration could conceivably be between one-half and one-fifth of the safe level for transient excitation; this is due to the possible magnification at resonance which depends primarily on the inherent damping characteristics of the structure.

Research on and Paving Practices for Wearing Surfaces on Orthotropic Steel Bridge Decks

F. F. FONDRIEST and M. J. SNYDER, Battelle Memorial Institute

Orthotropic steel deck bridge designs have experienced widespread growth in Europe and, because of certain inherent advantages, are gaining acceptance in this country. The success of orthotropic design is highly dependent on the performance of the wearing surface. Because little information on paving materials and methods was available in this country, a survey was made to fill this need. This report is a summary of the present status on wearing surfaces for orthotropic steel bridge decks in Europe and North America.

•THE use of metal plates on bridge decks dates from the 17th century. The modern developments in this area started to take shape around 1900 when composite docks were constructed of concrete and dished steel plates called buckle plates. The "battle deck" system was introduced during the 1930's whereby steel deck plates welded to longitudinal I-beams replaced the floors of old bridges to reduce the dead weight. The use of steel deck bridges was greatly advanced by integrally welded stiffening ribs in which the deck plate became an integral part of the main load-carrying members as well as the base for the wearing surface. This design system was primarily advanced in postwar Germany where a huge bridge rebuilding program was necessary in spite of an acute steel shortage. One of the principal advantages of this "orthotropic deck" design is a reduced steel requirement and a correspondingly lower dead weight. The advantages of orthotropic steel deck bridge design have been generally recognized and bridges of this type can now be found in more than 15 countries throughout the world.

Early experiences showed that the success of orthotropic bridges depends to a considerable extent on the wearing surface placed on the steel deck. The characteristics required of the surfacing or paving depend on the type and frequency of traffic and the life expected of the pavement. Increased vehicular loads and traffic volume in recent years have placed far greater demands on the paving materials, and correspondingly increased the problems of maintaining the pavements.

In general, the paving materials should have the following properties:

1. Lightweight. A chief advantage of an orthotropic bridge is its low dead weight. As the pavement does not contribute to the load-carrying capacity of the bridge, it is desirable to keep its weight to the minimum consistent with the other requirements.
2. Impervious. Since the steel deck is an integral part of the overall structure, it must be protected from corrosion. The paving materials must not permit water or moisture to penetrate to the steel plate.
3. Stable. The pavement must be stable enough to resist shoving and rutting under the pounding of moving traffic and to resist deformation under stationary loads at the highest temperatures encountered. It must be able to absorb without deformation the stress produced by braking and acceleration.

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4. Flexible. The pavement must be plastic enough to resist cracking under forces imposed on it by the plate deflections under wheel loads, or by differential contraction between the pavement and the steel deck, particularly at low temperatures.

5. Skid Resistant. The surface of the pavement must be acceptably skid resistant under any of the expected service conditions.

6. Durable. The pavement must maintain the desired properties throughout its expected lifetime. In addition, the paving materials must themselves be chemically resistant to gasoline and oil drippings from passing vehicles and to deicing agents, when used.

7. Smooth-Riding. The paving surface should be smooth and level for good riding qualities. A surface planeness tolerance of $\pm \frac{1}{8}$ in. in 12 ft is desirable for high-speed travel, but this tolerance may be increased for slower traffic speeds without impairing the riding qualities. The paving materials must be sufficiently thick to compensate for the deviations from a plane inherent in most orthotropic steel decks. The deviation from a plane of the steel deck may vary as much as $\pm \frac{1}{4}$ in. although normally it is considerably less. The pavement thickness may also be influenced by the protrusion of doubler plates and bold heads which are often used to field splice prefabricated deck sections. This problem can be eliminated by butt welding the deck sections.

Durability is probably the least well defined of the foregoing requirements. By European standards, a steel deck pavement is expected to remain serviceable for at least ten years with a minimum of maintenance.

In recent years, there has been considerable interest in orthotropic bridges in the United States. A major orthotropic bridge has already been completed in Canada and two more are presently under construction in the United States. A few small bridges have also been completed recently, and a number of other large and small bridges are being considered. Because the performance of the paving is critical and extensive experience in constructing pavements for steel decks is lacking in the U.S., a summary of the present knowledge on the subject is highly desirable.

A survey was made to assemble information on the present status of wearing surfaces on orthotropic steel bridge decks with reference to: (a) paving methods and materials used, (b) the performance of these materials, and (c) the status of research being conducted on the subject. This report gives the results of the survey. The information gathered was obtained from published literature, personal inspections, and contacts with people associated with this problem. The information is from both European and American sources, and because of some basic differences in the two, they are presented separately.

Much of the research being conducted is either incomplete or as yet unpublished; therefore, the results presented in this paper are only tentative and should not be taken as conclusive.

SUMMARY

European Practices

Three major types of steel deck paving systems are used in Europe: the layered system, the stabilized Mastix* system, and thin combination coatings. Only the first two systems have been used to any great extent.

Layered System. —The layered system is made up of two or more layers consisting of a deck prime coat, an Isolierung, a leveling course, and a surface course. A summary of the materials that have been used in this paving system follows:

*European asphalt paving practices and pavement formulations often differ substantially from U.S. practice and hence there may be no exact U.S. equivalent for a technical word. In these cases, the word is capitalized to distinguish it as a special term. A glossary of such terms and typical compositions is given in the Appendix.

Prime Coat	1.	Zinc metallizing
	2.	Lead-base paint
	3.	Bituminous-base materials
	4.	Epoxy coatings
	5.	Combinations of the above
Isolierung	1.	Aluminum foil
	2.	Copper foil
	3.	Asphalt Mastix
Leveling Course	1.	Gussasphalt
	2.	Asphalt binder
	3.	Stabilized Mastix
Surface Course	1.	Gussasphalt
	2.	Stone-filled mastic asphalt
	3.	Asphalt-concrete

Little difficulty has been experienced with the various deck prime coats and any of these materials may be expected to perform well when properly applied. Experience has shown, however, that a bituminous paint alone does not prevent moisture from reaching the deck plate if a crack penetrates down to it. When a metallized zinc coating is used, a very thin coat should be applied, preferably less than 1-mil thick; otherwise the bond strength of subsequent layers may be adversely affected.

Of the Isolierung materials, asphalt Mastix has proved to be the most effective. Earlier problems with the Mastix have been solved by decreasing the thickness of the Isolierung layer. Rubber powder is often added to help stabilize the Mastix when high bitumen contents are used. Metal foil Isolierungs have proved to be effective water barriers but are ineffective in absorbing the differential movement between the steel plate and the top two layers caused by temperature effects. The foils have also caused blistering problems. No evidence of the use of foil Isolierungs has been found subsequent to 1961.

The leveling course most often used was Gussasphalt, but an asphalt binder has been used for many recent installations. One of the latest pavements used a stabilized Mastix. When Gussasphalt is used, the binder is softer than that used in the wearing course so as to provide for greater flexibility.

Skid resistance on Gussasphalt wearing surfaces is provided by rolled-in stone chips and/or indentations formed in the surface.

The layered paving system has been in use for 15 years or more. Generally pavements of this type have performed quite well although some early problems were encountered. Such defects as shoving, blistering, rutting, and cracking have been noted, with the latter two being the most common. Cracking has been attributed to both severe temperature gradients and to plate deflections. Improvements in the Isolierung and in the general workmanship have alleviated many of the early problems, but some defects still persist.

Numerous inspections of existing installations have shown that corrosion of the steel deck plate has not been a problem. The deck prime coat coupled with the Isolierung has been effective in preventing water from reaching the plate, even when cracks occur in the top paving layers.

Stabilized Mastix System. —The stabilized Mastix system, which is patented in Germany, consists of a prime coat and a layer of Mastix which is choked with rolled-in crushed rock.

A $^3/_4$-inch layer of asphalt Mastix is placed over the prime coat and crushed rock, $^3/_4$ to 1 in. in size, is rolled into the Mastix at a rate of 130 to 160 lb/yd^2 while the Mastix is still soft. The resultant pavement is about 2 in. thick. In some of the earlier installations, closely spaced steel anchor bars, approximately $^1/_4$ by $^3/_4$-in. cross-section, were welded vertically to the deck plate in a zig-zag pattern to stabilize the pavement and prevent shoving. Experience showed that the bars could be smaller and spaced farther apart and in many cases completely omitted.

After some difficulty with the first installation, no further problems have been reported. This paving system is generally acknowledged to be the best in Germany but has been less used because of the higher cost.

Thin Combination Coatings.—The use of thin, single-coat paving systems, such as filled epoxies and cement-latex mixtures, has been limited. The results to date have been encouraging but there is not yet enough service experience to provide reliable evidence concerning their permanence and durability.

North American Practices

The wearing surfaces applied to orthotropic steel deck bridges have been about equally divided between a layered system, using an epoxy prime coat and one or two layers of either sand asphalt or asphaltic-concrete, and a combination system using a single layer of a sand/resin mixture. Many of the asphaltic materials used have been modified by additions of latex, rubber powder, or asbestos. All but one of the steel deck bridges discussed may be classified as an experimental installation.

The performance of the layered asphaltic materials has been mixed based on observations over a short period of time. No assessment of the sand/resin materials can yet be made as most of these installations have been in service for only a few months.

A number of research programs are being carried out in conjunction with the field installations to evaluate various prime coat and paving materials. Such properties as bond strength, impact resistance, fatigue strength and shear strength are being evaluated. Many of the results are still tentative, but the final results should be useful in properly assessing the materials under field use.

EUROPEAN SURFACING SYSTEMS

The various materials and methods used for steel deck pavements can be conveniently classified into three categories: layered systems, stabilized Mastix, and combination systems. These classifications are not necessarily absolute, but by pointing out the principal variations, they may be used to cover practically all of the paving systems in use.

Layered Systems

The layered system is made up of two or more layers of materials or compositions, each having a specific purpose, and the whole making up the deck surfacing. This system is most often composed of four layers: (a) the steel deck-protective or prime coat, (b) the insulation layer (Isolierung), (c) the leveling course, and (d) the surface course (1, 6). Figure 1 shows a cross-section of a typical layered paving system. Table 1 gives some details of a number of European bridges paved with the layered system.

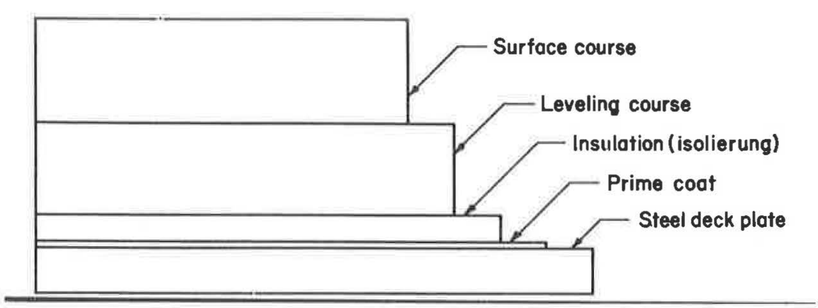

Figure 1. Cross-section showing typical makeup of layered paving system.

TABLE 1

DETAILS OF EUROPEAN BRIDGES SURFACED BY LAYERED SYSTEM

Bridge and Location	Date in Service	Structural Details: Type	Structural Details: Deck Plate Thickness[a] (in.)	Structural Details: Type Ribs and Spacing (in.)	Surfacing Details: Prime Coat	Surfacing Details: Isolierung (in.)	Surfacing Details: Leveling Course (in.)	Surfacing Details: Surface Course (in.)	Remarks	References
Spatzen-Mannheim	1948	Swing	0.47	Split I's at 17.7	Bit. varnish	None	1 binder	1 gussasphalt		(5)
Kurpfalz-Mannheim	1950	Girder	0.47	Angles at 11.8	Bit. varnish	Alum. foil[b]	1 3/16 gussasphalt	1 gussasphalt		(5, 7)
Koln-Mulheim	1951	Suspension	0.47	Bulb T's at 11.8	Bit. varnish	5/16 mastix	1 gussasphalt	1 gussasphalt	East 2/3; Isolierung = 22% bitumen, paper separtor over Isolierung	(8, 9, 10, 13, 14)
						5/16 mastix	1 gussasphalt	1 gussasphalt	West 1/3; Isolierung = 16% bitumen	
						5/16 mastix	1 gussasphalt	1 gussasphalt	East 2/3 replaced; Isolierung = 16% bitumen	
Werratal-Hedemunden	1952	Girder	0.47	Bulb T's at 11.8	Bit. varnish	Alum. foil	13/16 gussasphalt	13/16 gussasphalt		(2, 11, 12)
	1962				Okta-Haftmasse	5/16 mastix	1 gussasphalt	1 gussasphalt	Completely resurfaced in 1956	(2)
Main-Eddersheim	1953	Girder	0.55	Flats at 13.8	Bit. varnish	3/8 mastix	13/16 gussasphalt	13/16 gussasphalt		
					Bit. varnish	3/8 mastix	13/16 gussasphalt		5% rubber powder added to surface course	(2, 11, 14)
					Bit. varnish	Alum. foil	1 3/16 gussasphalt	13/16 gussasphalt	5% rubber powder added to surface course; paper separator over Isolierung	
					Bit. varnish	Alum. foil	1 3/16 gussasphalt	13/16 gussasphalt		
					Bit. varnish	Alum. foil	13/16 gussasphalt	1 3/16 gussasphalt	5% rubber powder added to surface course; paper separator over Isolierung	
					Bit. varnish	Alum. foil	13/16 gussasphalt	1 3/16 gussasphalt	1% asbestos added to surface course; paper separator over Isolierung	
Duisburg-Homburg	1954	Suspension	0.55	Pans[c] at 23.6	Okta-Hastmasse	5/16 mastix	1 gussasphalt	1 gussasphalt		(11, 13)
Weser Porta-Minden	1954	Girder	0.47	U Pans at 23.6	Bit. varnish	None	5/8 gussasphalt	5/8 gussasphalt	Paper separator over leveling course	(13, 15)
St. Albans-Basel	1955	Girder	0.47	Flats at 11.8	Lead paint	Bit.-latex	1 3/8 gussasphalt	3/8 asphalt powder	Stone embedded in the powdered asphalt; paper separator blow leveling course	(13, 16, 19)
Save-Belgrad, Jugoslavia	1956	Girder	0.39	Flats at 11.8	Bit. varnish	Alum. foil	7/8 gussasphalt	1 gussasphalt		(13, 17
1000 Window House-Duisburg	1958	Girder	0.47	—	Okta-Haftmasse	3/16 mastix	1 3/16 binder	1 gussasphalt		(14)
Rhine-Speyer	1958	Girder	0.47	Bulb T's at 11.8	—	Copper foil	1 gussasphalt	1 gussasphalt		(15, 18)
Rhine-Rhenen	1958	Girder	—	Bulb T's at 11.8	Bit.-latex	3/8 mastix	7/8 asphalt mastix	13/16 asphalt matrix		(15)
Mannheim-Ludwigshafen	1959	Girder	0.47	Pans[c] at 23.6	Bit. varnish	5/16 mastix	1 3/16 binder	1 gussasphalt		(5)
Koln-Severin	1959	Suspension	0.39	Flats at 11.8	Bit. varnish	Alum. foil	1 gussasphalt	1 gussasphalt		(2, 11)
Haseltal	1961	Girder	0.47	V Pans at 23.6	Bit. varnish	Alum. foil	13/16 gussasphalt	1 gussasphalt		(14, 11, 2)
Mainz-Wisenaver	1961	Girder	0.47	Flats at 11.8	Okta-Haftmasse	Alum. foil	1 gussasphalt	1 gussasphalt	Steel deck first zinc metalized	(20)
Norderelbe-Hamburg	1962	Suspension	0.47	Bulb T's at 13.8	Okta-Haftmasse	None	1 3/16 gussasphalt	1 3/16 gussasphalt	Steel deck first zinc metallized	(21, 22, 23)
					Epoxy	None	1 3/16 gussasphalt	1 3/16 gussasphalt		
Fehmarnsund	1964	Suspension	0.47	—	Okta-Haftmasse	3/16 mastix	1 binder	1 3/16 gussasphalt	Rubber powder in Isolierung and surface course	(2, 24)
Forth-Scotland	1964	Suspension	0.50	Pans at 27	Bit. varnish	3/16 mastix	1 3/8 stone-filled mastix	1 3/8 stone-filled mastix	Rubber powder in Isolierung	(15, 25)
Gablenz-Kiel	1956		—	—	1/32 mastix	—	11/16 gussasphalt	11/16 gussasphalt		(39)
Schnettker-Dortmund	1957	Truss	0.47	I-beams	Bit. varnish	5/16 mastix	1 gussaspahlt	1 gussasphalt	Ribs transverse to traffic lanes	(11, 39, 40)
Reuter-Bonn	1963	Girder	—	—	Bit. varnish	5/16 mastix	1 gussasphalt	1 gussasphalt	South lane only-north lane unknown	(2, 41)
Berliner Duisburg	1963	Girder	0.47	Pans at 22.9	Bit. varnish	3/8 mastix	1 asphalt concrete	1 3/8 gussasphalt		(42)
Sinzing-Regensburg	1965	Girder	—	Angles at 11.8	Okta-Haftmasse	—	2 stablized mastix	1 asphalt fine concrete	Leveling course applied in two layers	(4, 43, 21)
Herren-Lubeck	1964	Bascule	0.47	—	Bit. varnish	—	1 gussasphalt	1 1/8 gussasphalt		(44)
Rhine-Leverkusen	1965	Girder	—	—	Bit. varnish	5/16 mastix	1 asphalt concrete	1 gussasphalt		(42)

[a]Deck-plate thickness given is the minimum thickness used, and in most cases is used over the greater portion of the bridge deck.

[b]Through not indicated, the metal-foil isolierungs are placed with a bituminous cement.

[c]Trapezoidal shaped stringers.

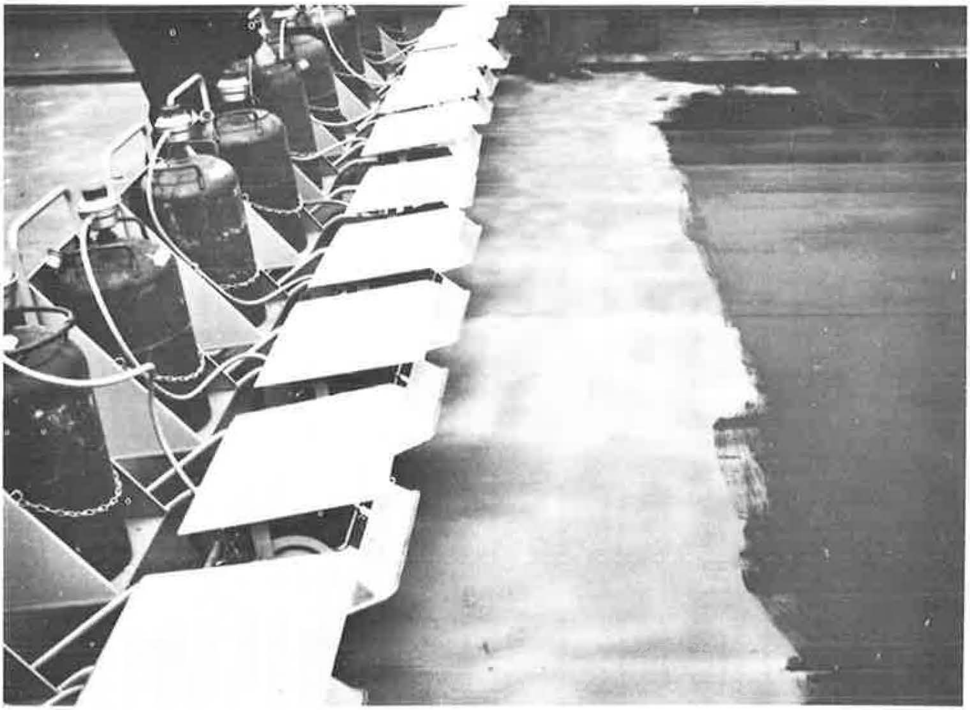

Figure 2. Infrared heating of steel deck plate in front of bituminous prime coat application.

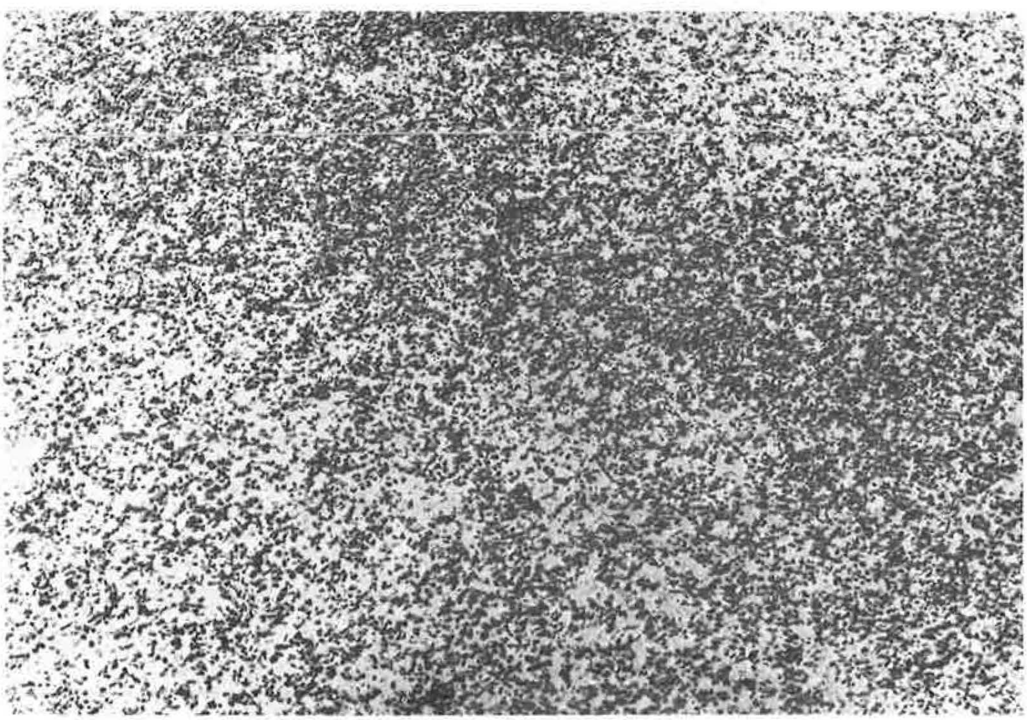

Figure 3. Surface appearance of epoxy prime coat with sprinkled-in stone chips.

Prime Coating.—The prime coating is a thin organic or inorganic coating applied to the steel plate to prevent corrosion and to act as a bonding material for the subsequent layers. In many early installations, the steel deck had been zinc metallized as an extra precaution against corrosion before applying the prime coat. Experience has shown that metallizing is not necessary and can cause some problems. The practice has generally been discontinued.

Several materials have been used for the prime coat, such as bituminous varnish (both hot and cold applied), lead-base paint and epoxies. Most of the bituminous coatings used in Europe are proprietary materials generally made up of a high-penetration bitumen containing a variety of additions to improve the characteristics so as to fulfill special requirements. In most cases, earlier bridge decks were painted with a bituminous varnish. One of the most successful proprietary coatings used in Germany is Okta-Haffmasse (Teerbau, Inc., of Essen). This material is believed to be a low-penetration bitumen containing rubber powder and a long-chain hydrocarbon additive (2, 3). It is flame-sprayed onto the steel deck to promote wetting of the deck by the coating (4).

The prime coatings may be applied by hand painting or brushing onto the steel deck immediately after sandblasting. In all cases, the steel plate is sandblasted before the prime coating is applied and one of the more recent innovations is to heat the steel plate with infrared heaters just ahead of the application of the prime coat to drive off any moisture and assist wetting (Fig. 2).

Where epoxy resins are used for the prime coat, stone chips are broadcast over the surface before the resin sets to act as mechanical anchors for the subsequent layers (Fig. 3). No Isolierung is used when an epoxy prime coat is used as the epoxy alone will provide all the deck protection necessary.

Isolierung.—The Isolierung, or insulation layer, is placed directly over the prime coat and is thoroughly bonded to it. The purposes of the Isolierung are to (a) provide an impermeable barrier between the prime coat and the surfacing layers in the event cracks occur in the top layers, (b) act as a flexible layer or plane between the steel plate and the surface courses to absorb the differential movement caused by sudden changes in temperature, and (c) serve as a heat shield for any heat-sensitive prime coatings when the leveling course is applied.

The Isolierung is generally made up of either a thin sheet of metal foil (aluminum or copper) cemented to the deck coating or a ⅛- to ⅜-in. layer of Mastix.

Aluminum foil has been used on a number of bridges in Germany and on the Save River Bridge in Belgrade, Yugoslavia. The foil is usually about 8 mils thick and may have either a smooth or dimpled surface. Dimpled foil is preferred as the dimpling is believed to improve the bond strength. The foil sheets come in widths of about 2 ft and are overlapped 2 to 4 in. when cemented down. The sheets are rolled into a hot bituminous cement, with great care taken to avoid entrapping air under the sheets. In the last few years the trend has been away from using metal foil for the Isolierung. Because this operation is performed entirely by hand, the quality of the job depends on the workmanship. Air voids cannot always be avoided, and the entrapped air has caused blistering of the surface courses, and promotes further destruction of bond. The metal foil is also somewhat ineffective in absorbing the relative movement between the steel plate and the surface courses caused by temperature changes. High shear stresses are produced by the differences in the thermal properties of steel and asphaltic compositions.

Mastix Isolierung was used as early as 1951 and has gained in acceptance since then. The Mastix is made of fine sand, filler material, and bitumen. The bitumen content varies from 14 to 25 percent by weight with the higher contents associated with thinner layers. With the higher bitumen contents, natural rubber powder is often incorporated in amounts up to 5 percent by weight of the bitumen content. The Mastix composition results in a voidless and impermeable layer that gives added corrosion protection to the steel deck.

The cooking temperature of Mastix may be as high as 425 F and mixing times may be several hours. The ingredients are usually plant mixed for about 30 seconds and then transported to the job sites in portable mixers; the mixing times, depending on

Figure 4. Placing and hand spreading Mastix Isolierung.

Figure 5. Method of placing and spreading Gussasphalt by hand.

the type of equipment and the conditions at the site, may vary from 1 hour to over 8 hours. The composition is extremely fluid when placed and is hand troweled or squeezed in place (Fig. 4). Because of the thin layers involved, the temperature of the fresh mix drops rapidly and does not destroy or burn the prime coat.

Leveling Course.—The leveling course, placed directly over the Isolierung, varies in thickness from $\frac{3}{4}$ to $1\frac{1}{4}$ in. and may be either an asphalt binder or Gussasphalt (see Appendix). One of the most recent paving installations used a stabilized Mastix as a leveling course.

Gussasphalt, which may be considered a particular variety of asphalt Mastix, is a dense, hard and impermeable composition of aggregate, sand, filler and bitumen. Because of the fluidity when cast, Gussasphalt is not readily amenable to machine casting and is quite often laid by hand (Fig. 5). Like Mastix, Gussasphalt is plant mixed and continuously agitated while being transported to the job site in special cookers.

The asphalt binder is a coarse, open-grained composition similar to the asphaltic-concrete used in this country. It contains 4 to 8 percent voids and is permeable. The mixing time and temperature are similar to those used for asphaltic-concrete in the United States; it is machine laid and compacted by rollers.

Before 1959, the leveling course used in the layered surfacing system was almost exclusively Gussasphalt, but the use of asphalt binder has since increased for two reasons: (a) the binder can be more readily laid by machine, thereby reducing the cost; and (b) the open gradation of the binder supposedly results in greater flexibility. Because asphalt binder is machine laid and compacted by rollers, it cannot be placed on a metal-foil Isolierung because of the possibility of the aggregate punching a hole in the foil. The use of binder has paralleled the increase in the use of a Mastix Isolierung.

Surface Course.—The surface course is a stable, skid-resistant layer placed directly on top of the leveling course and forms the wearing pavement. The surface course is almost always Gussasphalt varying in thickness from $\frac{3}{4}$ to $1\frac{1}{4}$ in. The composition of the Gussasphalt surface course is generally the same as that used in the leveling course except that it contains a slightly harder bitumen.

The Gussasphalt in the surface course may be hand laid or placed and spread with a self-propelled screed. In the as-laid condition, it produces a hard but smooth glassy finish. To provide skid resistance to the surface, crushed rock ranging in size from $\frac{5}{16}$ to $\frac{1}{2}$ in. is broadcast over the freshly laid surface at a rate of approximately 30 $\mathrm{lb/yd^2}$ and rolled into the Gussasphalt surface before it hardens. The crushed rock is lightly coated with 1 to 2 percent of bitumen to facilitate adhesion. The rolling operation is performed by a roller having approximately $\frac{1}{2}$ in. square protrusions which, in addition to depressing the rock into the surface, also produces a dimpled surface. The roller may be pulled by hand or pulled behind the self-propelled screed.

A paving operation involving Gussasphalt, which is still sometimes placed by hand, has been fairly well mechanized by the use of a paving train. The Gussasphalt is plant mixed and transported to the job site in portable cookers where it is poured and spread in front of a self-propelled screed. The adjustable screed, which is heated with propane, levels and smooths the Gussasphalt. Spaced behind the screed is a full-width trough filled with the crushed rock. The rock is broadcast over the freshly smoothed Gussasphalt by workmen who ride on the trough. The roller which produces the finished surface is pulled behind the rock trough. The entire train runs on steel rails placed ahead of the paving and is adjustable up to 37.5 ft in width. The sequences of this operation are shown in Figures 6 through 8. The finished surface has excellent skid resistance (Fig. 9).

Another type of surface course consists of a thin layer of powdered asphalt into which is embedded lightly bitumened crushed stone. This type of pavement has a rough surface texture with excellent skid resistance. Only one example was found of the use of this type of surface course (16).

In the British version of the layered system, the leveling course and wearing course are combined into one $1\frac{3}{8}$-in. layer of stone-filled Mastic asphalt. A premixed Mastic, which has been solidified in chunks of approximately 50 lb each, is transported to the job site and remelted in agitating mixers. While remelting, $\frac{1}{4}$ to $\frac{3}{8}$-inch stone chips

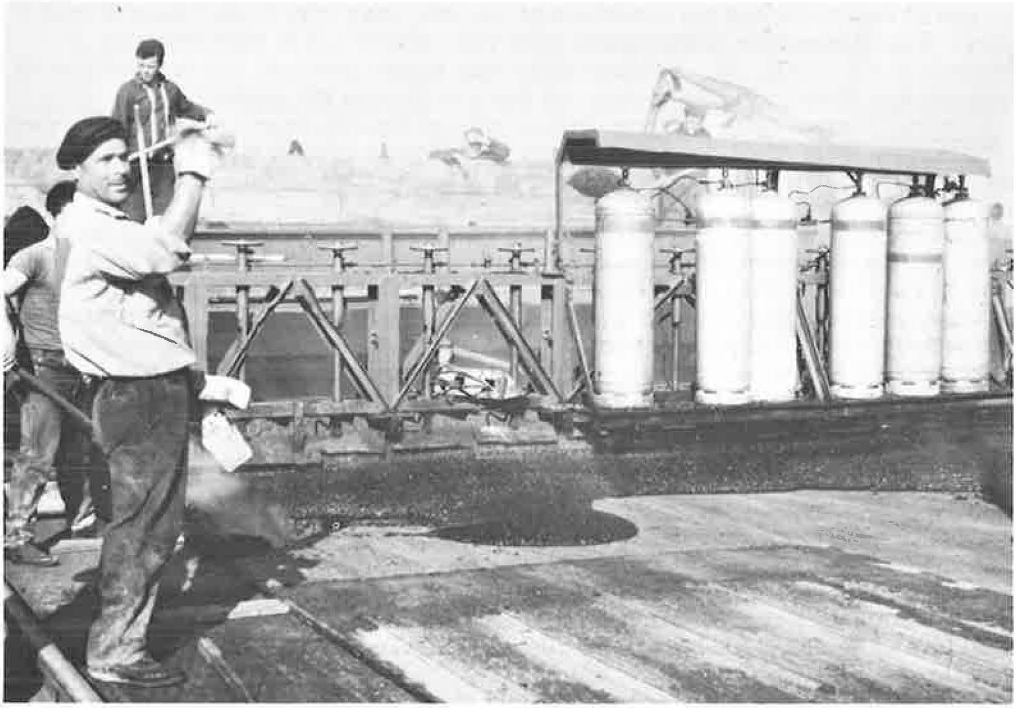

Figure 6. Dumping and spreading Gussasphalt in front of a self-propelled leveling screed.

Figure 7. Hand spreading lightly bitumen-coated stone chips over surface of freshly laid Gussasphalt.

Figure 8. Indentation roller used to depress stone chips and form dimpled surface on freshly laid Gussasphalt.

Figure 9. Appearance of finished Gussasphalt wearing surface.

are added to the Mastic, accounting for approximately 45 percent of the mix by weight. The mixture is then hand placed and spread to the required thickness at a temperature of approximately 425 F. A skid-resistant surface is obtained by casting lightly bitumen-coated $\frac{1}{2}$-in. stone chips over the surface at a rate of approximately 17 lb/yd^2.

Stabilized Mastix System

The second principal paving system used is the stabilized Mastix system, which consists basically of poured asphalt Mastix which is stabilized with a gap-graded stone rolled into the Mastix.

In this system, an asphaltic Mastix layer $\frac{3}{4}$ to 1 in. thick is placed directly on the prime coating. Crushed stone $\frac{3}{4}$ to 1 in. in size is then broadcast over the surface at a rate of 130 lb to 160 lb/yd^2 and rolled into the Mastix while it is still soft. The stone is precoated with $1\frac{1}{2}$ to 2 percent of bitumen to facilitate adhesion. The stone is rolled completely into the Mastix, resulting in a finished thickness of $1\frac{3}{4}$ to 2 in.

This system may be used with and without anchor bars welded to the top of the steel deck plate. The purpose of the bars, bent into a zig-zag pattern of 30 deg at 12-in. intervals, is to help stabilize the stone-filled Mastix, increase the bonding area to the steel deck, and prevent shoving.

The stabilized Mastix system with anchor lugs was first used on the Düsseldorf-Neuss bridge. The anchor bars were approximately $\frac{1}{4}$ by 1 in. in cross-section and were spaced a little over 3 in. apart. In subsequent installations, the cross-section of the bars was reduced to approximately $\frac{1}{4}$ by $\frac{7}{8}$ in. and the spacing between them was increased to almost 6 in. In the most recent application, the cross-section of the anchor bars was further reduced to approximately $\frac{3}{16}$ by $\frac{5}{8}$ in. while the spacing remained at about 6 in. (45).

In the former applications, each preformed bar was skip welded to the deck plate, either in the shop or in the field. In the latter installation, the anchor bars were prefabricated into grid sections by welding the anchors bars to similarly sized bars placed

Figure 10. Flame-spraying application of prime coat Okta-Haftmasse.

Figure 11. Placing and spreading Mastix.

Figure 12. Placing and rolling stone into freshly-placed Mastix.

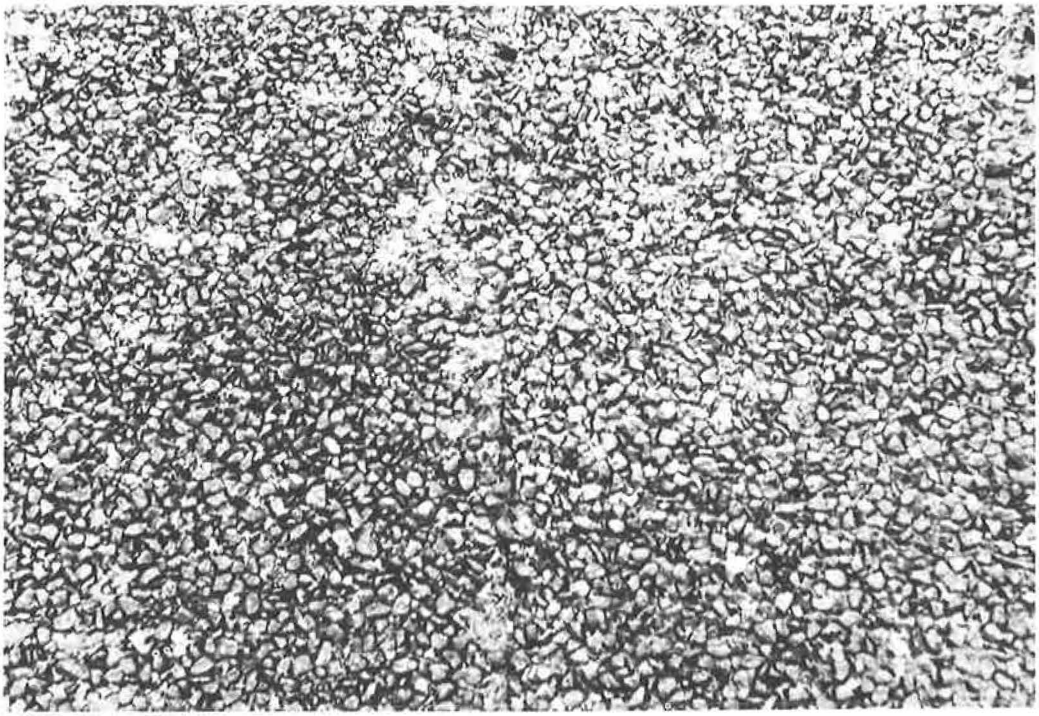

Figure 13. Appearance of stone-stabilized Mastix before placing surface dressing.

Figure 14. Placing and roller compacting fine asphalt concrete surface course.

TABLE 2

DETAILS OF BRIDGES SURFACED WITH STABILIZED MASTIX SYSTEM

Bridge and/or Location	Structural Details					Surfacing Details[a]			References
	Date in Service	Type	Deck-Plate Thickness (in.)	Type Stringers and Spacing (in.)	Anchoring Devices	Surfacing	Surface Treatment	Traffic	
Düsseldorf-Neuss	1951	Suspension	0.55	T's at 15.75	Zig zag at 3.15 in.	2 inches rolled in stone-stabilized mastix asphalt	Tack coat and stone chips	VH	(4, 7, 28, 29, 30)
Canal-Dusseldorf	1953	Bascule	—	—	Zig zag at 3.15 in.	2 inches rolled in stone-stabilized mastix asphalt	Tack coat and stone chips	M	(4, 28)
Dusseldorf-North	1957	Suspension	0.55	Angles at 15.75	Zig zag at 5.91 in.	2 inches rolled in stone-stabilized mastix asphalt	Tack coat and stone chips	H	(4, 28, 14, 29)
Breitscheid Dusseldorf	1961	Girder	0.39		None	2 inches rolled in stone-stabilized mastix asphalt[b]	Tack coat and stone chips	H	(4, 28, 14, 11)
Fuldatal-Kassel	1962	Truss	—	Y pans at 23.62	None	2 inches rolled in stone-stabilized mastix asphalt[b]	⅝-in. asphalt-concrete	L	(4, 28)
Frankfurt-Schwanheim	1963	Girder	—		None	2 inches rolled in stone-stabilized mastix asphalt[b]	⅝-in. asphalt-concrete	H	(4, 28)
Silltal-Innsbruck	1963	Girder	0.39	Flats at 15.75	Zig zag at 5.91 in.	2 inches rolled in stone-stabilized mastix asphalt	⅝-in. asphalt-concrete	H	(4, 28, 1)
Donau-Wörth	1964	Girder	0.47	Flats at 11.8	Zig zag at 5.91 in.	2 inches rolled in stone-stabilized mastix asphalt	1⅛-in. asphalt-concrete		(45)

[a]Okta-Haftmasse deck prime coat.

[b]Surfacing placed in two layers.

about 12 in. on center in the longitudinal direction. These preformed grids, approximately 6½ by 13 ft, were then tack welded to the steel deck in the field, thereby greatly reducing the time and labor involved.

When the stabilized Mastix system is used without anchor bars, the pavement is applied in two layers with all the material quantities halved. The rolled-in crushed rock is also reduced in size to ½ to ¾ in.

After the stabilized Mastix has been completed, a surface dressing is applied to complete the pavement surface. The dressing may consist of a bituminous seal coat with brushed-in stone chips or a ½ to ¾-in. layer of fine asphalt-concrete.

Figures 10 to 14 show various construction operations in the placement of a stabilized Mastix bridge pavement; Table 2 gives details of a number of bridges paved with this system.

Thin Combination Coatings

A thin combination coating system refers to a single layer of material which serves the multiple purpose of prime coat, leveling course and wearing surface. These lightweight pavements may consist of aggregate/binder mixtures or preformed sheet materials which are cemented to the deck plate. The materials may be field or shop applied, generally in thicknesses of ½ in. or less.

Few applications of thin combination coatings on steel deck bridges have been found in Europe although a number of materials are being investigated in laboratory and field trials. The only confirmed application in Germany has been a cement-latex material applied to a bascule bridge (18). The cement-latex mixture, applied in a ½-in. thickness, replaced an asphalt surfacing which broke loose from the deck when the bridge was raised to the near-vertical position. A number of bridge sidewalks and bicycle paths have been paved with cement-latex mixtures with good success. More recently, a new prefabricated steel bridge system has been developed in which standardized units are field assembled to form any given size bridge. The deck units are shop coated with a plastic material which the literature describes as a wear-resistant polyurethane film (48). A similar prefabricated bridge erected in England was shop coated with a ¼ to ⅜-in. epoxy resin/calcined bauxite grit mixture (27, 46).

To date, European experiences with thin combination coatings have been somewhat limited, and no definite procedures have evolved. The only known preparation has been sandblasting of the steel plate. Where resin/aggregate mixtures are used, the mixture may be placed directly on the steel plate or the resin binder sprayed or squeezed

TABLE 3
DETAILS OF BRIDGES SURFACED WITH THIN COMBINATION COATINGS

Bridge and/or Location	Structural Details				Surfacing Details		References
	Date in Service	Type	Deck-Plate Thickness (in.)	Type Stringers and Spacing (in.)	Surfacing	Remarks	
Eider Friedrichstadt	1958	Bascule	0.51	U pans at 27.56	½-in. sand-cement-rubber latex ½-in. sand-cement-neoprene latex	East lane West lane	(2, 11, 14)
Camphill-Birmingham	1961	Girder	½	Angles at 12	Epoxy resin-sand mixture	Shop applied	(15, 25)

over the surface, and the aggregate sprinkled onto the resin. Finishing, if required, is done by hand. Cement-latex mixtures are also applied and finished by hand. Hand finishing of thin coatings, plus the normal variations in the level of the deck, somewhat complicates the problem of maintaining a smooth riding surface and at the same time maintaining a minimum coating thickness. The use of bolted doubler plates for field connection of deck sections could not be permitted with this type of surfacing system.

Table 3 gives details of the European bridges paved with thin combination materials.

Performance and Maintenance

With few exceptions, performance data on bridges outside of Germany were not available or readily accessible; hence, most of the data are of German origin.

Before discussing the performance and maintenance of various deck surfacings, it is well, however, to explain the German contractual practices in deck paving. For any given project, contractors may bid on a number of approved paving systems unless a specific type of construction is called for by the owner. The paving contract is a prime contract between the owner and the paving company and not a subcontract with the structural contractor. Because few city, state, or federal bridge agencies in Germany have their own paving or maintenance crews, the successful bidder for any paving contract includes in his price a guarantee of performance and maintenance of the pavement for a given period of time, usually 5 or 10 years. During that time, any deficiencies or defects in the material or workmanship must be repaired or replaced by the contractor at his expense. Any remedial measures taken are usually agreed on between the owner and the contractor.

Great difficulty arises in determining what defects and deficiencies are directly attributable to the performance of the paving contractor and when distress in the pavement is sufficiently bad to warrant corrective measures. In many instances, the deterioration of a marginal paving system could probably be considerably prolonged, if not prevented, by appropriate and immediate corrective action. Delays or neglect in maintenance could conceivably accelerate the deterioration of a good pavement. Many of the factors affecting the performance are so interrelated that distress or failure from one cause may weaken the system to such an extent that succeeding failures due to other causes might be accelerated. Thus, in subsequent examinations it may be impossible to determine the original cause of failure.

In view of this situation, the paving contractors for orthotropic bridges must be reputable and long-established companies, preferably having past experience in this type of work. The quality of workmanship going into paving steel decks is considered to be of paramount importance, particularly since much of the work is done by hand. The quality of the work was acknowledged to have been considerably improved over the years, and coupled with the increased knowledge of the problems involved, the results are steadily becoming more satisfactory.

Layered System.—The layered system has been used on steel decks since 1948 and accounts for 80 to 90 percent of all orthotropic bridge surfacings. Because of its greater length of service, more information is available about its performance.

The types of distress associated with the layered paving systems include shoving, rutting, blistering, raveling, potholing, and cracking due to thermal and mechanical plate deflections.

SHOVING. The outstanding examples of shoving of steel deck pavement occurred on the Köln-Mulheim Bridge [1951] in Germany (8) and the St. Albans Bridge [1955] in

Basle, Switzerland (16). In the former case, the deck pavement consisted of a bituminous prime coat, $\frac{5}{16}$-in. Mastix Isolierung, and two layers of Gussasphalt. On two-thirds of the bridge, the Mastix Isolierung contained 22 percent bitumen and a bituminous-impregnated paper layer between the Isolierung and the leveling course. On the remaining third, the Isolierung contained 16 percent bitumen and no paper separator. After one year, shoving of the surface layers over the Isolierung having the higher bitumen content became apparent, and after two years the bond between the Isolierung and the leveling course was almost completely lost, with large chunks of the surface layers breaking loose. An investigation of the failure revealed that the shoving was caused by the instability of the Mastix layer containing 22 percent bitumen. Complete replacement of this two-thirds section was started in 1953 and completed in 1956. The new pavement was the same as that applied originally to the one-third that had performed very well.

The pavement on the St. Albans Bridge contained a bitumen-impregnated felt paper over a prime coat made up of two coats of bitumized latex. The felt paper was added to prevent bleeding of the prime coats which never completely dried (the surfacing was placed during the winter time). During the following hot summer, the surfaces began to shove, particularly near the ends of the bridge where vehicles were stopped by a traffic light. It was determined that the felt paper delaminated and formed a slip plane, resulting in loss of bond and the subsequent shoving. This situation became progressively worse over the next several years. The distressed areas near the ends of the bridge were replaced in 1962. The remainder of the deck, though rough, has given satisfactory performance and is still in use. Of the two major repaired areas, one was replaced with the original design without the felt paper layer. The other was replaced with a metallized zinc prime coating, a $\frac{1}{8}$-in. Mastix Isolierung, and two layers of bituminous concrete containing rubber latex. It is anticipated that the entire bridge will eventually be repaved with whichever composition is found to be the better in these two patched sections.

Small confined areas of shoving have occurred on other bridge pavements, such as the Eddersheim Bridge (2, 11). These have been attributed to loss of bond.

RUTTING. Rutting has occurred to some degree on nearly all deck pavements, especially in the outer or truck lanes. The rutting occurs during the hot summer months under heavy traffic and normally only becomes noticeable after 2 or 3 years in service. One of the more severe conditions of rutting occurred on the Eddersheim Bridge where the Mastix Isolierung was more than $\frac{3}{8}$ in. thick. This condition, however, was not considered serious enough to require extensive repairs.

Two other cases of rutting were reported in which a series of deformations formed parallel to one another and reflected across almost the entire traffic lane. In both situations, other defects such as shoving or cracking contributed to the general deterioration of the wearing surface. The entire bridge or the larger portion of both bridges has since been repaired. In the 4 and 9 years, respectively, since repair, the previous defects have not created a problem.

BLISTERING. Blistering of the surface courses occurred on the Eddersheim, the Haseltal, the Norderelbe in Hamburg, and the Rhine Bridge at Mainz-Weisenauer. In a all cases except the Norderelbe Bridge, metal foil was used as the Isolierung, and the blistering was believed due to entrapped air. On the Norderelbe Bridge, the blistering was attributed to entrapped moisture between the prime coat and the leveling course as it rained during the laying operation (21). In all cases, the blistering was confined to small pockets (usually near the median of the bridge) and the area of bond destruction was very small. In many instances, these blisters have been cut out and patched, usually with a cold bituminous mix.

RAVELING. Raveling is considered more of a nuisance than a problem and can normally be prevented. The most severe case in Germany was on the Eddersheim Bridge in the paving section which contained asbestos (2). This was the first time asbestos had been used as an admixture in Gussasphalt and handling difficulties were experienced. The raveling was considered more the result of mishandling the material than the asbestos itself. Another known case of light raveling was on the Save River Bridge in Belgrade (17). This case was attributed to the passage of steel-rimmed wheels (horse-drawn carts), and it did not impair the usefulness of the pavement.

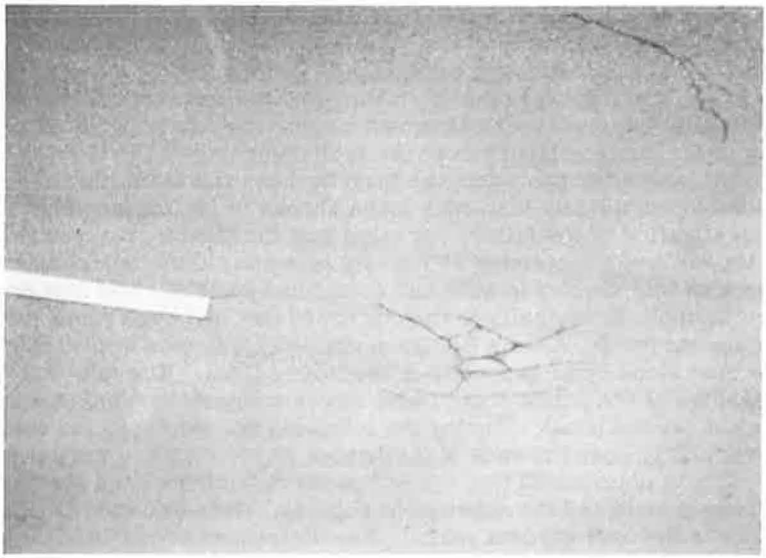

Figure 15. Potholing on Mannheim-Ludwigshafen Bridge.

POTHOLING. A few cases of potholing resulted from blisters, but these can easily be repaired and prevented by repairing the blisters. Another case not due to blistering was observed on the Mannheim-Ludwigshafen Bridge (5). The pavement consisted of a bituminous varnish prime coat, $^{5}/_{16}$-in. Mastix Isolierung, $1^{1}/_{8}$-in. asphalt binder leveling course, and a 1-in. Gussasphalt surface course. In constructing the bridge, the upstream half was completed and paved up to the binder leveling course and opened to traffic. About 6 months later, the downstream half of the bridge was completed and paved up to the leveling course. The Gussasphalt surface course was then placed on the entire bridge. One to two years later, potholes began appearing on the downstream but not on the upstream bridge. The owners venture the opinion that on the downstream half the binder leveling course, which is an open-graded material, eventually was compacted by traffic, and the hard wearing surface acting as a plate eventually broke in forming the potholes (Fig. 15). The lack of potholes on the upstream bridge is attributed to the compaction given the leveling course by the heavy traffic before placement of the wearing surface. No other known factors could have contributed to this situation.

No maintenance measures were applied to these potholes except for inspecting a few to determine the extent of the damage. In no case did the cracks affect the bond or penetrate down to the Isolierung. As expected under the circumstances, there was no evidence of corrosion of the deck plate.

CRACKING. Surface cracking in an orthotropic deck pavement is a frequent problem. Cracks attributed both to temperature changes and deck plate deflections have been experienced in a number of structures (2, 5, 4, 11).

Temperature cracking was particularly severe during the cold winter of 1956. Numerous instances of thermal cracking were reported during this winter. On the Hedemunden Bridge, over 60 cracks occurred in one night when the temperature dropped some 70 deg in a period of a few hours. Other bridges which experienced thermal cracks at that time, were the Kurpfalz Bridge in Mannheim, the Köln-Mulheim, the Eddersheim and the Wimpfen Bridges. During other winters, cracks were reported to have occurred to a lesser degree on the Rhine Bridge at Speyer, the St. Albans Bridge, the Mannheim-Ludwigshafen Bridge, and the Haseltal Bridge. In no case did the cracks extend through to the steel plate, and only in the case of the Hedemunden Bridge did the cracks penetrate to the Isolierung.

TABLE 4

CRACKING DUE TO PLATE DEFLECTIONS

Bridge	Plate Thickness (mm)	Type Stiffeners and Spacing (in.)	Location of Cracks	Years Before Cracks Observed	Depth of Penetration
Köln-Mulheim	12	Bulb T's at 11.8	Over ribs	6	Top layer
Eddersheim	14	Flats at 13.8	Over ribs	4	Top layer
Hedemunden	12	Bulb T's at 11.8	Over ribs	5	Isolierung[a]
Weser Porta	12	U pans at 23.6	Center of pans	6	Top layer
Strasbourg-Kehl	12	Flats at 11.8	Over ribs	3	Top layer
Köln-Severin	10	Flat at 11.8	Over ribs	2	Top layer

[a]Cracks were also found in the leveling course; they had started at the bottom and progressed upwards but were not visible from the surface.

Of all the various surfacing systems used on these bridges, there is no readily available common denominator (other than they all had a Gussasphalt surface layer) which could explain the reason for the thermal cracks. However, thermal cracking occurred most often and generally to a greater degree in pavements with a metal-foil Isolierung. The use of a metal-foil Isolierung, in addition to being expensive, has been of questionable value because of the reliance on good workmanship for its success. No use of metal foil as the Isolierung has been reported subsequent to 1961.

The severity of the thermal cracks is difficult to assess. The reportedly mild cracks which developed in the Mannheim bridges were filled soon thereafter as were the cracks on the Köln-Mulheim and Eddersheim bridges. The very severe cracks on the Hedemunden Bridge were left unattended. This bridge, however, was completely repaved in 1962, 6 years after the cracks occurred. At the time of repaving, no evidence of corrosion of the deck plate was found.

Deflection cracks are longitudinal cracks running parallel to the stringers or rib stiffeners with the predominant location being directly over the stiffeners. The continuous flexing of the steel plate by local wheel loads eventually causes the asphalt to rupture.

Cracking due to plate deflections has been reported on the Köln-Mulheim, Hedemunden, Edersheim, Weser Porta, the Köln-Severin, and the Strasbourg-Kehl bridges. Some of the specific circumstances of the cracking on these bridges are given in Table 4.

Figure 16. General condition of wearing surface of Köln-Severin Bridge.

The cracks on the Köln-Severin Bridge, which appeared after two years in service, have been attributed to the flexibility of the deck plate. The cracking has been limited in extent, but will probably get progressively worse. The general condition of this bridge pavement is shown in Figure 16. The reflection of all of the plate stiffeners is easily distinguished, and this was the only instance where the condition was readily observable.

In the case of the Strasbourg-Kehl Bridge, the heavy traffic is very slow moving and very often standing because there are customs stations at either end. This slow-moving or stationary traffic causes increased deflections because no structural support is obtained from the surfacing materials. Also, vibratory loads from idling vehicles tend to deform the pavement. There has been considerable patching, which indicates that the problems are severe.

The Eddersheim Bridge, which has a 14-mm deck plate, would ordinarily be expected to perform quite well. This particular bridge, however, has experienced almost every type of distress. The general condition of the entire bridge surface is poor and is rough riding. The overall problem here is attributed to poor workmanship. In certain respects, this was a test bridge in that six different material combinations were used in the pavement. Several sections contained additives, such as rubber and asbestos, some used for the first time. Inexperience in handling these materials, and the fact that the entire job was performed by hand, resulted in a poor-quality pavement giving unsatisfactory results. Continuous patching has prolonged the life of the pavement, but it will undoubtedly have to be replaced.

Deflection cracks in the Hedemunden Bridge were observed after thermal cracks had appeared. The contribution of the previous distress to the deflection cracks could not be determined. The new pavement on this bridge has been trouble-free after three years in service.

The most puzzling cracks occurred in the Weser Porta Bridge. Although reported as deflection cracks, they occurred over the center of the closed pan-rib stiffeners. The stiffeners themselves also are enclosed in a box girder which is the main supporting member. The strains and relative deflections are usually low at these points. Under conditions of rapidly changing temperature, the expansion or contraction of the deck plate would probably lag behind that of the pavement because of the insulating effect of the air in the closed stiffener, presenting the possibility of cracking due to thermal effects. The cracks have not been repaired, and are reported to be getting progressively worse.

CORROSION. Corrosion of steel deck plates has not been a problem in Europe. The waterproofing of the steel deck is enhanced by the use of a Gussasphalt or Mastix surface course which is impermeable. If cracks occur in the surface course, permitting water to penetrate the pavement, the steel deck is still protected by the Mastix or foil Isolierung, both of which are also impermeable. No evidence was found in the field of water penetrating to the steel plate to cause corrosion. In many instances, test patches had been removed from bridge decks to inspect for corrosion, but none was found. Corrosion was not even found in the case of the Hedemunden Bridge which developed a number of cracks over a period of six years before the deck was repaved. In general, it was concluded that the deck-protection systems used in Europe are very effective in preventing corrosion.

Successful Layered Systems.—There have been a number of cases where excellent results have been obtained with layered pavements. Notable among these are the City of Mannheim bridges. The Spatzen swing bridge, built in 1948, is essentially defect-free, and has not required maintenance of any sort. However, this bridge is subjected to relatively light traffic. The Kurpfalz Bridge and the Mannheim-Ludwigshafen Bridge, despite a few temperature cracks in the upstream span, are still in excellent condition after 15 and 6 years of service, respectively. Both experience heavy traffic. The Duisburg-Hamburg Bridge and the Thousand Window House Bridge, also in Duisburg, are both defect-free after 11 and 7 years of service, respectively. Last reports indicated the Save River Bridge in Yugoslavia to be in excellent condition. This bridge has a 10-mm deck plate over much of its surface. The traffic is, however, rather light, as trucks are only infrequently routed over this bridge.

A number of other bridges with 2 or 3 years of service have been reported in excellent condition with no signs of impending distress.

Stabilized Mastix.—Only one problem has been reported (4) in bridges paved with stabilized Mastix. The Düsseldorf-Neuss Bridge, the first on which this system was used, had zig-zag anchor bars, 0.39 by 1.10 in. in cross-section, spot welded to the deck 3.15 in. on center. The deck prime coat of Okta-Haftmasse was hand painted and Mastix asphalt was poured level with the top of the anchor lugs. No crushed stone was rolled in and only small stone chips were imbedded in the surface to give a skid-resistant texture. During the first summer, excess prime coat material which had accumulated at the corners of the anchors and the deck began to bleed to the surface causing a slick layer to form on the surface. The contractor corrected the surface by rolling heated crushed rock into the Mastix with a heavy heated roller. The crushed rock was spread at the now-specified rate of 130 to 160 lb/yd^2, and it increased the pavement thickness to approximately 2 in. The finished surface, although somewhat rough, has given excellent service with no further distress during the past 12 years under extremely heavy traffic.

A similar pavement on the Düsseldorf-North Bridge, after 10 years of service under heavy traffic, is in near-perfect condition. The spacing of the anchor lugs was increased to almost 6 in. and the crushed stone was rolled into the Mastix before hardening. The Okta-Haftmasse prime coat was flame-sprayed on this and subsequent installations. The oldest installation using this system without the anchor lugs (Breitshied Test Bridge) is now 4 years old and reported to be in excellent condition.

Thin Combination Coatings.—The only major steel deck bridge using thin combination coatings is the bascule portion of the Friedrichstadt Bridge over the Eider which was paved with a latex-cement mortar, half containing a natural latex and half containing a neoprene latex. The coating was hand troweled directly onto the steel plate to a ½-in. thickness. No information was available about the performance of this bridge other than that there was no apparent distress after 6 years of service.

The prefabricated flyover bridge erected in Birmingham, England, in 1961 was shop coated with a ¼ to ⅜-in. layer of epoxy resin filled with calcined bauxite grit. After three years of service, the surface was reported in excellent condition.

Research on Paving Materials

German Programs.—Prewar tests for evaluating surfacing materials on bridge decks carried out in Germany generally led to the conclusion that the Mastix asphalts were superior to other types of asphalt mixes. It was also found that a waterproof membrane was unneccessary under the Mastix and that anchoring devices, such as steel-mesh reinforcing welded to or supported by studs on the steel decks, were highly desirable. A number of prewar steel deck bridges were built using this system. These conclusions, however, were somewhat modified by later experience.

Shortly after the war, research on bridge-wearing surfaces was reactivated in conjunction with the Kurpfalz Bridge which was the first major bridge using orthotropic design. Various Mastix asphalts were evaluated, with and without rubber additives and with and without anchoring devices (15). The results of these tests were inconclusive but, based on observations of performance on other bridges, it was concluded that anchor reinforcement should not be used because voids often remained around the wire mesh. It was also recommended that waterproof membranes be used.

A large-scale test program was carried out in connection with the Köln-Mulheim Bridge by the Machinenfabrik-Augsburg-Nurnberg (M-A-N) works in Gustavsburg in which 0.312-in. steel plates with a 2-in. Gussasphalt topping were tested (30, 31). The plates were deflected by a pulsating mechanism to determine (a) durability of the topping, (b) damping effects under vibration, and (c) contribution of the topping to the stiffness of the plate at various temperatures and for different loading rates. Additional tests were carried out to determine the bond strength of various Isolierung layers as a function of temperature, the indentation in Gussasphalt of a loaded wheel with different grades (penetrations) of bitumen, and the effects of temperature on the sonic modulus of Gussasphalt.

One of the findings of this test program was the importance of good bond to the steel plate. Lack of bond would soon result in shoving and cracking, or both, depending on the temperature of the pavement. Achieving good bond was not considered a difficult problem and could be accomplished with a variety of materials even at low temperatures. The bond strength was also found to be important from the standpoint of increased stiffness of the steel plate. In the M-A-N tests, the stiffness of a 0.312-in. steel plate with a 2-in. Gussasphalt topping was increased as much as 50 percent under an instantaneously applied load at 122 F. The percentage increase in stiffness decreased with increased loading time, as expected at that temperature; but it was still measurable at loading durations of 8 sec.

In 1959, a test program was started by the German Ministry of Transportation to evaluate various prime coatings (38). These tests completed in 1963 examined metal-base, epoxy-base, and bituminous-base coatings. These materials were evaluated by means of adhesion, impact and bend tests at various temperatures. The general conclusions were that bituminous-base materials performed best under all conditions, particularly the proprietary material Okta-Haftmasse. The epoxy-base materials, although having good adhesive and impact properties, performed poorly in the bend test, especially at low temperatures. Another significant finding concerned the performance of zinc prime coatings. Cold-applied zinc-base paints performed poorly under all test conditions. The hot-sprayed zinc coatings, although considered to be effective rust preventives, performed well under the other tests only when applied in thicknesses of 1 mil or less. Any greater thickness greatly reduced the bond strength. It was found that zinc particles deposited outside the heat-affected zone were not intimately bonded and could be flaked off. Hot-sprayed zinc coatings applied in more than one layer also were found to have poor bond strength.

British Programs.—Tests on paving materials made by the Road Research Laboratory (32) confirmed the contribution of the topping to the plate stiffness. Dynamic strain measurements were taken at critical locations on the plate of a test panel having a ½-in. plate thickness with bulb-shaped stringers spaced 15 in. on centers. The dynamic strains were measured for a wheel load of 11,000 lb at various temperatures at different traveling speeds on the steel deck with 1-in. and 1½-in. Mastix asphalt toppings. These readings were compared to those measured on the steel deck without any asphalt topping. At 32 F, the deck stiffness was increased on the order of 400 percent and 200 percent for the 1½-in. and 1-in. toppings, respectively (at a loading vehicle speed of 2 mph). The increase in the plate stiffness was approximately 250 percent and 100 percent, respectively, at a temperature of 47 F. The plate stiffness appeared to increase slightly with increased loading speeds at both temperatures, but to a lesser degree than in the German tests. At a temperature of 86 F, however, the British found that neither topping thickness contributed significantly to the plate stiffness.

The increased stiffness of a steel plate with a Mastix asphalt topping at low temperatures is attributed to the increase in modulus of the asphalt. German sonic measurements have indicated that the modulus of Gussasphalt varied from 1×10^6 psi at 0 F to 8,000 psi at 120 F. An increase in modulus obviously results in a higher stress at any given strain level, but fortunately the flexural strength of asphalt also increases with a decrease in temperature and usually at a faster rate than the modulus increases. The bond of the asphalt pavement to the steel deck also is an important factor in the stress produced in the topping. For a given load on a ½-in. plate with a 2-in. topping, the stress in unbonded asphalt will be almost twice that when the asphalt is intimately bonded to the steel plate.

The purpose of the British tests was to determine the minimum topping thickness that (a) would sustain traffic for a period of 5 years, (b) could be placed in a single layer, and (c) would adequately protect the steel deck from corrosion. A stone-filled 1½-in. mastic layer was found to be suitable, and although a few cracks did occur over the plate stiffeners after about 4½ years, the cracks did not penetrate to the steel deck. Cracks occurred in a 1-in. stone-filled mastic pavement after 1½ years and eventually penetrated to the steel plate. Corrosion occurred at these points, and it was deduced that a bituminous primer alone would not prevent corrosion once a crack penetrated to it. It was also determined that great care must be taken at the edge of the topping to

prevent water ingress which would eventually destroy the bond and cause corrosion. There was no advantage in using a checkered steel plate to increase the bond strength or to prevent shoving.

The Road Research Laboratory has also been active in the evaluation of thin combination coatings consisting primarily of epoxy resin binders (46). Several test installations have been made, with mixed results. Thin surface dressings, formed by hand painting or spreading pure resin onto the sand-blasted steel plate at a rate of approximately 2 lb/yd^2 and sprinkling with a $\frac{1}{8}$-in. to No. 14 sieve grit at the rate of approximately 8 lb/yd^2, have a tendency to wear away under the action of traffic. Other types of resinous coatings investigated were slurry coats consisting of 1:1 or 2:1 sand/resin mixtures applied to the steel plates in approximately $\frac{1}{8}$-in. thicknesses and sprinkled with $\frac{1}{8}$ to $\frac{3}{16}$-in. sized grit and a trowel coat consisting of a 6:1 or 7:1 sand/resin mixture applied over a prime coat of pure binder material to a thickness generally not exceeding $\frac{3}{8}$ in. The latter two methods have in general given fairly satisfactory service. In the opinion of the Road Research Laboratory, however, it is still too early to make any definite conclusions as to the suitability of resin/grit mixtures for steel deck bridge pavements.

Additional exploratory tests, both in the laboratory and in the field, have been conducted on other combination coatings such as rubberized bitumen, rubber latex-cement mixtures, and filled and unfilled preformed sheets of polyvinyl chloride (27). All of these materials had some merit as wearing surfaces for steel decks. These programs, however, were not carried to the point where their properties could be evaluated in terms of permanence and durability.

Structural Systems

Although this survey was primarily concerned with paving systems, a few comments on the plate structural systems used by the Germans appear in order because the pavement performance is at least indirectly affected by the deck plate.

The plate thickness over any given bridge deck is determined from stiffness and strength considerations, generally resulting in a number of different thicknesses over the length of the bridge. The larger portion of the bridge, however, will have some minimum plate thickness which is determined by the allowable plate bending stresses due to local wheel loads. The local plate stresses are determined for a wheel load of 14 tons (10 tons load plus a 40 percent impact factor) acting over an area of 7.85 in. in the longitudinal direction and 23.6 in. in the transverse. This area is increased to 12.6 and 28.4 in., respectively, when using a thick asphalt pavement (20). The most common plate thickness in German designs is 0.472 in., although 0.393-in. and 0.552-in. plates are often used. Two grades of steel plate are used, St 37 or St 52, with allowable combined stress values of 28,300 and 42,500 psi, respectively. There are no standard German specifications governing plate deflections, although they normally do not exceed one three-hundredth of the span (L/300).

The plate deflections, of course, are influenced by the rib stiffeners. Two basic rib systems are used: (a) the open ribs such as flats, bulb T's, or angles, and (b) closed ribs such as U-shaped or trapezoidal pans. The spacing of the stiffeners varies depending on the plate thickness, but no spacing less than 11.8 in. has been reported for any plate thickness. Both rib-stiffening systems are patented in Germany. The open-rib system was an earlier development and most of the early bridges were constructed using this system. The closed stiffeners, which add considerably to the torsional stiffness of the orthotropic deck, usually resulting in a lighter structure, have gained considerable ground and now appear to be the preferred system. In the opinion of German authorities, the relative merit of the two structural systems, as they affect the pavement performance, has not been clearly established. Most believe that a 10-mm (0.393-in.) plate supported by ribs on 11.8-in. centers is too flexible to permanently support the paving materials now used. The performance of one of the major bridges with these structural features appears to bear out their concern. The performance of pavement on decks having 12-mm (0.472-in.) plates has been mixed and appears to depend as much on workmanship of the paving operation as on the structural

TABLE 5

SUMMARY OF DETAILS ON WEARING SURFACES FOR AMERICAN ORTHOTROPIC STEEL DECK BRIDGES

Bridge and Location	Date in Service	Structural Details			Surfacing Details			Remarks	References
		Type	Thickness (in.)	Type of Ribs and Spacing (in.)	Prime Coat	Leveling Course (in.)	Wearing Course (in.)		
Troy-Illinois	1962	Girder	1/2	Pans at 26	Coal-tar epoxy	1 1/4 AC with latex	1 1/4 AC with latex		(33) (53)
						1 1/4 AC with rubber powder	1 1/4 AC with rubber powder		
						1 1/2 AC with rubber	1 SA with latex and asbestos	Replaced after 2 years	
						1 1/4 AC with asbestos	1 1/4 AC with asbestos	Replaced after 2 years	
						1 1/2 AC	1 SA with latex		
	1964					1 1/2 AC	1 AC with rubber powder		
						1 1/2 AC	1 AC		
						1 1/2 AC	1 Miradon	AC with synthetic resin binder	(50)
Port Mann, Vancouver	1964	Tied arch	7/16	Pans at 24	Coal-tar epoxy	1/2 to 3/4 SA	1 1/2 AC	Deck first primed with red lead epoxy paint	
Humphreys G Creek, Maryland	1964	Girder	3/16	Corrugated pans at 2 1/2	Coal-tar epoxy	1 SA	1 SA		(51)
						1 SA with rubber	1 SA with rubber		
Dublin, California	1965	Girder	3/8	Pans at 24	Coal-tar epoxy	—	1 7/8 AC	Stone chips embedded in prime coat	(34)
			7/16	Pans at 24		—	Epoxy resin mortar	Coal for epoxy binder	
Ulatis Creek, California	1965	"Battledeck"	—	—		—	1 1/2 AC		(36) (37)
						—	1 1/2 Epoxy AC		
					Epoxy resin	—	1 1/2 Epoxy resin mortar	Guardkote 250 binder	
					Epoxy resin	—	1 1/2 Epoxy resin mortar	Concresive 1064-3 binder	
					Epoxy resin		1 1/2 Epoxy resin mortar	Resiweld R-7122 binder	
NJ Turnpike, Elizabeth	1965	Girder	3/8	Flats at 14	—	—	1/4 to 3/8 Epoxy resin slurry	Steel mesh welded to plate	(51) (52)
					Polyester resin	—	1/4 to 3/8 Polyester mortar		

Note: AC = Asphalt concrete SA = Sand asphalt.

considerations. All the paving technologists are agreed that 14-mm (0.550-in.) deck plates would be preferable as the resulting lower deck flexibility would materially increase the permanence of the wearing surfaces. Their opinion is not shared by the bridge designers who suggest that a change in the pavement design would be a more appropriate and economical solution than structural changes not required by strength considerations. The use of zig-zag bars is another method of increasing the deck plate stiffness while stabilizing the wearing surface. The addition of the zig-zag bars is considered a rather expensive solution and some concern has been expressed that the tack welds on the deck may act as stress raisers giving rise to possible fatigue cracks. These latter fears appear to be unjustified on the basis of a fatigue testing program carried out at the Technical University of Stuttgart (47).

SURFACING SYSTEMS IN THE UNITED STATES AND CANADA

The use of orthotropic steel bridge deck design has only recently come into use in North America and at present only one major structure is in actual service. More recently, three smaller steel deck structures were placed in service with two of them having been opened to traffic less than a month at the time of this writing.

Since their numbers are limited and much of the information concerning these bridges is of an experimental nature, they will be discussed separately indicating, where applicable, the performance to date. A summary of all the bridges now in service, giving some of the structural and surfacing details, is given in Table 5.

Port Mann Bridge

The Port Mann Bridge is a 1,920-ft tied-arch (1,200-ft main arch span with two 360-ft side spans) spanning the Fraser River in Vancouver, British Columbia (50). This bridge, opened in June 1964, has a $\frac{7}{16}$-in. deck plate stiffened by spread U-shaped ribs, spaced 2 ft on center. The deck was surfaced using the layered system as previously defined. After sand blasting the steel plate, a 4 to 5-mil primer of red lead epoxy was applied. The primer was cured with heat generated within a portable shelter placed on the deck. Within 24 hours of applying the primer, a coal tar epoxy prime coat was applied at a rate of approximately 30 ft^2/gal. After mixing the two coal tar epoxy components, $\frac{1}{2}$-lb asbestos was added to each gallon and mixed in thoroughly. Prior to setting, the coal tar epoxy was sprinkled with crushed rock chips passing a No. 4 but retained on a No. 10 sieve at a rate of approximately 7.5 lb/yd^2. The leveling course consisted of a $\frac{1}{2}$ to $\frac{3}{4}$-in. layer of sand asphalt, machine laid and compacted with rollers level with the top of the $\frac{1}{2}$-in. splice plates used to join the deck sections. The sand asphalt was designed to contain no more than 8 percent voids, and to have a modified Marshall stability of 1,200 with a bearing capacity of 130 psi at 140 F. An asphalt emulsion tack coat was applied before placing the leveling course. The surface course was $1\frac{1}{2}$-in. asphalt concrete containing 6.3 percent of 90-penetration bitumen and $\frac{1}{2}$-in. maximum aggregate.

The bridge has now been open to traffic for approximately $1\frac{1}{2}$ years. The performance of the deck pavement has been reported as excellent with no cracks of any kind visible. The only distress encountered was the polishing of the surface aggregate due to extensive use of tire chains. There has not been any inspection of the subsurface courses as yet, but they are presumed to be performing as expected.

Troy Test Bridge

A small test bridge was installed near Troy, Ill., late in 1962 to evaluate experimental pavements for the Poplar Street Bridge now under construction in St. Louis, Mo. (33). Four different paving combinations were used.

1. Type I pavement consisted of two $1\frac{1}{4}$-in. layers of asphaltic-concrete with a synthetic latex additive. Aggregate for the asphaltic-concrete consisted of approximately 92 percent traprock, 5 percent fine sand, and 3 percent mineral filler. To this was added 5.7 percent of 85 to 100 penetration asphalt cement and 0.6 percent liquid latex.
2. Type II pavement consisted of two $1\frac{1}{4}$-in. layers of asphalt-concrete with an asphalt rubberizer additive. The aggregate composition was the same as for asphalt

pavement Type I, but the asphalt cement (85 to 100 penetration) addition was 6.00 percent, and 0.235 percent asphalt rubberizer was used.

3. Type III pavement consisted of an approximately 1½-in. leveling course of asphalt-concrete with an asphalt rubberizer as specified for the Type II pavement and a 1-in. thick sand-asphalt surface course containing asbestos fiber and liquid latex additive. The sand-asphalt aggregate contained about 55 percent coarse sand, 18 percent fine sand, 20 percent traprock fines, 5 percent mineral filler, and 2 percent asbestos fiber. To this aggregate was added 9.3 percent 85 to 100 penetration asphalt cement.

4. Type IV pavement consisted of two 1¼-in. layers of asphaltic-concrete with asbestos fiber additive. The aggregate was 63 percent traprock, 9.7 percent course sand, 19.4 percent fine sand, 4.9 percent mineral filler, and 3 percent asbestos fiber. To this was added 8.7 percent of 85 to 100 penetration asphalt cement.

Each type of paving material was laid in two sections, and was placed over a different type of coal tar-epoxy deck primer. Sand was sprinkled onto each type of deck primer before setting to increase the bond with the wearing surfaces.

After one year in service, cracks had occurred in the Type IV pavement, one of which extended into an adjacent section with another paving type. Shortly thereafter, many additional cracks occurred in all four pavement types, with those in Types I and II being minor. It was reported that most of these cracks were of the hairline variety and healed during the hot summer months. Cores taken through the crack revealed that the cracks did not penetrate beyond the top course.

During early summer 1964 all but a short section of Types I and II pavement were removed and the remainder of the bridge was repaved using four new paving compositions. All the replaced sections were removed down to the coal tar-epoxy prime coat which was still tightly intact. The leveling course for all the new sections consisted of approximately 1½-in. asphalt-concrete conforming to the Illinois Standard Specifications for surface course mixes. The new surface courses, applied in 1-in. thicknesses, were of the following types:

1. Type I-A pavement consisted of a sand-asphalt paving mixture containing a liquid latex additive. The mixture consisted of 90 to 93 percent sand and filler with a binder content of 7 to 10 percent by weight. The binder contained approximately 8 percent rubber solids by weight. The mixture was laid at approximately 300 F.

2. Type II-A pavement consisted of an asphaltic-concrete containing a rubber powder. The aggregate blend was the same as used in the leveling course and contained approximately 6 percent binder; 5 percent rubber powder by weight of the binder was added to the mix. This mix was also laid at a temperature of approximately 300 F.

3. Type V pavement conformed to the I-11 Illinois Standard Specification for bituminous surface course.

4. Type VI pavement consisted of Miradon (Humble Oil Co.), a proprietary paving formulation using a synthetic thermoplastic binder material. The gradations, proportions, and handling procedures were in accordance with the recommendations of the binder material supplier.

The repaved test bridge was reopened to heavy traffic in September 1964. Inspections during the spring of 1965 showed that hairline cracking had occurred in Types II-A and VI. Type I-A pavement had somewhat wider cracks, whereas the Type V pavement showed no visible distress. The hairline cracks which were observed in Types I and II pavements before the repaving operation had extended slightly, but none of the new cracks were considered to be significant. Continuing observations are being made on this bridge.

Dublin Bridge

A 324-ft four-span orthotropic steel deck bridge was recently completed on I-680 over US 50 near Dublin, Calif. (34). Two spans have a $^7/_{16}$-in. deck plate; the remaining two have a ⅜-in. plate. All rib stiffeners are made up of $^5/_{16}$-in. plate trapezoidal pans 12 in. wide and 6 in. wide at the top and bottom, respectively, 8½ in. deep, and spaced 2 ft on centers.

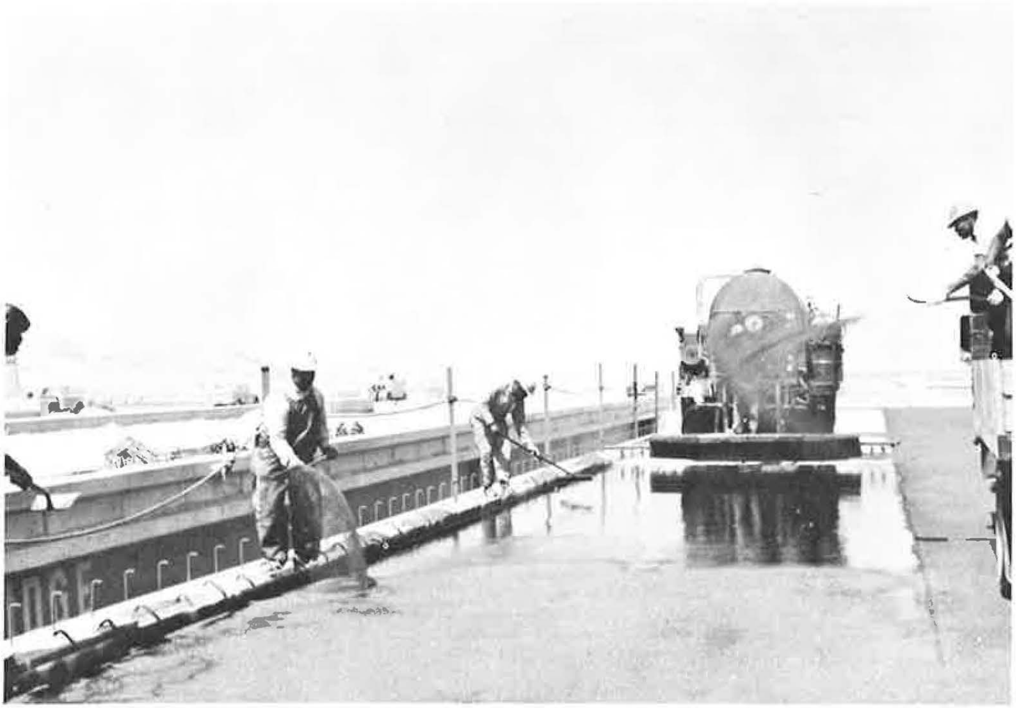

Figure 17. Spray bar application of coal tar epoxy and broadcasting of alumina grit over surface on Dublin Bridge.

Figure 18. Application of coal tar and crushed rock as prime coat and keying anchors for asphalt-concrete wearing surface on Dublin Bridge.

Figure 19. Spray bar application of second epoxy coating over anchoring keys on Dublin Bridge.

Figure 20. The laying of the asphalt-concrete wearing course on the Dublin Bridge.

The bridge half with the $^7/_{16}$-in. deck plate is paved with a coal tar epoxy-grit mixture with an average finished thickness of $^1/_4$-in. The epoxy was applied in one pass with a Broyhill spreader at a rate of 6.6 lb/yd^2. Immediately after applying the epoxy, aluminum oxide grit passing a No. 8 sieve and retained on a No. 16 sieve was broadcast over the surface at an approximate rate of 27 lb/yd^2. This operation is shown in Figure 17.

The portion of the bridge having the $^3/_8$-in. deck plate has a prime coat of coal tar epoxy applied by hand and spread with a squeegee to a thickness of approximately $^1/_{16}$ in. Immediately thereafter $^1/_2$ to $^3/_4$ in. of crushed rock was scattered by hand over the epoxy. A second $^1/_{16}$-in. layer of epoxy was applied over the crushed rock with the Broyhill spreader. An asphalt-concrete wearing surface was placed on the prime coat in one lift to produce a final wearing surface thickness of 2 in. The wearing surface contained 6 percent bitumen and had a maximum aggregate size of $^1/_2$ in. An asphalt emulsion fog coat was added over the wearing surface as a sealer. The paving operations of the layered system are shown in Figures 18 to 20.

Under each of these paving systems, the steel deck was divided into three areas. The deck preparation for these areas was (a) sand blasted only; (b) sand blasted with a 1-mil hot-sprayed zinc coating, and (c) sand blasted with a 5-mil hot-sprayed zinc coating. The only difficulty encountered in the application of the pavements was with the thickness tolerance of the epoxy-grit surfacing. The epoxy was applied during very hot weather and because of the high material viscosity it immediately began to flow slightly toward the low end of the deck which was on a $1^1/_2$ percent slope. The flow was stopped when the grit was applied but the surfacing ended up slightly thicker at one side than at the other.

This bridge was opened to traffic during December 1965, and as yet has no performance history.

Ulatis Creek Bridge

The Ulatis Creek Bridge is a short-span steel deck bridge of the "battledeck" type on Route 40 near Vacaville, Calif. The steel deck of this bridge, installed in 1952, has been previously surfaced with a number of experimental coatings (26). During late summer 1965, this bridge was surfaced with five experimental paving materials being evaluated by the California Division of Bay Toll Crossings for possible use on the San Mateo-Hayward Bridge presently under construction (36). The five materials used are as follows: (a) an asphaltic-concrete (particulars unknown); (b) Epon asphalt-concrete (Shell Oil Co.); (c) epoxy resin mortar (binder: Guardkote 250, Shell Oil Co.); (d) epoxy resin mortar (binder: Concresive 1064-3, Adhesive Engineering Co.); and (e) epoxy resin mortar (binder: Resiweld R-7122, Fuller Co.).

All materials were placed in a single lift on a 10- by 12-ft section to a thickness of $1^1/_2$ in. and applied to the steel plate which had been sandblasted and coated with 3 mils of inorganic zinc primer. The asphalt-concretes were hand placed and compacted with steel wheel rollers. Tack coats of SS-1h and Epon asphalt were applied prior to placing the asphalt-concrete and the Epon asphalt-concrete, respectively. In the case of the epoxy mortars, all were primed with the pure resin binder before applying the mortar mix (Fig. 21). The three methods of placing and finishing epoxy resin mortars were hand troweling, power troweling, and power screeding (Figs. 22 through 24). All three methods were used in placing the epoxy resin mortars on the Ulatis Creek Bridge.

Since paving, the Ulatis Creek Bridge has been in service approximately five months. All test pavements have been reported as performing satisfactorily in this early traffic period.

New Jersey Turnpike Bridge

An orthotropic steel deck bridge section was recently constructed on the Route 161 exit of the New Jersey Turnpike near Elizabeth. This bridge has a $^3/_8$-in. thick deck plate supported by bar stiffeners arranged in a grid pattern 14 in. on centers. Two types of surfacing materials were used, each on half the surface of the bridge; one type was shop applied and the other field applied (51, 52).

Figure 21. Method of applying pure resin prime coat to steel deck plate.

Figure 22. Hand-troweling method of finishing epoxy resin mortars.

Figure 23. Power-troweling method of finishing epoxy resin mortars.

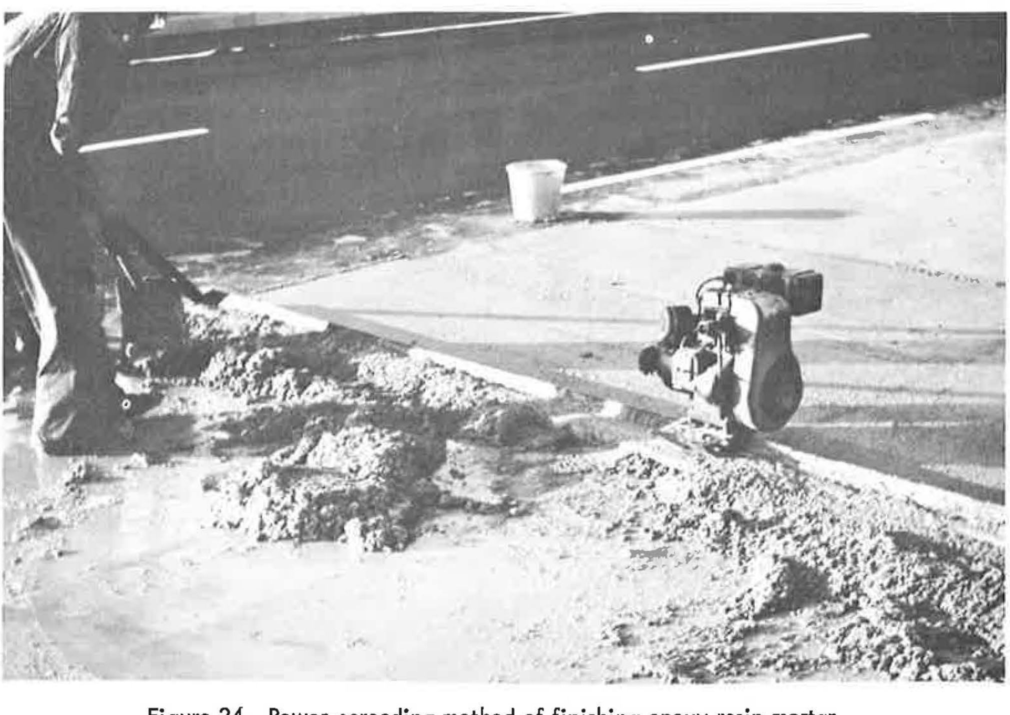

Figure 24. Power-screeding method of finishing epoxy resin mortar.

Figure 25. Tack welding expanded metal mesh to deck plate over field joint.

Figure 26. Pouring the epoxy-grit slurry over expanded steel mesh.

The shop-applied surfacing system, Realgrit, is a proprietary system (Reliance Steel Co.). The system involves tack welding an expanded steel mesh to the top of the steel deck and then coating the surface with $\frac{1}{4}$ to $\frac{3}{8}$-in. epoxy grit slurry. The grit is carborundum. Additional carbordundum grit is sprinkled on the surface of the slurry to improve skid resistance. The coating is heat cured at 250 F; it is applied to within

Figure 27. Sprinkling carborundum grit over surface of freshly poured epoxy-grit slurry.

Figure 28. Applying polyester prime coat to steel deck.

a few inches of the edge of the deck panels to allow for joining in the field. The joints are finished by tacking on a short section of expanded metal, applying the slurry, and curing by covering the joint with sand heated to 300 F. The coating of the joint in the field is shown in Figures 25 to 27.

The field-applied system (Figs. 28 and 29) was Cybond (American Cyanamid Co.), a polyester resin choked with a 12 to 30 mesh silica sand. After sandblasting the deck plate, a resin primer was applied with a bristle broom. After allowing approximately one hour drying time, the polyester resin was poured and spread over the surface at a

Figure 29. Brushing off excess grit from completed polyester-grit mortar on New Jersey Turnpike Bridge.

rate of approximately 5.7 gal/100 ft^2 to give a $\frac{1}{8}$-in. thickness. The sand was sprinkled on the surface at a rate of 2 to 3 lb/ft^2 resulting in a finished thickness of $\frac{1}{4}$ to $\frac{3}{8}$ in. The excess sand was broomed off the surface. The bridge has only recently been opened to traffic and as yet no comments are available about its performance.

Experimental Programs

Some of the bridges placed in service are basically experimental and are being used to evaluate and compare various paving materials under field conditions. In many cases, field installations were preceded by or are concurrent with laboratory studies. The purpose of the programs is to evaluate the materials before field application and to establish some correlation between laboratory and field performance. In other instances, paving material suppliers are conducting some limited tests of their own to determine the applicability of their products as suitable steel deck paving materials.

Prime Coats and Sealants.—Before the installation of the Troy Test Bridge, a laboratory program was carried out to evaluate a number of materials as a deck primer or sealer (49). Several materials, including various types of epoxies, bituminous coatings, and latex materials were evaluated. The materials were applied to metallized steel plates and examined as to chemical resistance, thermal shock, impact and bending stresses. Their permeability was also studied. The general conclusions of these tests showed that (a) only the bituminous sealants were permeable; (b) none of the materials were affected by the thermal shock test; (c) at low temperatures, the epoxy materials were generally better under impact; (d) no material was completely unaffected by the severe chemical tests but the coal tar epoxies were generally superior; and (e) all but one of the epoxy materials withstood the bending tests.

A somewhat similar but less comprehensive study of two primer materials was carried out in conjunction with the Port Mann Bridge in Vancouver (35). Steel plates primed with a red lead epoxy paint were coated with a straight epoxy and a coal tar epoxy sealant and examined for impact and chipping resistance at both room temperature and approximately 10 F. Before testing, the specimens were subjected to 15 cycles of freezing and thawing. Results indicated that neither material was superior to the other, and under the conditions imposed, both materials exhibited excellent adhesion and impact resistance and would be suitable for field use.

A portion of a program on various paving systems carried out by the California Division of Bay Toll Crossings in conjunction with the San Mateo-Hayward Bridge was also concerned with evaluating different prime coats and bond coats (37). Steel plates were sandblasted clean and then the following material combinations were applied:

1. Corrosion protection: (a) hot-sprayed zinc metallizing (5 mil minimum), and (b) zinc-filled inorganic coating (3.5 mil minimum).
2. Bond Coat: (a) coal tar epoxy (10 mil minimum) with fine sand sprinkled on surface, and (b) no bond coat.
3. Tack coat: (a) liquid asphalt applied cold, and (b) Epon asphalt applied hot.

A $1\frac{1}{2}$-in. wearing surface of plain asphalt-concrete and Epon asphalt-concrete was applied to all of the above combinations and tested for bond and shear strength. In the bond tests, a core was cut out with an area of 5.04 sq in. (2 specimens had 2.40-sq in. areas), and a test cap was cemented to the top surface of the wearing course for pulloff. The shear specimens were made by cementing steel plates 9 in. long (the test specimens were 4 in. wide) to the wearing course and making a saw cut behind the plate. The plates were then mounted in a jig to keep the plates parallel while they were pulled apart in a universal testing machine. Slow and fast shear tests were run; the slow tests at a shear rate of 0.002 in./min and the fast tests at a rate sufficient to cause failure within two seconds. The general conclusions from these tests were as follows:

1. Bond tests: (a) metallic zinc prime coat had poor bond when applied in more than one coat; (b) with few exceptions, the bond strength was less than 100 psi with considerable scatter in the results; and (c) most bond failures occurred in or adjacent to the tack coat.
2. Shear tests: (a) The fast shear tests generally gave values two or more times higher than the slow shear tests; (b) The fast shear strengths with few exceptions were below 100 psi with considerable scatter; (c) the inorganic zinc prime coat had poor bond to the steel plate; (d) the failure location of the fast shear tests varied considerably; and (e) the predominant failure location in the slow shear test was in or adjacent to the tack coat.

These results were somewhat confirmed by selective field tests run by the Illinois Highway Department on the Troy Test Bridge (53). Prior to removal, bond and percolation tests were made between the old pavements and the deck plate. Pulloff tests indicated low bond strength between the pavement and the surfacing. The epoxy prime coat was still tightly bonded to the steel plate but a plane of low bond strength existed at the tack coat between the prime coat and the surfacing.

Percolation tests were run by coring holes out of the pavement and sealing the surface of the holes with metal cylinders. Water was then poured into the cylinders to

TABLE 6

SUMMARY OF FATIGUE TESTS

Wearing Surface	Binder Material	Cycles First Crack	Total Cycles	Remarks
Resin-mortar	8.5% Miradon	227,000	714,000	Five different cracks, incipient bond failure, $\frac{1}{2}$-in. plate
Epoxy concrete	6% coal tar epoxy	107,000	144,000	Three different cracks, bond failure
Epoxy resin mortar	11.25% epoxy resin	—	1,057,000	No cracks
Epoxy resin mortar	Content unknown	—	1,000,000	No cracks
Epoxy resin mortar	11.5% epoxy resin	47,000	300,000	Four different cracks, bond failure
Epoxy resin mortar	11.25% coal tar epoxy	—	1,000,000	No cracks
Asphalt-concrete	6% rubberized bitumer	3,200	120,000	Five different cracks, bond failure
Asphalt-concrete	Standard asphalt	6,875	70,624	Cracks, bond failure
Epon asphalt	Epon asphalt	534,200	970,125	Small cracks, no failure in some plates

see if water would percolate through the interface between the epoxy prime coat and the bottom of the asphalt-concrete leveling course. Despite the low bond strength, the percolation tests indicated a very low percolation rate at the interface between the prime coat and the pavement.

Paving Materials. — In the program carried out before the Troy Test Bridge, effort was devoted to examining various sand-asphalt and asphaltic-concrete mixes (49). In addition to the standard mixes, various mix designs containing additions of long and short asbestos fibers, liquid latex, rubber powder, rubber pellets, and combinations of additives were evaluated. These various asphalt mixes were subjected to tests to determine stability, permeability, impact resistance, and resistance to repeated flexing at low temperatures. The shear strength between the various sand asphalt mixes and prime coat materials was also investigated. There was a significant amount of scatter in the test results, making any conclusions difficult. Two factors which were apparent, however, were that all of the additives used in the asphalt mixes increased the impact resistance and that the shear tests showed consistently low values with the predominant failure location, for specimens using epoxy prime coats, in the tack coat.

The Division of Bay Toll Crossings conducted a number of fatigue tests (Table 6) on candidate wearing surfaces attached to steel plates (37). The steel plates were 4 in. wide by 18 in. long and $5/8$ in. thick. The plates were sandblasted and coated with a metallic zinc primer before surfacing. All surfacings were applied $1\frac{1}{2}$ in. thick. The plates were supported at the ends over a 15-in. span and loaded with a center point load sufficient to produce an alternating bending stress in the steel plate from 0 to 10,000 psi with the wearing surface on the tension side of the plate.

The specimens were run at room temperature at a testing speed of 5 cpm. The first crack was determined electronically while subsequent cracking or other damage was determined visually. The test was stopped when the cracking was considered severe.

Another research program, concerned with determining the physical and structural characteristics of a number of potentially suitable wearing surface materials, is being conducted by the authors of this report. The materials are resin-mortar mixtures using such binders as coal-tar epoxy, oil extended epoxy, polyamide modified epoxy, polyester and silicone rubber and cement latex. The properties being investigated are fatigue resistance, flexural strength, modulus of elasticity, expansion coefficient, bond strength, abrasion resistance and high temperature stability. In practically all cases, the properties are being determined at temperatures of 0, 77, and 140 F and for two or more sand-resin ratios. Where applicable, the properties are being compared with those of Gussasphalt and asphalt-concrete.

CONCLUSION

The most common European steel deck paving systems and their performances have been described. A number of research programs which led to their development have been outlined. Also discussed were the paving systems used on the American steel deck bridges with some interim results of the paving material research programs under way.

In general, many of the paving problems associated with the early development of orthotropic bridges in Europe have been solved. Although none of the present systems are considered perfect, satisfactory results are being obtained. The stabilized Mastix system has proved quite effective and it is the most permanent system used so far in Germany, especially when used with the zig-zag anchor bars. These anchor bars help distribute the wheel loads over a greater area, reducing deck deflections and thereby reducing the flexibility requirements of the Mastix. The performance of this system without the anchor bars will receive considerable attention. The layered paving system, which has been used most often, experienced some early difficulties but has been developed into a reliable system. The use of the Mastix Isolierung has proved to be effective both in protecting the steel deck plate from corrosion and in absorbing thermal shear stresses that develop between the steel deck and the surface courses. A complete solution to cracking has not been found, but these cracks do not greatly impair the performance of the deck pavement, and with some maintenance, the pavement may

be expected to last ten years or more. The British approach of using a single paving layer, if successful, would be more desirable than the use of two layers as it would reduce the labor involved as well as the dead weight.

Pavement development for orthotropic steel bridge decks in this country is just beginning and is likely to be different from European practices. The lack of facilities and experience to produce a poured mastic asphalt is almost certain to rule out its use. The suitability of our standard rolled sand-asphalts and asphaltic-concretes without some modification to increase the flexibility has yet to be determined. With the effort that is being devoted to developing thin combination pavements of resinous materials, these materials may well prove ultimately to be the most suitable. Provided the synthetic binder materials are found to be sufficiently durable, the disadvantage of higher initial cost could easily be outweighed by such advantages as significantly reduced dead weight and greater ease of installation and repair.

REFERENCES

1. Forschungsgesellshaft für des Strassenwesen, E. V. (Road Research Society). Preliminary Specifications for Bituminous Pavements on Lightweight Steel Bridges. 1961.
2. Interview, representatives of German Fed. Transport Ministry: Dr. W. Klingenberg (now retired), Dr. Thul, Mr. Stohl, and Mr. Martain.
3. German Patent No. 1,029, 851, Jan. 17, 1963.
4. Interview, Dr. H. Kohler, Pres., Teerban, Inc., paving firm, Essen, Germany.
5. Interview, representatives of Construction Dept., City of Mannheim, Germany: Dr. Borely and Mr. Fuchts.
6. Mersch, K. Asphalt Pavements in Bridge Construction. Bau u Bauindustrie, Sept., 1961.
7. Seegers, K. H. Bitumen and Tar in Lightweight Pavements for Steel Highway Bridges. Bitumen, Teer, Asphalt, u. Peche, Vol. 4, No. 6, 1953.
8. Interview and private communications, Dr. K. Mersch, Pres., Kölner Asphalt-Fabrik, paving firm, Cologne, Germany.
9. Seegers, K. H. Asphalt Paving of Steel Highway Bridges. Bitumen, No. 9/10, 1952.
10. Mersch, K. The Gussasphalt Pavement of the Cologne-Mülheim Bridge. Bitumen, No. 9/10, 1952.
11. Interview and private communication, Dr. H. Henneke, Director, Beratungstelle Für Asphaltverwendung E. V. (Council for Asphalt Utilization), Braunschweig, Germany.
12. Ehnert. The Pavement on the New Autobahn Bridge at Hedemunden. Bitumen, No. 9/10, 1952.
13. Design Manual for Orthotropic Steel Plate Deck Bridges. Amer. Inst. of Steel Construction, New York, 1963.
14. Coatings and Bituminous Pavements as Lightweight Pavements for Steel Bridges. Bitumen, No. 8, 1962.
15. Unpublished data from Road Res. Lab., Dept. of Sci. and Indust. Res., Harmondsworth-West Drayton, Middlesex, England.
16. Private communication, P. Jost, City Construction Dept., Basel, Switzerland.
17. Duric, N. Experiments and Construction of Pavement in the New Save Bridge Near Belgrade. Publ. No. 16, Inst. of Experimental Materials, Republic of Serbia, 1960.
18. Wahl, E. F. Bridge Decks. Strassen u. Tiefbau, No. 12, 1960.
19. The St. Alban Bridge in Basel. Brucke u. Strasse, No. 9, 1955.
20. Interview and private communication, Dr. K. Kunert, Machine Fabrik-Augsburg-Nunberg (M-A-N), construction firm, Gustavsburg, Germany.
21. Interviews and private communications, representatives of ESSO, Inc., Hamburg, Germany: Dr. W. Becker, Dr. Schmidt, and Mr. Bielenberg.
22. Interview and private communication, Dr. H. Mittmeyer, Director, ARBIT, organization representing German Bitumen Industry, Hamburg, Germany.

23. Beckman, K. Asphalt Paving of the Federal Autobahn, Southern Hamburg By-Pass and the Eidelstadt By-Pass in Hamburg. Bitumen, 1963.
24. The Fehmarnsund Bridge and Its Pavement. Bitumen, No. 7/8, 1963.
25. Interviews and private communications, representatives of the British Road Res. Lab., Harmondsworth-West Drayton, Middlesex, England: Dr. A. R. Lee, Mr. James, and Mr. Price.
26. Zube, Ernest. Use of Asphalt-Latex Emulsion in Thin Wearing Surfaces for Steel Bridges and Structures. Reprint, Proc., Thirty-Fourth Annual Meeting, Highway Research Board, Jan. 1955.
27. James, J. G., and Wilson, D. S. Roadway Surfacings on Steel and Aluminum Bridge Decks: Research and Progress in Great Britain. Road Res. Lab., Dept. of Sci. and Indust. Res., England.
28. Thiele, Max. Bituminous Pavements for Orthotropic Steel Plate Bridges. Strassen u. Tiefbau, No. 1, 1964.
29. Auberlen, R. The Approach to the North Bridge in Dusseldorf. Bitumen, No. 6, 1958.
30. Kirschmer, Otto. Gussasphalt Pavements on Steel Decks—Research and Test Methods. Bitumen, No. 1, 1954.
31. Kirschmer, Otto. Investigation of Gussasphalt Pavements on Steel Decks. Bitumen, No. 9/10, 1952.
32. Trott, J. J., and Wilson, D. S. The Development of Asphalt Surfacings for Steel Bridge Decking. presented at the Sixth Cong., Internat. Assoc, for Bridge and Structural Engineering, Stockholm, 1960.
33. Shields, E. J. Artistic Orthotropic Design Also Saves Money. Engineering News-Record, Nov. 21, 1963.
34. Private communication, A. L. Elliott, Chief Bridge Engineer, State Highway Dept., State of California, Sacramento.
35. Unpublished data on: Tests on Epoxy Coatings. prepared for CBA Engineering, Ltd., Vancouver, Canada.
36. Interviews and private communication, Messrs. Foley and Balala, Div. of San Francisco Bay Toll Crossings, San Francisco, Calif.
37. Unpublished data on: Laboratory Tests for Steel Deck Overlay San Mateo-Hayward Bridge.
38. Unpublished data on tests of sealing materials for steel plates, German Fed. Ministry of Transp., Bonn, Germany.
39. Ziegler, Heinz. Problems with Pavements on Orthotropic Deck Pavements. Bitumen, No. 5, Aug. 1964.
40. Henneke, Hubert. Thoughts About Gussasphalt Pavings on Bridges with Orthotropic Plates. Bitumen, No. 5, Aug. 1964.
41. Wichert, Paul. Asphalt Pavements on Three Steel Bridges with Orthotropic Plates. Bitumen, No. 5, Aug. 1964.
42. Meyer, H. The Autobahn 'Northern Cologne Bypass'. Bitumen, No. 6, Sept. 1965.
43. Preprint of Report by Herren Schemerber, Brunner and Mantler. The Bituminous Pavement on the Autobahn Bridge over the Donau (River) at Regensburg. To be published in technical bulletin from the Street Construction of Teerbau, Vol. 12, 1965.
44. Piechowsky, C. V. Construction News—The Gussasphalt on the Bascule Bridge over the Trave in Lübeck. Bitumen, No. 5, 1964.
45. Henneke, H. Further Developments with Asphalt Mastix Deck Construction on Orthotropic Plates as Applied on the Donau Bridge at Wörth-Pfaffer. Bitumen, Vol. 2, 1965.
46. James, J. G. Epoxy Resins as Binders for Roads and Bridge Surfacings—A Review of the Present Position, Roads and Road Construction. Vol. 41, No. 488, Aug. 1963.
47. Private communication, Prof. Dr. Ing. F. Leonhardt, partner, Leonhardt and Andrä, Stuttgart, Germany.
48. Steel Elevated Street Paved with Plastic. Brücke u. Strasse, Vol. 8, Aug. 1965.

49. Unpublished data on: Deck Plate Protection and Wearing Surface Studies. Submitted to Illinois and Missouri Highway Dept. by Sverdrup and Parcel and Assoc.
50. Private communication, Norman Hilton, Project Manager for C.B.A. Engineering, Ltd., Vancouver, B. C., Canada.
51. Private communications and unpublished data, A. J. Oudheusden, Assist. Manager, Appl. Res. Dept., Bethlehem Steel Co., Bethlehem, Pa.
52. Private communication and unpublished data, Messrs. Blanchard and McGinty, American Cyanamid Co., Stamford, Conn.
53. Private communication, officials of the Illinois Highway Dept.

Appendix

EUROPEAN ASPHALTIC TERMS

Asphalt binder. —A coarse open-grained mixture of bitumen, filler, sand, and crushed stone. A typical mix composition is as follows.

20-25 percent crushed stone, $^5/_{16}$ to $^1/_2$ in.
40-45 percent crushed stone, $^5/_{64}$ to $^5/_{16}$ in.
20-25 percent sand, 0 to $^1/_8$ in.
3-8 percent filler, 80 percent passing No. 140 sieve
4-6 percent bitumen, 60 to 70 penetration

The foregoing material is mixed and placed at temperatures of approximately 300 to 350 F. It is plant mixed, hauled to the job site in open-bed dump trucks, and machine laid. Rolling by steel- or pneumatic-tired rollers is necessary to compact the mix. The composition has high stability and contains 4 to 8 percent voids.

Asphalt Mastix. — A dense, hard, and impermeable mixture of bitumen, filler, and sand. The following is a typical mix composition.

30-55 percent sand, 0 to $^1/_8$ in.
25-35 percent filler, 80 percent passing No. 140 sieve
14-16 percent bitumen, 15 to 60 penetration

Mastix compositions are plant mixed and transported to the job site in special cookers. Because of the high filler contents, the bitumen and filler are often premixed and added in the mixing operation in chunk form. Plant mixing is done in a few minutes, but the Mastix must be further mixed in the portable cookers before use. Longer mixing times are required when high filler contents are used. The mixing temperature may be as high as 400 F, depending on the bitumen penetration. The Mastix is placed at temperatures of about 350 F. The mix is extremely fluid, and is often placed by hand.

Gussasphalt.—A hard, dense, and impermeable mixture of bitumen, filler, sand, and crushed stone. A typical mix composition would be the following.

10-15 percent crushed stone, $^5/_{16}$ to $^1/_2$ in.
10-20 percent crushed stone, $^3/_{16}$ to $^5/_{16}$ in.
10-20 percent crushed stone, $^1/_8$ to $^3/_{16}$ in.
20-30 percent sand, 0 to $^1/_8$ in.
20-30 percent filler, 80 percent passing No. 140 sieve
7-10 percent bitumen, 20 to 40 penetration

The bitumen portion often comprises mixtures of bitumens having various penetration values; it may contain 1 to 3 percent of Trinidad Epuree. (high mineral content, natural asphalt from Trinidad). Rubber powder may also be added in amounts of up to 5 percent by weight of the bitumen content.

Gussasphalt is generally plant mixed and transported to the job site in special cookers. The mixing times varies from $^1/_2$ to 6 hr, depending on the equipment and composition.

The placing temperature of gussasphalt is about 375 F. It may be hand placed, although special laying machines are now available.

Stone filler mastic asphalt. —A dense, impermeable mixture of aggregate, sand, filler, and bitumen. The following is a typical mix composition of the mastic:

15-25 percent passing No. 7 and retained on No. 25 sieve
10-30 percent passing No. 25 and retained on No. 72 sieve
10-25 percent passing No. 72 and retained on No. 200 sieve
40-45 percent filler passing No. 200 sieve
14-15 percent bitumen, 10 to 15 penetration

The foregoing mastic composition is often premixed and poured in blocks of approximately 50 lb for storage and transporting. Before laying and while remelting the mastic, $\frac{1}{4}$ to $\frac{3}{8}$-in. stone chips are incorporated into the mastic in a proportion to equal approximately 45 percent by weight of the mixture.

Flexural Tests of Paving Materials for Orthotropic Steel Plate Bridges

C. T. METCALF, Shell Development Co., Emeryville, California

As an aid in the selection of an appropriate wearing surface for the new orthotropic (steel-plate) bridge across San Francisco Bay at San Mateo, the Division of Bay Toll Crossings initiated a test program to investigate the properties of designated materials. The program required the preparation of special composite beam test specimens in which the paving material is compacted on top of a steel plate. Necessary beams for the testing program were prepared by special techniques and were ultimately tested for flexural fatigue at the California Materials and Research Laboratory.

A theoretical analysis of the behavior of composite beams in flexural fatigue reveals that relatively large tensile strains can be expected in the paving material during the fatigue test. This finding is based on properties of the paving materials calculated from the stiffness of the binders in the mixes at the designated rate of loading. The relatively large strains produced during the test provide an explanation for the short fatigue life of the asphalt-concrete mixes. An experimental material which was also included in the program showed a significantly longer fatigue life which is attributed to an improved strain tolerance of the material.

In a subsequent series of tests, actual strains measured showed reasonably good agreement with calculated strain values. This indicates that fairly reliable estimates of composite beam behavior can be accomplished through application of theoretical principles.

•CONSTRUCTION of a new orthotropic bridge across San Francisco Bay at San Mateo is to be completed in 1967. The orthotropic bridge (from the words "orthogonal" and "anisotropic") is a recently developed type of steel plate deck construction in which the steel deck is welded to the structural supports (1). In the construction of such a bridge, a wearing surface is provided to protect the deck from corrosion and to offer a smooth riding surface. Because the orthotropic structure is such a recent development, no extensive background of service has been established in the United States which can serve to define the properties needed in the wearing surface. Performance records of orthotropic pavements in North America are relatively scarce (2). The Port Mann Bridge at Vancouver, B. C., which opened in 1964 and is paved with 2 in. of asphalt-concrete, is the only major orthotropic structure to be completed to date (3). Other large orthotropic spans are soon to be built, however. These include the new Poplar Street Bridge across the Mississippi River at St. Louis, Mo. (4), scheduled for completion during summer 1966.

In the United States, attention has centered on the use of asphalt-concrete, or some modification of it, as a paving material for orthotropic bridges. This differs from European practice in which the emphasis is placed on the use of asphalt mastics for

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bridge pavements. Problems involved in the use of mastics are different from those with asphalt-concrete; consequently, European experience cannot be applied directly to practice in this country. A knowledge of foreign projects, however, provides a useful background against which an investigation of orthotropic pavements can be initiated.

In England, the Road Research Laboratory began a series of field trials in 1949 (5) to investigate pavements for steel plate decks. It was found that $1\frac{1}{2}$ in. of a single-course, stone-filled mastic gave satisfactory performance for about five years under heavy traffic. Construction practices in Holland (6) are similar to those in Germany (7, 8, 9). Both countries are influenced by the extensive use of hand-troweled mastics for bridge applications. German experience, which is the most extensive on record, indicates that mastics exposed to heavy traffic tend to show some plastic instability. In an attempt to overcome this problem, the deck plates of new bridges frequently have steel ribs welded in place while the bridge is under construction. Although expensive, it is hoped that this measure will effectively reduce lateral displacement in the mastic surface(9).

After careful consideration of information available from all sources, engineers of the Division of Bay Toll Crossings decided to initiate their own testing program to evaluate some of the possible paving materials for the San Mateo Bridge. Included are flexural fatigue tests performed on the special composite beam test specimens. The beams represent full-scale reproductions of the orthotropic deck between the individual supporting ribs. The materials in the first series of tests included conventional asphalt-concrete and an experimental paving material containing a thermal-setting, asphalt-modified resin binder. The experimental binder is not considered a commercial material at present. Other variables in the program included the type of corrosion resistance (sprayed metallic zinc, or zinc-filled coating) and the use of a bond coat (tar-modified epoxy resin). Actual testing of the composite beams was undertaken in the California Division of Highways Materials and Research Laboratory at Sacramento.

A reasonable estimate of the behavior of the two paving materials in the flexural fatigue test requires an understanding of the theoretical behavior of composite beams and a substitution of known (or estimated) properties of the materials into theoretical equations describing the behavior. Composite beams are made of separate materials combined to act together as a single or composite unit. Methods are readily available to investigate stress-strain relationships of such beams. Methods such as these were previously employed by Nijboer (10) in his equivalent slab analysis. Use of these theoretical tools contributes to a better understanding of the requirements to be met by the paving material and the properties to be attained for satisfactory performance.

PREPARATION OF COMPOSITE BEAM TEST SPECIMENS

Mineral aggregates used in the preparation of composite beam test specimens are representative of locally available materials in the San Francisco Bay area that can be used in conventional asphalt-concrete paving mixes.

Materials

Aggregates were combined to provide a grading to meet the California specification for $\frac{3}{8}$-in. asphalt-concrete (11). Gradings of individual materials are given in Table 1. The aggregate gradation (Fig. 1) was used in the preparation of the experimental paving mix as well as the asphalt-concrete mix.

Mix Properties. — To provide a thorough background for the design of the asphalt-concrete mixes, design data were obtained by use of both the Marshall and Hveem methods (12). Mix design curves produced by these two methods are shown in Figures 2 and 3.

In the selection of the asphalt content for the mixes, information from both the Marshall and Hveem curves was taken into consideration. On this basis a single content was selected at which the mix properties would meet the requirements of both methods of design as closely as possible. The selected content of the mix is 6.0

TABLE 1

GRADATIONS OF AGGREGATES IN MIXES

Item	Cumulative Percent Passing Sieve No.							
	$^3/_8$-in.	4	8	16	30	50	100	200
Aggregates								
$^1/_4$ × dust	100	81	56	40	30	22	13	7
$^3/_{16}$-in. and Bloomquist sand		100	72	47	30	18	8	3
Combined grading 75% $^1/_4$ × dust 19% $^3/_{16}$ and sand 6% Dolomark	100	85	62	45	34	26	17	12
Specifications (11) California $^3/_8$ in. asphalt-concrete	95-100	65-85	50-70		28-40			7-14

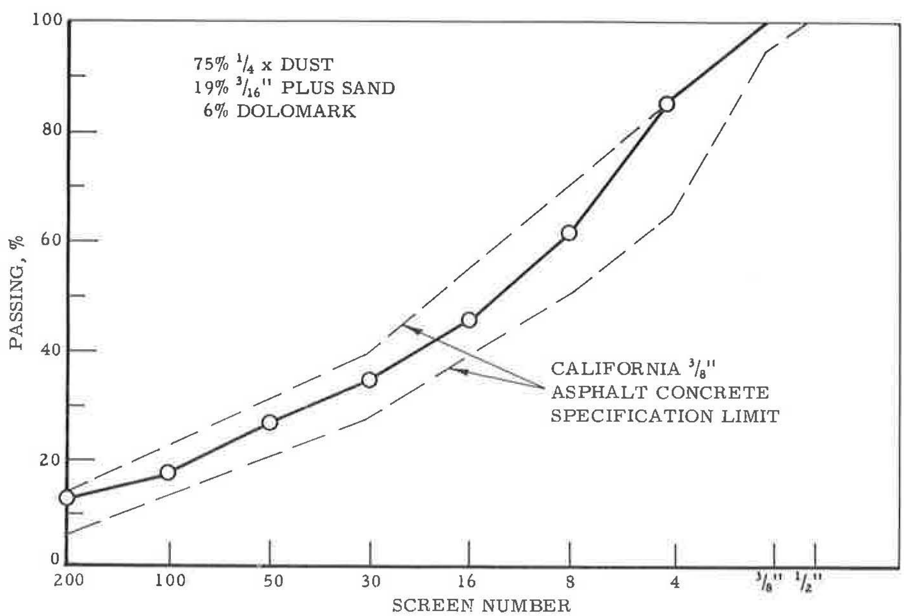


Figure 1. Gradation of combined aggregates in paving mixes.

percent based on aggregate (5.66% by wt total mix). Properties of the mix containing regular 85/100 asphalt (penetration 92, softening point 114) are given in Table 2.

On the basis of laboratory testing of the experimental paving mix, a binder content of 7.5 percent based on aggregate (6.97% by wt total mix) was selected as the design content. This resulted from the use of a reduced compactive effort in the preparation of the laboratory specimens. The reduced effort is equivalent to 20-blow Marshall hammer compaction and represents the compaction that might reasonably be expected to be produced by a roller during field construction. Marshall stability properties are given in Table 2.

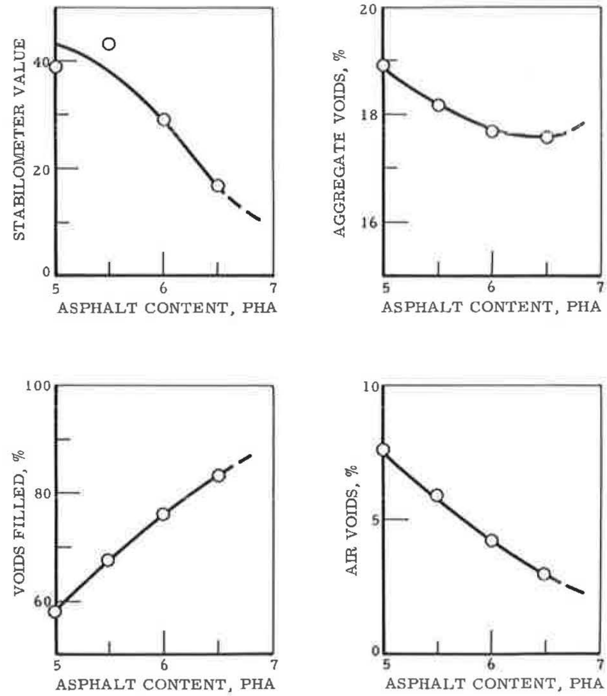


Figure 2. Hveem method design curves.

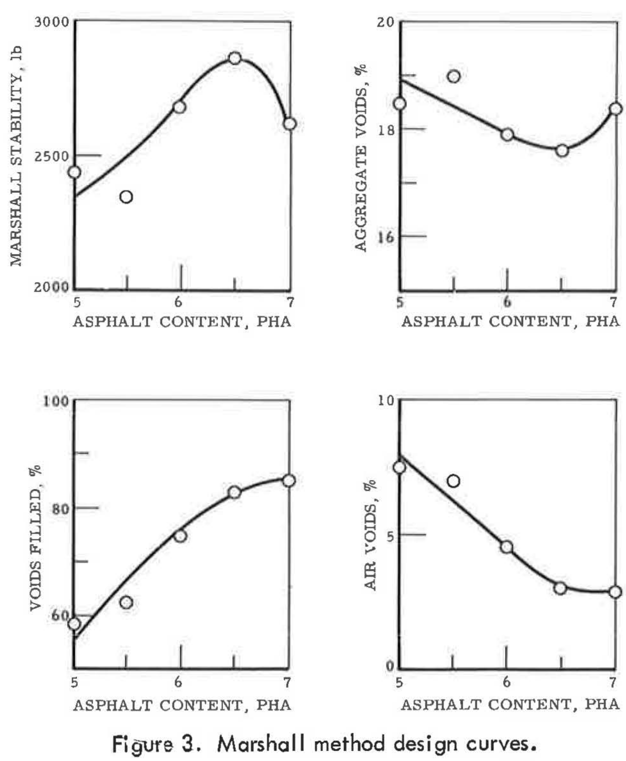


Figure 3. Marshall method design curves.

TABLE 2

DESIGN PROPERTIES OF PAVING MIXES

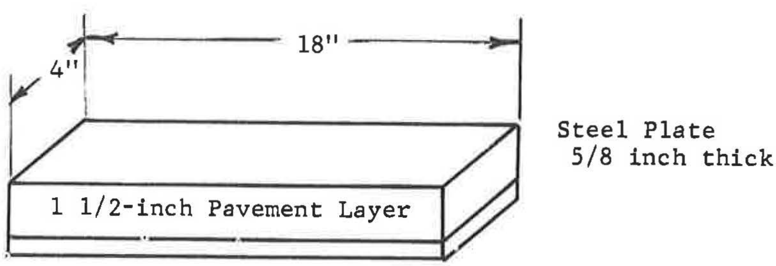
Method	Asphalt-Concrete	Experimental Paving Material[a]
Marshall		
Marshall stability (lb)	2,685	10,625[b]
Flow value (0.01 in.)	11	9
Bulk specific gravity	2.377	2.360
Voids in mineral aggregate (%)	17.9	19.5
Percent voids filled	75.0	85.0
Air voids (%)	4.5	2.9
Hveem		
Stabilometer value	29	
Bulk specific gravity	2.381	
Voids in mineral aggregate (%)	17.7	
Percent voids filled	76.0	
Air voids (%)	4.2	

[a]Properties of experimental binder: tensile strength at 77° F (psi), 233; elongation at break (%), 234; initial stiffness (psi), 149; and glass transition temperature (° F), 9

[b]Compaction equal to 20 blows of Marshall hammer.

Composite Beam Specimens

The testing program required the preparation of 40 composite beam test specimens 4 in. wide and 18 in. long. Preparation of the beams involves compaction of 1½ in. of the paving material on top of a ⅝-in. steel plate (ASTM A 36 structural steel).



In the preparation of the composite beam test specimens there are several variables involved in addition to the type of paving material placed on the plate, including (a) the type of corrosion protection (two types of inorganic zinc paint), (b) the type of bond coat on top of the corrosion protection (tar-modified epoxy resin), and (c) the type of tack coat (asphalt emulsion SS-1h or experimental binder).

Compaction of Mixes. —Because of the relatively large size of the test specimens, no standard procedure could be followed for compaction of the materials, and it was necessary to develop a special technique for the purpose. Of greatest importance in the development of this technique was the necessity for the method to produce specimens having both uniform and adequate compaction. Compaction by the method should be capable of producing materials that would be reasonably representative of materials used in the field.

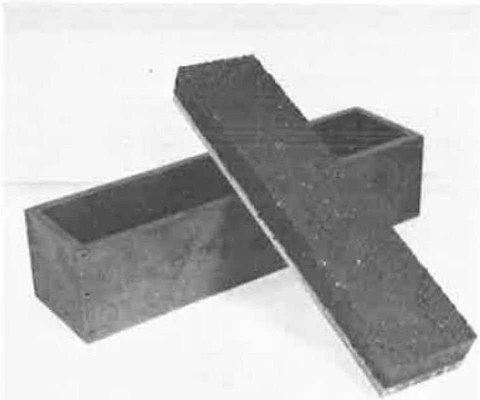


Figure 4. Completed composite beam specimen on removal from mold.

To meet these requirements within the short time available, compaction with the pneumatic hammer was investigated. This instrument is a Thor size 83 hammer which operates at a 60-psi line air pressure, and is equipped with a timing device (Appendix A). For compaction of the beams, the air hammer was equipped with a square compaction foot. After several trials, a suitable technique was established by which uniformly well-compacted beams could be produced with the air hammer. The method involves compaction of the beam in segments. During development of the method, it was learned that compactive resistance of the material in the mold varies according to the longitudinal position along the beam. The material in the end of the mold is more difficult to compact than material at the center; consequently the compactive effort and sequence of compaction must compensate for the effect. Figure 4 shows the completed beam specimen after removal from the compaction mold. Appendix A contains the procedure followed in the compaction of the beams.

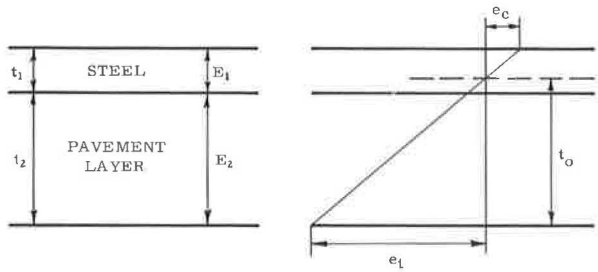
Properties of Materials Compacted on Beams.—During development of the compaction technique, uniformity of compaction was checked by sawing the trial beams into equal segments and then measuring the specific gravity of each segment. Unconfined compression tests at 140 F were also performed on the segments and Marshall stability values estimated from the unconfined compressive strengths. These estimations are based on a previously developed relationship between the two types of test (13). Tabel 3 indicates properties of segments taken from the center of the beams prepared by the same technique used in the preparation of the actual test specimens. The specific gravity of the asphalt-concrete (2.362) is close to the specific gravity produced in the Marshall design specimens (2.377). However, the estimated Marshall stabilities are somewhat lower than the 2,685-lb stability of the design specimens. Although the

TABLE 3

PROPERTIES OF MATERIALS ON BEAMS (TEST TEMPERATURE: 140 F)

Material	Bulk Specific Gravity	Air Voids (%)	Unconf. Compr. Strength (psi)	Est.[a] Marshall Stability (lb)
Asphalt-concrete	2.362	4.9	116.5	1,945
Experimental paving material[b]	2.371	2.6	754	12,590

[a]From previously developed relationship with unconfined compression test (13)
[b]The higher binder content of the experimental mix (7.5 as compared to 6.0 percent based on aggregate) accounts for the lower air void content of the material. Difference in Marshall stabilities is a reflection difference between binders. Asphalt-concrete contains a conventional, thermal-plastic asphalt, whereas the experimental material contains a thermal setting resin binder of much greater strength.



E_1 = ELASTIC MODULUS OF THE STEEL PLATE
E_2 = ELASTIC MODULUS OF THE PAVEMENT LAYER
n = E_1/E_2 = MODULAR RATIO
t_1 = THICKNESS OF STEEL PLATE
t_2 = THICKNESS OF PAVEMENT LAYER
a = t_2/t_1 = THICKNESS RATIO
t_o = DISTANCE TO NEUTRAL AXIS FROM SURFACE OF PAVEMENT LAYER
b = WIDTH OF BEAM
I_o = MOMENT OF INERTIA OF TRANSFORMED BEAM
e_c = COMPRESSIVE STRAIN IN STEEL
e_t = TENSILE STRAIN IN PAVEMENT LAYER

Figure 5. Strain distribution in composition beam.

stabilities appear slightly low, it is believed that the asphalt-concrete on the composite beams is well compacted and provides realistic test specimens.

For the experimental paving material, the estimated Marshall stability in excess of 12,000 lb is slightly higher than the 10,625-lb stability of the Marshall design specimen. Specific gravity of the beam segments is also slightly higher than the 2.360 gravity of the Marshall specimens. Thus, as for the asphalt-concrete materials, the experimental paving material on the beams also appears to be well-compacted.

THEORETICAL BEHAVIOR OF COMPOSITE BEAMS

The flexural fatigue test to which the beams are subjected involves the application of a 700-lb load at the center of a 15-in. span at a rate of 5 cps. The beams are positioned so that the paving layer is loaded in tension. Figure 5 shows the dimensional and modular relationships for the beams (left), and the theoretical strain distribution (right). For this analysis, the conventional assumptions for beam behavior are employed: stress-strain proportionality, etc. Strain continuity (no slippage) across the interface is also assumed.

Location of Neutral Axis

Location of the neutral axis depends on the modular ratio (E_1/E_2) between the materials:

$$t_o = t_2 \frac{a^2 + n(1 + 2a)}{2a(a + n)} \tag{1}$$

The equation in Figure 6 shows that when the modular ratio is greater than 5.76, the neutral axis is located above the interface between the steel plate and the paving layer (the paving layer is in tension throughout its thickness).

For modular ratios greater than 100, the neutral axis is close to its limiting position at the center of the steel plate (t_o = 1.8125 in.). For this condition, the applied load is largely carried by the steel plate, and the pavement layer contributes little to the strength of the beam.

For a given position of the neutral axis, the moment of inertia of the transformed beam (i.e., the moment of inertia of a beam consisting of one material, but with the same strain distribution as the composite beam):

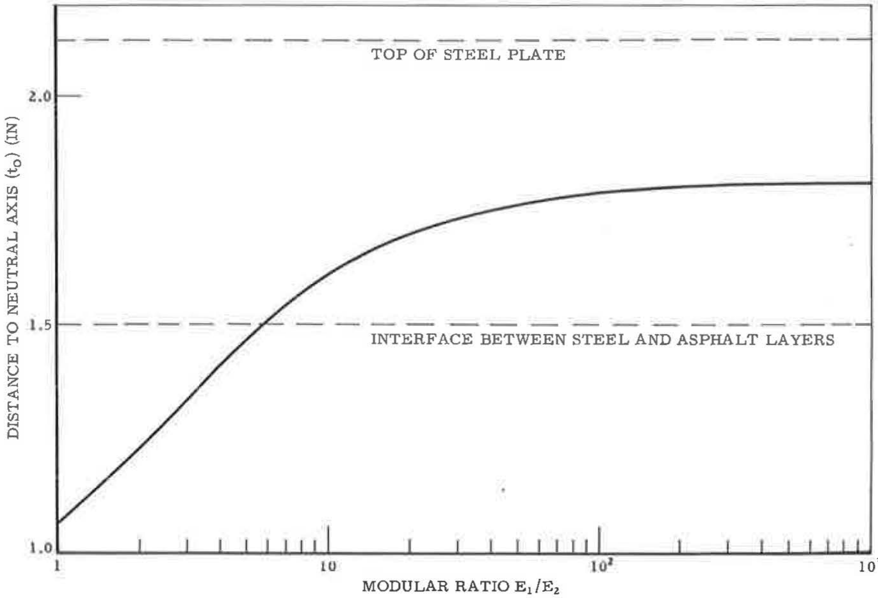


Figure 6. Relation of neutral axis to modular ratio.

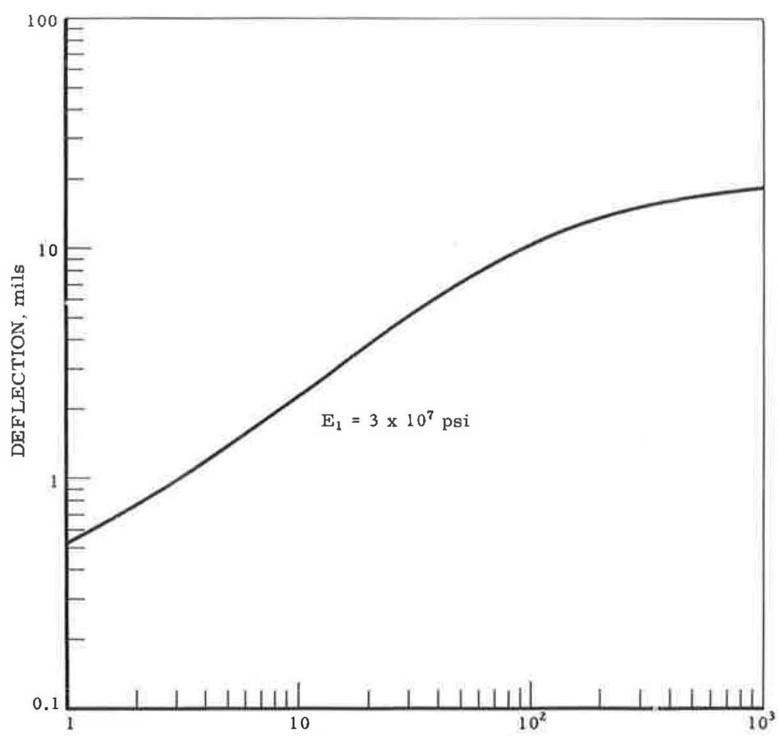


Figure 7. Relation of deflection to modular ratio.

$$I_o = \frac{1}{3} bt_2^3 [1 + n/a^3 (1 + 3a + 3a^2)] - bt_2 (n/a + 1) t_o^2 \quad (2)$$

Beam Deflection

The deflection (d) at the center of the beam under a 700-lb load on a 15-in. span:

$$d = \frac{Pl^3}{48 E_2 I_o} = \frac{700 (15)^3}{48 E_2 I_o} \quad (3)$$

where

P = load at center of beam, and
I = beam span.

The plot of deflection against modular ratio in Figure 7 indicates that for modular ratios in the range of 10 to 100, deflections are in the range of 2 to 10 mils. At higher modular ratios, deflections approach the limiting value of 20.16 mils ($E_1 = 3 \times 10^7$ psi).

Tensile Strain in the Pavement Layer

Maximum tensile strain at center of beam under load:

$$e_t = \frac{Mc}{E_2 I_o} = \frac{2625\, t_o}{E_2 I_o} \quad (4)$$

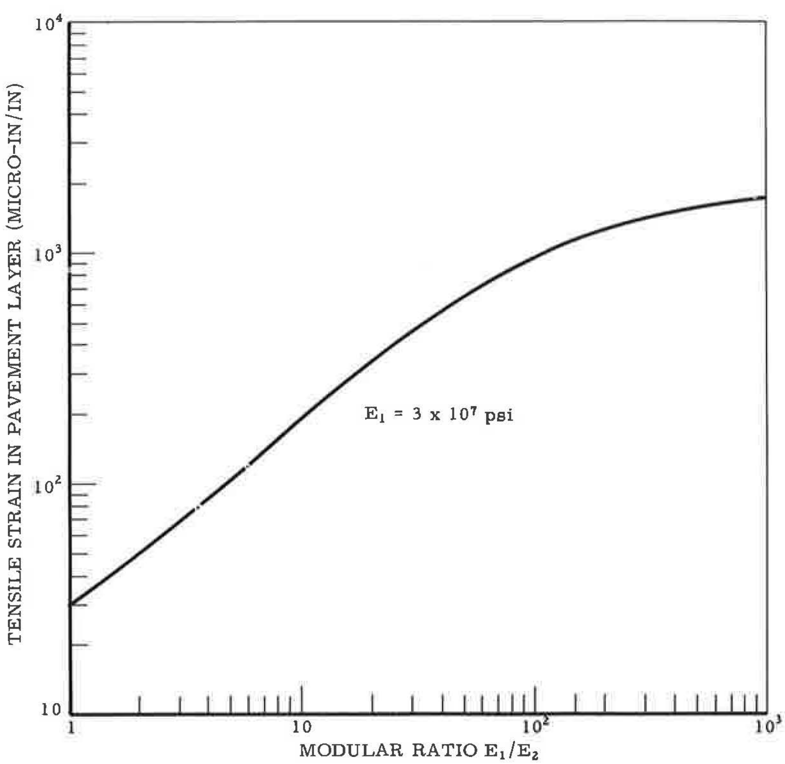


Figure 8. Relation of tensile strain to modular ratio.

where

M = resisting moment (700-lb load at center of 15-in. span), and
c = distance from neutral axis to tensile fiber (t_o).

Figure 8 shows that strain increases as modular ratio increases (i.e., modulus of the pavement layer decreases). A greater part of the applied load is carried by the steel plate as the pavement modulus is reduced, but strain in the pavement continues to increase.

Tensile strain is related to beam deflection:

$$e_t = (12\, t_o/1^2)\, d = (t_o/18.75)\, d \tag{5}$$

ESTIMATED PERFORMANCE OF COMPOSITE BEAMS IN FLEXURE

From the foregoing theoretical stress-strain relationships for composite beams, it is possible to estimate the reaction of the beam in flexure if the elastic modulus of the paving layer is known. Combining this modulus with the dimensions of the beam and the load applied in the flexural fatigue test permits tensile strain in the paving layer to be calculated. Calculated strains can then be compared with appropriate fatigue data and the fatigue life estimated.

Because of a lack of specific information on the properties of the materials at the designated rate of loading in the fatigue test (5 cps), it is necessary to estimate these properties from available data.

Asphalt-Concrete

The stiffness of asphalt-concrete at 72 F and 5 cps indicated in Table 4 is estimated from the relationship between the stiffness of an asphalt/aggregate mix and the stiffness of the asphalt in Figure 9 (Heukelom and Klomp, 14). Asphalt stiffness is calculated from the nomograph (15) in Figure 10. The estimated stiffness of the mix of

TABLE 4

ESTIMATED PROPERTIES OF COMPOSITE BEAM SPECIMENS

Property	Estimate
Asphalt-Concrete	
Stiffness of asphalt[a]	ca 70 kg/cm^2
Stiffness of mix (Fig. 9) (E_2)	2×10^5 psi
Elastic modulus of steel (E_1)	3×10^7 psi
Modular ratio (E_1/E_2)	150
Beam deflection (Fig. 7)	12.0 mils
Tensile strain in asphalt (Fig. 8)	1,200 μin./in.
Number of load repetitions (Fig. 11)	less than 1,000
Experimental Paving Material	
Stiffness of mix (E_2)	ca 3.5×10^5 psi
Modular ratio	86
Beam deflection (Fig. 7)	9.5 mils
Tensile strain (Fig. 8)	900 μin./in.
Number of load repetitions (Fig. 11)	9×10^4

[a]From nomograph, Figure 10.

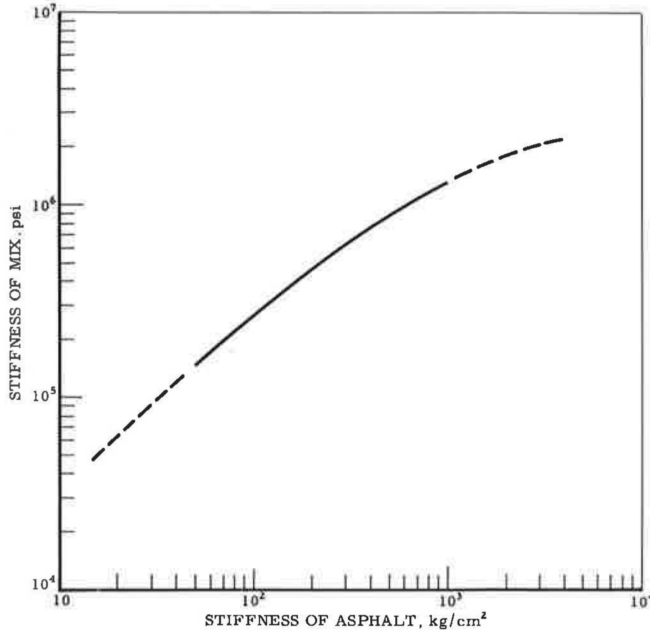


Figure 9. Relation of stiffness of mix to stiffness of asphalt.

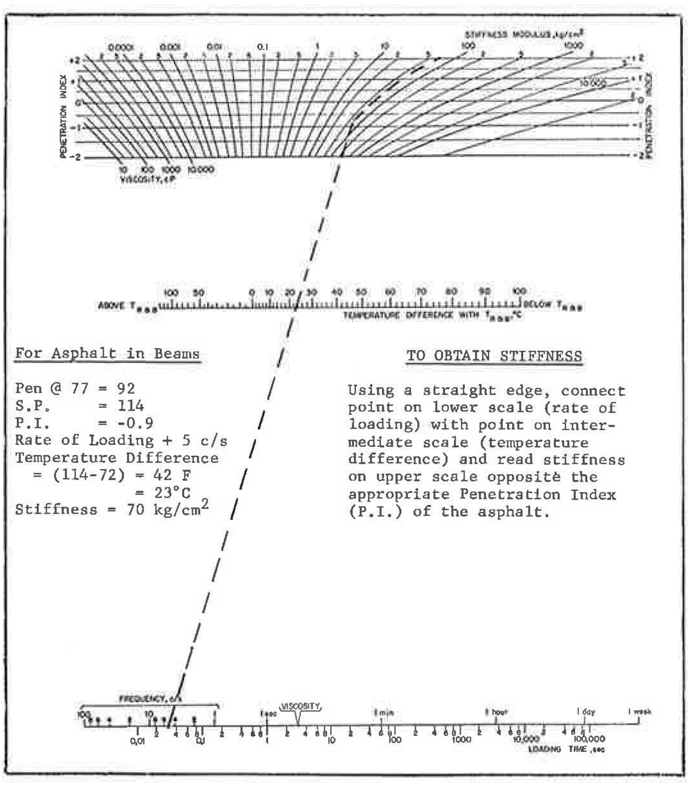


Figure 10. Nomograph for determining stiffness modulus of asphalts.

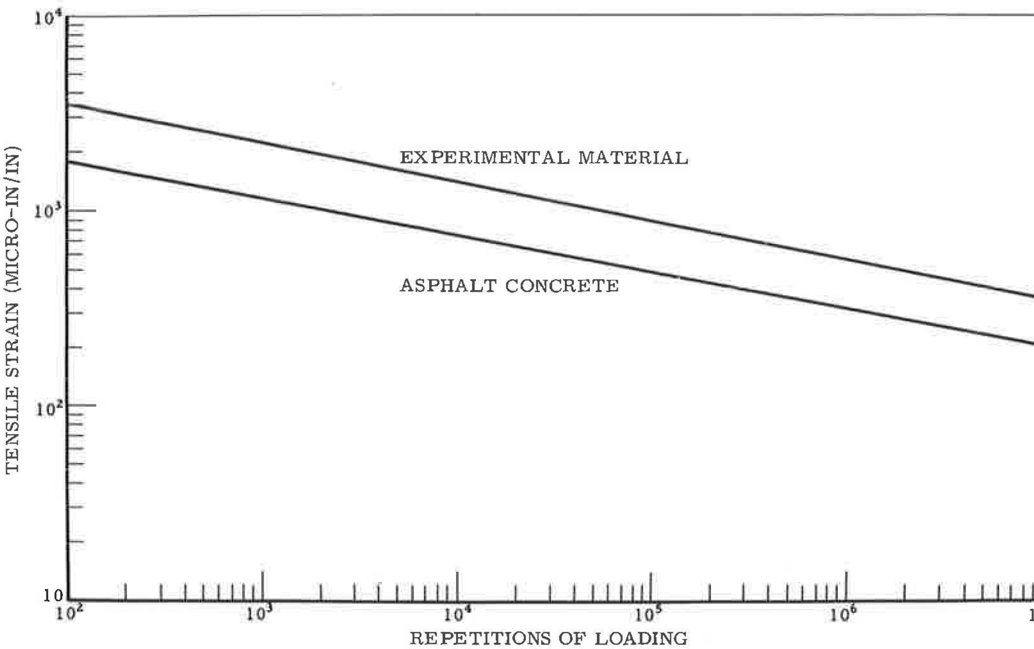


Figure 11. Estimated fatigue curves.

200,000 psi corresponds to an asphalt stiffness of 70 kg/cm^2, or approximately 1,000 psi (Fig. 10). For an assumed elastic modulus of the steel of 3×10^7 psi, the modular ratio between steel and asphalt-concrete is 150. Beam deflection at the center of the span corresponding to this modular ratio is 12 mils (Fig. 7). Tensile strain in the pavement layer is 1,200 μin./in. (Fig. 8). When this strain is compared to provisional fatigue data for asphalt-bound mixes employed by Heukelom and Klomp (16), it is evident that the asphalt-concrete cannot be expected to withstand strains this large without failure within a relatively short time. Fatigue life estimated from the curve in Figure 11 is less than 1,000 cycles (less than 3 min. in the fatigue test).

Experimental Material

Modulus of the experimental paving material in Table 4 is estimated from dynamic tests with a three-point bending apparatus. The modulus of 350,000 psi is obtained by extrapolation from tests on a material similar to that prepared for the composite beams. This modulus produces a modular ratio between steel and pavement layer equal to 86. Midspan deflection for this ratio is approximately 9.5 mils (Fig. 7). Tensile strain in the pavement layer is equal to 900 μin./in. (Fig. 8). Comparison of this strain with fatigue data (Fig. 11) indicates an estimated fatigue life of 90,000 to 100,000 repetitions of loading. This is significantly longer than the estimated life of the asphalt-concrete, and it might be expected that this experimental material would sustain approximately two magnitudes of loading more than asphalt-concrete in the fatigue test.

FLEXURAL TESTS ON COMPOSITE BEAMS

Tests performed on composite beam test specimens are described in a report by the California Division of Highways Materials and Research Department (17). After completion of the tests, results were obtained for comparison with data for the estimated performance of the beams.

Test Procedure

The test procedure for the flexural fatigue test as described by the San Francisco Division of Bay Toll Crossings (17) is as follows:

> The cyclic load for flexure test shall be sufficient to procedure a stress of 10,000 psi in the extreme fiber of the steel plate in contact with the overlay material. The cyclic flexure load shall be applied at a rate between 5 and 50 cycles per second. The test shall be continued until the overlay material shows signs of severe cracking, at which time the total number of cycles shall be recorded. Flexure test shall be stopped after 1,000,000 cycles. [Two beams of each type are tested in flexure.]

Test Results

Complete results of flexural fatigue tests on composite beams are given in Table 5. Results are separated according to the type of corrosion protection or bond coat applied to the steel plate before application of the pavement layer.

Results clearly show that there is a marked difference in the fatigue resistance of the experimental material and the conventional asphalt-concrete beams. As expected, all asphalt-concrete beams have a relatively brief fatigue life, and they failed soon after the start of the test. Maximum fatigue life of any asphalt-concrete specimen is only 13, 800 cycles, and one specimen failed after only 2, 100 cycles. This means that all asphalt-concrete specimens failed within an hour's time (2, 100 cycles is only 7 min). The experimental material, however, showed a somewhat better endurance. Six of the eight experimental beams sustained more than 500, 000 applications, and three beams exceeded more than 1, 000, 000 load applications; thus, the majority of these beams lasted longer than 27 hr (applications) in the fatigue test and some surpassed 55 hr.

Comparison of Results with Theoretical Estimates

The estimated fatigue resistance of the composite beams (Table 4) shows the same relative evaluation of the pavement materials as is demonstrated in the actual tests. Estimated fatigue lives, however, are somewhat less than the values established during testing. Theoretical fatigue life of asphalt-concrete is less than 1, 000 cycles, but

TABLE 5

RESULTS OF FLEXURAL FATIGUE TESTS ON COMPOSITE BEAMS

Corrosion Protection	Flexural Fatigue (cycles at first crack)			
	Asphalt-Concrete		Experimental Paving Material	
	1	2	1	2
Paint A bond coat[a]				
Tar epoxy	3, 800	3, 000	$<10^6$	167, 000
None	2, 100	11, 000	505, 000	213, 000
Paint B bond coat[a]				
Tar epoxy	4, 100	10, 000	$<10^6$	992, 000
None	13, 800	8, 000	$<10^6$	794, 000

[a]Inorganic zinc.

TABLE 6

STRAIN MEASUREMENTS ON COMPOSITE BEAMS[a]

Item	Tensile Strain in Asphalt Layer (μin./in.)		Compressive Strain in Steel Plate (μin./in.)
	Midspan[b]	Offset[c]	Offset
		Asphalt-Concrete	
Beam no. 1			
Gage 1	—	1,100	250
Gage 2	—	1,250	250
Beam no. 2			
Gage 1	1,450	1,350	240
		Experimental Paving Material	
Beam no. 3			
Gage 1	1,300	1,300	210
Gage 2	1,325	1,300	215
Beam no. 4			
Gage 1	1,400	1,300	200
		Steel Plate Alone	
		310	280

[a]Strains produced in composite beams by exerting 700-lb load on beam at center of 15-in. span at 5 cps.

[b]Strain gage at midspan located on line with applied load (location of maximum moment).

[c]Strain gage offset by 1½-in. from midspan to avoid damage by loading head.

test specimens withstood 2,100 to 13,800 repetitions of loading before cracks were detected. Estimated fatigue life for the experimental material is 90,000 to 100,000 cycles, and the majority of these specimens exceeded 500,000 cycles.

These differences between theory and practice are not entirely unreasonable however, when the difference between the criteria of failure in the two methods is recognized. Failure of the composite beam in the actual tests occurs when a crack is visibly evident in the material (17). Theoretical fatigue criteria, however, are based on the relation between force and deflection during the test. When a material is tested in a laboratory fatigue test, failure is said to occur when there is a sharp reduction in the force required to produce a given deflection in the test beam. Thus, it is possible for failure to occur within this concept without the formation of a crack. This consequence causes the theoretical criteria to be more restrictive, and some difference in fatigue life according to the two methods might be expected. In the actual tests, a composite beam might undergo many repetitions of loading after it is initially weakened (theoretical failure), but before the crack can be detected (actual failure). With this understanding, the differences between actual and estimated fatigue lives of the composite beams do not appear excessive.

STRAIN GAGE MEASUREMENTS

To check the accuracy of the theoretical calculations, an additional series of four beams was fitted with strain gages and the beams were tested in the flexural fatigue apparatus at the Materials and Research Laboratory. Results of these strain measurements in Table 6 provide an additional insight into the behavior of the composite beams.

Two beams with the experimental material and two with asphalt-concrete were used. Strain gages were positioned so that tensile strains could be measured at midspan and also at a position offset $1\frac{1}{2}$-in. from midspan (Fig. 12). The tensile offset gage is paired with a compression gage on the steel plate in a similar offset position. (It was necessary to offset the gage to remove it from a position under the loading head.) The use of tension and compression gages in pairs permits an investigation of the "effective" modular ratio between the steel and pavement material.

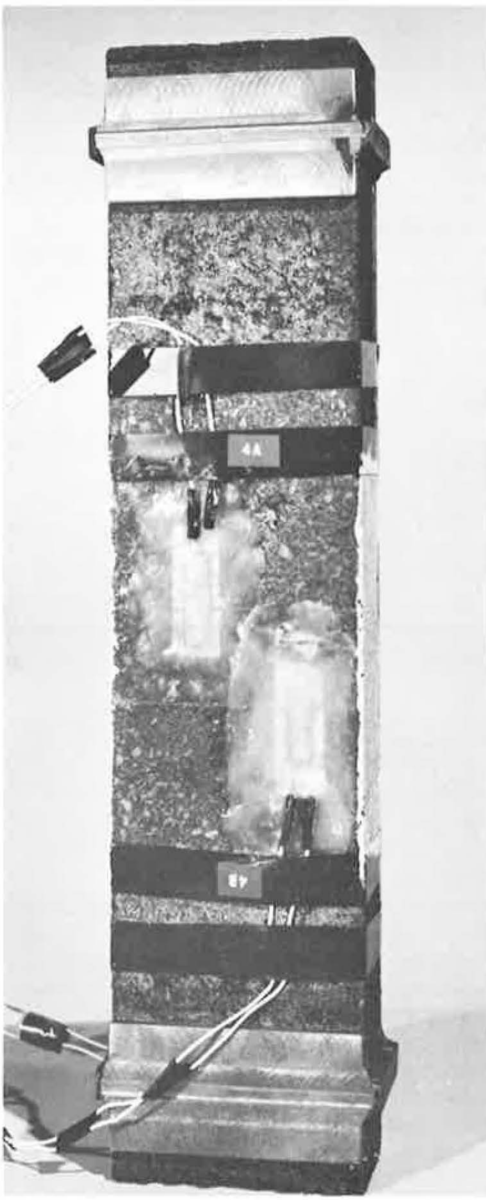


Figure 12. Composite beam specimen with gages attached for strain measurements.

Measured tensile strains in the beams (Table 6) are somewhat higher than anticipated on the basis of theoretical calculations. Tensile strains in the experimental material of 1,300 to 1,400 μin./in. are greater than the predicted value of 900 μin./in. This signifies that the effective modulus of the experimental material is lower than predicted. Measured strains of this magnitude correspond to elastic moduli in the range of 120,000 to 150,000 psi (Fig. 8). Thus the modulus is less than the theoretical value of 350,000 psi.

Tensile strain in the asphalt-concrete of 1,450 μin./in. is also higher than the calculated value of 1,200 μin./in., but the difference is not large. The effective modulus of the asphalt-concrete equal to this strain level is 100,000 psi compared to a calculated value of 200,000 psi. This is not an unreasonable difference and the theoretical estimate is in fairly good agreement with the measured value. Fatigue resistance at this fairly high measured strain level is not very great. On this basis, it appears doubtful that conventional asphalt-concrete could withstand any long exposure to these strains without cracking. The relatively short fatigue life of the asphalt-concrete beams is more easily understood when the strain level is considered.

The steel plate alone was tested in flexure to investigate the accuracy of the estimated modulus of the steel. The compressive strain of 280 μin./in. at the offset gage corresponds to a modulus of 30,000,000 psi which is the value used in calculations.

Compression gages on the beam specimens show that the addition of the pavement

layer is effective in reducing the strain in the steel. (This tends to substantiate the assumption of "composite" beam action.) If the pavement layer made no contribution to the strength of the beam, compressive strain in the steel would have its maximum value of 280 μin./in. by asphalt-concrete, and to 200 to 215 μin./in. by the experimental material.

INFLUENCE OF TEMPERATURE ON FLEXURAL FATIGUE

In respect to the test temperature of 72 F for the flexural fatigue tests, the question arises as to what significance this particular temperature has in relation to the results of the tests. For example, what would happen if the test were performed at some other temperature? Fortunately, there is sufficient information available from which a reasonable estimate can be made of the fatigue properties of the asphalt-concrete through a wide range of temperatures. (Similar information concerning the influence of temperature on the fatigue behavior of the experimental material is incomplete.)

Figure 13 shows the influence of test temperature on the tensile strain in the asphalt-concrete of the composite beam specimens. This graph is based on values of the moduli of asphalt-concrete calculated for different temperatures (and at the rate of loading in the flexure test of 5 cps). Similar information on the relation of temperature to modulus has been published in a report on layered-system analysis (18). Tensile strains corresponding to the respective moduli are shown in Figure 8. Figure 13 shows that as the temperature rises, tensile strains also rise and ultimately approach the limiting

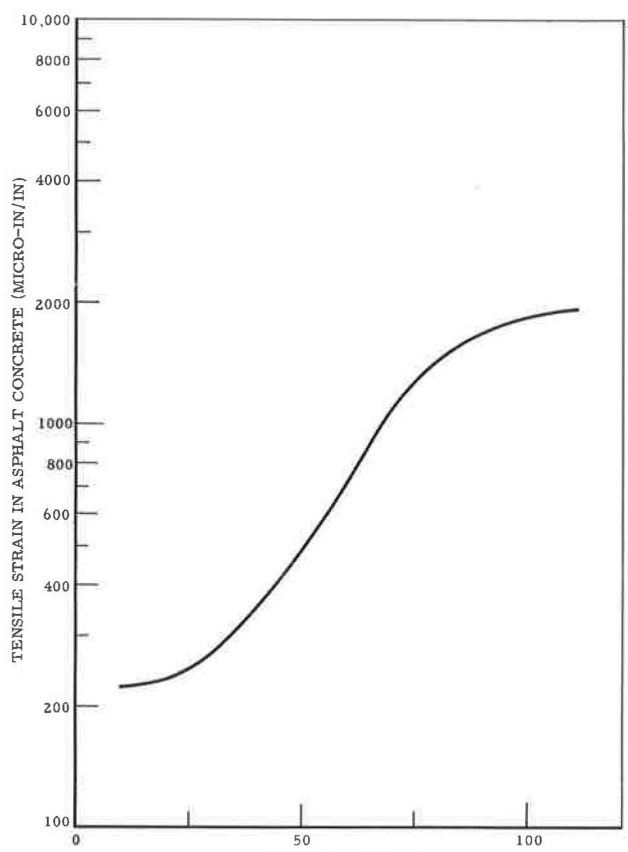


Figure 13. Effect of temperature on tensile strain in asphalt concrete in composite beams.

value of 1,950 μin./in. (strain value as the modulus of the asphalt-concrete approaches zero). As temperature goes down, tensile strains similarly grow smaller and approach a value slightly greater than 200 μin./in. at a temperature near 0 F. Thus, within this temperature range, strains can be expected to range between 200 and 1,950 μin./in. If all other factors were equal, composite beams might be expected to be most susceptible to fatigue failure when strains are highest; i.e., they would be most susceptible at high temperatures. This does not seem to be the case, however, for when the strain curve is compared to the fatigue resistance at different temperatures, the greatest susceptibility to fatigue in the composite beams does not occur at either high or low temperatures, but at an intermediate temperature,

The curve in Figure 14 is the result obtained by comparing the tensile strain at any given temperature with the corresponding fatigue lift (number of load repetitions) at the same temperature. This information is developed by superimposing the fatigue curves for asphalt-bound materials (16) on the curve (Fig. 13) representing the tensile strain in the asphalt layer. The tensile strain curve intersects the various fatigue curves at different temperatures. These intersections represent the fatigue life corresponding to the temperature at the intersection. When a plot is made of the intersections between the tensile strain curve and the various fatigue curves, the graph in Figure 14 is produced.

This curve (Fig. 14) emphasizes that the adopted testing temperature of 72 F is particularly critical for asphalt-concrete in this test. The curve passes through a minimum at a temperature near 70 F. At temperatures both higher and lower than

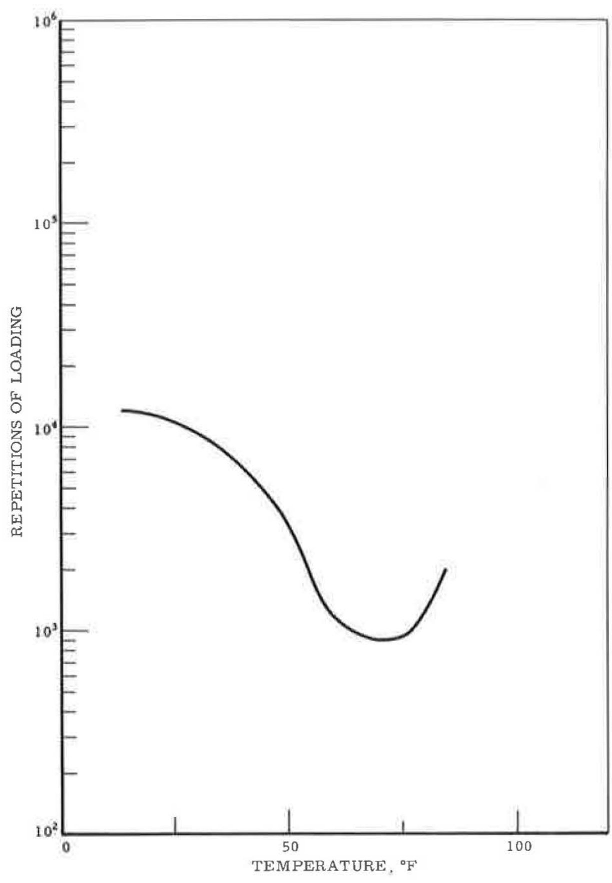


Figure 14. Effect of temperature on fatigue life on asphalt concrete composite beams.

this, the curve rises, reflecting the improved fatigue resistance. If the test had been accomplished at 50 F instead of 72 F, fatigue resistance of the beam would be expected to improve, but the improvement would be relatively small. Load repetitions would increase from an estimated low value of 900 up to approximately 3,500 repetitions. This small difference is probably within the experimental error of the test. By a reduction in temperature to 32 F, fatigue resistance is improved by approximately one magnitude. The improved resistance at temperatures below 70 F is largely the product of the reduction in tensile strain caused by the increase in the elastic modulus of the asphalt-concrete. Extremely low (brittle range) temperatures are not included. These are outside the temperature range for the bridge site.

Although 72 F appears to be a critical temperature for this test, use of another temperature would not have a significant influence on the evaluation of asphalt-concrete for use in composite beams. It would not tend to improve the fatigue behavior of asphalt-concrete to a level that might exceed 500,000 load repetitions. Approximately the greatest improvement that could be expected would be one magnitude or 10,000 repetitions.

SIGNIFICANCE OF ELASTIC PROPERTIES OF PAVING MATERIALS

Although the foregoing example demonstrates the reasons for which asphalt-concrete can be expected to have a relatively short fatigue life in the flexural test, it also points up the properties that a material should have for satisfactory performance. Generally, the elastic modulus should be large enough to keep strains within acceptable limits. If the elastic modulus is small (approximately 100,000 psi), the material must have the resilience to resist relatively large strains (approximately 1,400 to 1,500 μin./in.). If the elastic modulus is large enough to make a significant reduction in strain, the material must retain a similar ability to resist the smaller strains. The acceptable material must be able to tolerate the applied strain whether it is large or small.

Obtaining reliable information on the properties of the materials involved in an investigation of this type permits a comparison to be made of the severity of the loading conditions and the ability of the material to resist those conditions. Information such as this is invaluable in measuring the ability of a material to fill a need.

CONCLUSIONS

1. A method has been devised for the preparation of composite beam test specimens which are required for a testing program to evaluate paving materials for orthotropic bridges.
2. Paving materials compacted by the method are reasonably representative of the materials as they would be used in service on a bridge deck. Specific gravities of the compacted materials approach the values that can be accomplished in service.
3. Use of theoretical methods permits a reasonably accurate analysis of the flexural behavior of composite beam test specimens.
4. Theoretical behavior agrees fairly well with the results of actual fatigue tests on composite beams. A possible explanation for the difference in theoretical and actual fatigue resistance can be found in the differences in failure criteria in the two methods.
5. Measured tensile strains indicate that effective moduli of the pavement materials during the tests are lower than the calculated moduli.
6. The exceptionally large tensile strains imposed during the flexural tests provide an explanation for the relatively short fatigue life of the asphalt-concrete beams.
7. The use of a test temperature of 72 F is found to be fairly critical for flexural fatigue of the asphalt-concrete beams. Fatigue resistance passes a minimum value near this temperature and tends to increase at both higher and lower temperatures.

ACKNOWLEDGMENTS

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The author also wishes to acknowledge the assistance of the California Division of Highways Materials and Research Department for their permission to use the flexural fatigue apparatus to perform the needed strain measurements. Bob Stoker was particularly helpful in arranging the tests.

REFERENCES

1. Design Manual for Orthotropic Steel Plate Deck Bridges. American Inst. of Steel Construction, New York, 1963.
2. Gilligan, J. A. Wearing Surfaces for Orthotropic Bridges. Eng. Jour., July 1961.
3. Hardenberg, G. Design of the Superstructure of the Port Mann Bridge. The Eng. Jour., July 1961.
4. Shields, E. J. Artistic Orthotropic Design Also Saves Money. Eng. News-Record, Nov. 21, 1963.
5. Trott, J. J., and Wilson, D. S. Asphalt Surfacing for Steel Bridge Decks. Queens Highway, 28(2), 1962.
6. Bitumen for Modern Bridge Deck Surfacing: Dutch Practice. Shell Bitumen Rev. No. 11, May 1962.
7. Wichert, P. Asphalt Pavements on Three Steel Bridges with Orthotropic Plates. Bitumen, Vol. 26, August 1964.
8. Ziegler, H. Problems in the Practical Use of Pavements on Orthotropic Plates for Traffic Lanes. Bitumen, Vol. 26, August 1964.
9. Wildeboer, A. F. W., and Dubois, P. Asphalt Surfacings on Bridges. Roads and Road Construction, January 1963.
10. Nijboer, L. W. Dynamic Investigations of Road Constructions. Shell Bitumen Monograph No. 2.
11. Standard Specifications, State of California, Div. of Highways, July 1964.
12. Mix Design Methods for Hot Mix Asphalt Paving. Manual Ser. No. 2, The Asphalt Inst.
13. Metcalf, C. T. Use of Marshall Stability Test in Asphalt Paving Mix Design. HRB Bull. 234, pp. 12-22, 1959.
14. Heukelom, W., and Klomp, A. J. G. Proc. Assoc. of Asphalt Paving Technologists, Vol. 33, Feb. 1964.
15. Van der Poel, C. A General System Describing the Visco-Elastic Properties of Bitumens, and Its Relation to Routine Test Data. Jour. Appl. Chem., Vol. 4, 1959.
16. Heukelom, W., and Klomp, A. J. G. Internat. Conf. on the Structural Design of Asphalt Pavements, 1962.
17. Laboratory Tests for Steel Deck Overlay, San Mateo-Hayward Bridge. California Div. of Highways Materials and Research Dept., Sept. 1964.
18. Dormon, G. M., and Metcalf, C. T. Design Curves for Flexible Pavements Based on Layered System Theory. Highway Research Record 71, pp. 69-84, 1965.
19. Hveem, F. N. Pavement Deflections and Fatigue Failures. HRB Bull. 114, pp. 43-79, 1955.

Appendix A

COMPACTION PROCEDURE FOR COMPOSITE BEAM SPECIMENS

The following procedure was adopted for the compaction of asphalt-concrete beams:

1. Place the 4- by 18- by $\frac{5}{8}$-in. steel plate in the bottom of the heated beam mold. Spread the hot mix (280 ± 20 F) uniformly in the mold on top of the steel plate. (For the mix used in preparation of these specimens, 4,200 gm of material is required for a compacted thickness of $1\frac{1}{2}$ in.)

2. By use of the special leveling tool, spread the mix in the mold until it is distributed to a uniform depth in the mold.

3. Apply 20 blows to the mix with the Marshall hammer fitted with a square foot ($3\frac{15}{16}$ in. square). Space the blows evenly over the surface of the mix. (This initial compaction with the Marshall hammer serves to stabilize the mix in the mold so that the mix can be effectively compacted with the air hammer.)

4. Apply the principal compaction to the beam specimen with the air hammer (air pressure 60 psi), as shown in Figure 15. Compactive effort is applied through a square compaction foot ($3\frac{7}{8}$ in. square). Compact the material in the mold in the following sequence: (a) compact the opposite ends of the specimen using a residue time of 10 sec; (b) compact the next adjacent areas (closer to the center of the beam) using a residence time of 7 sec, and overlap the initial compaction areas by approximately $\frac{1}{2}$ in.; and (c) compact the remaining center section using a residence time of 5 sec.

5. Following compaction with the air hammer, apply a static leveling load to the surface of the specimen: (a) place heated (260-300 F) steel plate ($\frac{1}{2}$ by $17\frac{1}{2}$ by 4 in.) on surface of specimen; (b) place flange of 4-in. H-beam (17 in. long) on top of steel plate; (c) apply load to H-beam at rate of 0.05 in./min until a load of 25,000 lb is accomplished and then release the load.

6. Chill the beam and mold in a water bath (approximately 3 min required), and then press the completed composite beam specimen from the mold by pressing against the bottom of the compaction mold.

For the preparation of the experimental material, a procedure similar to the foregoing was followed with the exception that a reduced compactive effort was employed. Compaction sequence with the air hammer involved the application of 4-sec compaction to the ends of the beam which was followed by 3-sec compaction on the adjacent areas, and then 2-sec compaction on the center section. The 25,000-lb static leveling load was also employed.

Figure 15. Air hammer used for compaction of composite beams.

Appendix B

FLEXURAL FATIGUE TEST

The apparatus used in performance of flexural fatigue tests (Fig. 16) is a machine previously developed by the State of California for the fatigue testing of ordinary asphalt-concrete beams (19). The equipment is installed in the California Materials and Research Laboratory at Sacramento, and it was adapted to the testing of the composite beam specimens by a few simple modifications.

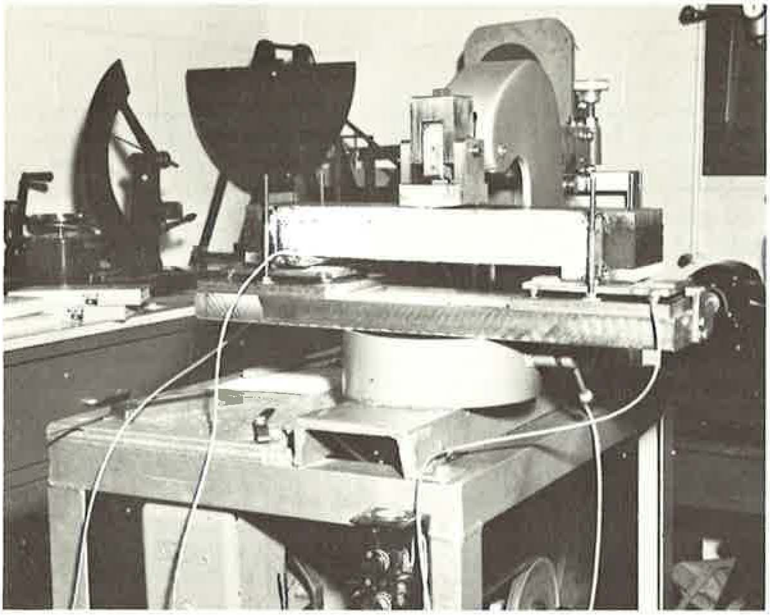

Figure 16. Flexural fatigue apparatus with beam in position for testing.

Essentially, the machine can be described as a "constant deflection" type of instrument in which the test beams are simply supported at each end, and the load is applied at the center of the beam. A cam and lever arrangement is used to apply the load. Deflection of the beam is controlled by a threaded screw adjustment at one end of the lever system.

In the performance of the fatigue test, the beam is positioned so that the center of the beam is under the loading head. (For composite beam tests, the beam was positioned so that the paving material would be in tension.) The loading head is then brought into contact with the beam and a load is applied to the specimen by changing the position of the lever mechanism. This changes the deflection produced at the center of the beam under the lever arm. Thus, the load on the beam is controlled through control of deflection.

For the composite beam tests, a "constant load" technique was specified. This required the exertion of a 700-lb load (at 5 cps) on the center of the beam throughout the test. To provide this load, the deflection of the beam was adjusted until the specified load was accomplished. This deflection was then maintained through control of the lever system which was used to apply the load.

Load on the beam was measured by load cells (steel members equipped with strain gages) placed under each end of the beam. Initially, the apparatus was calibrated by adjusting the lever system until the sum of the loads indicated by the load cells was equal to 700 lb. During the progress of a test, load was monitored through load-cell readings, and the beam deflection was adjusted to compensate for any change in the load. Under this procedure, it is conceivable that stress and strain conditions were not maintained at a constant level throughout the test.

Individual fatigue tests were continued until a crack could be detected in the pavement layer. Quite often, the inception of a crack was difficult to determine. Frequently, the crack had progressed some distance through the material before it was large enough to be seen. Consequently, this presented somewhat of a problem in the establishment of a reliable criterion of failure. Some consideration was given to the use of strain gages (which would break when the crack formed) as an aid in crack detection, but this practice also failed to give conclusive results.

An Analysis of the Three Material Beam

ELTON G. ENDEBROCK and RALPH M. RICHARD
Respectively, Assistant Professor of Civil Engineering, New Mexico State University, and Associate Professor of Civil Engineering, University of Arizona

A plane stress elasticity solution to the simply supported, three material beam is developed. Results are obtained for beams whose dimensions and material properties simulate composite beams consisting of two materials bonded together by a third material. It is shown that the elementary formulas for stresses and deflections yield sufficiently accurate values under certain conditions. In addition, slip, a quantity often measured in experimental work, is influenced by many factors, and therefore is generally not reliable for use in interpreting test results. Experimental results are presented to verify the effectiveness of the mathematical solution.

•IN recent years considerable research has been conducted on beams composed of two or more materials. Most of this research has been concerned with the behavior of composite beams for use in highway and building construction (1). Composite beams usually consist of a steel beam and a concrete slab joined either by mechanical connectors, or recently, by epoxy resins (2, 3). Many experimental studies of composite beams have been conducted and several mathematical solutions have been developed (4, 5, 6). Through experimental studies and mathematical analyses, the conditions under which the elementary formulas for stresses and deflections are sufficiently accurate have been determined for composite beams utilizing spaced mechanical connectors; however, these conditions may not apply to composite beams having a continuous connection between the slab and beam even though some of the solutions for beams with spaced mechanical connectors were based on the assumption that the connection was continuous. Also, the existing mathematical solutions have not compared favorably with all experimental results, i.e., in the case of slips. Because of these uncertainties and discrepancies, a more exact mathematical solution is desirable.

THEORETICAL DEVELOPMENT

The model beam consists of three material layers or individual beams of different depths and having different material properties (Fig. 1). The mathematical expressions describing the behavior of the three material, simply supported beam are obtained by employing the methods of the theory of elasticity. A plane stress condition is assumed; hence, determination of the solution consists of generating a stress function that satisfies the biharmonic equation of elasticity and the appropriate boundary conditions. A stress function is determined for each material layer, and the normal and shearing stresses and the displacements at the interface of the adjoining layers are equated.

The stress function ϕ for each material layer is assumed in the form

$$\phi = f(y)\sin\frac{n\pi x}{L} \tag{1a}$$

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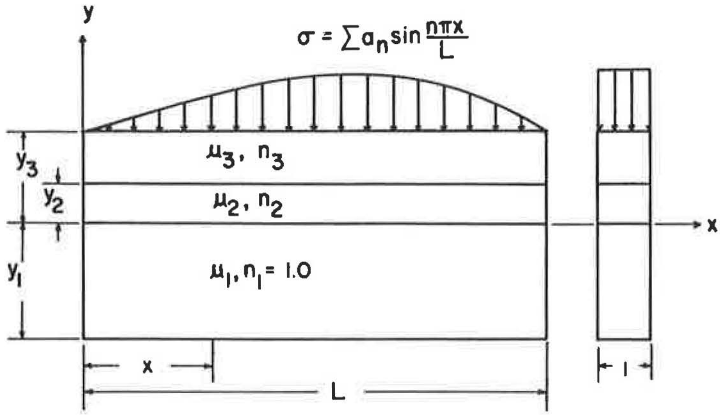

Figure 1. Mathematical model.

where f(y) is a function of y only, L is the length of the beam, and n is an integer. Substitution of ϕ into the biharmonic equation yields an ordinary linear differential equation in f(y) which has a solution

$$f(y) = C_1\cosh\alpha y + C_2\sinh\alpha y + C_3 y\cosh\alpha y + C_4 y\sinh\alpha y \tag{1b}$$

where C_1, C_2, C_3, and C_4 are constants of integration, and $\alpha = n\pi/L$.

The normal stresses σ_x and σ_y and the shearing stress τ_{xy} are obtained from the stress function ϕ. They are

$$\sigma_x = \frac{\partial^2\phi}{\partial y^2} = \sin\alpha x[C_1\alpha^2\cosh\alpha y + C_2\alpha^2\sinh\alpha y + C_3\alpha(2\sinh\alpha y + \alpha y\cosh\alpha y) + C_4(2\cosh\alpha y + \alpha y\sinh\alpha y)] \tag{2a}$$

$$\sigma_y = \frac{\partial^2\phi}{\partial x^2} = -\alpha^2\sin\alpha x[C_1\cosh\alpha y + C_2\sinh\alpha y + C_3 y\cosh\alpha y + C_4 y\sinh\alpha y] \tag{2b}$$

$$\tau_{xy} = -\frac{\partial^2\phi}{\partial x\partial y} = -\cos\alpha x[C_1\alpha\sinh\alpha y + C_2\alpha\cosh\alpha y + C_3(\cosh\alpha y + \alpha y\sinh\alpha y) + C_4(\sinh\alpha y + \alpha y\cosh\alpha y)] \tag{2c}$$

The stress functions for all three material layers have the same form as Eq. 1; however, they differ because the constants of integration are not the same for each layer. The horizontal displacement u and the vertical displacement v enter into the boundary conditions; hence, it is required that they be obtained. The relationships between the strains, displacements, and stresses are

$$\epsilon_x = \frac{\partial u}{\partial x} = \frac{1}{E}(\sigma_x - \mu\sigma_y) \tag{3a}$$

$$\epsilon_y = \frac{\partial v}{\partial y} = \frac{1}{E}(\sigma_y - \mu\sigma_x) \tag{3b}$$

$$\gamma_{xy} = \frac{\partial v}{\partial x} + \frac{\partial u}{\partial y} = \frac{2(1+\mu)}{E}\tau_{xy} \tag{3c}$$

where E is the modulus of elasticity and μ is Poisson's ratio.

The stress equations (Eq. 2) are substituted into the strain equation (Eq. 3). Eqs. 3a and 3b are integrated to obtain the displacements u and v. Integration of these equations yields two arbitrary functions which are evaluated by substituting the displacements u and v into Eq. 3c. The resulting expressions for u and v are

$$\begin{aligned} u = & -\frac{\cos\alpha x}{E}\{C_1\alpha(1+\mu)\cosh\alpha y + C_2\alpha(1+\mu)\sinh\alpha y + C_3[2\sinh\alpha y \\ & + \alpha y(1+\mu)\cosh\alpha y] + C_4[2\cosh\alpha y + \alpha y(1+\mu)\sinh\alpha y]\} \\ & + Ay + B \end{aligned} \tag{4a}$$

$$\begin{aligned} v = & -\frac{\sin x}{E}\{C_1\alpha(1+\mu)\sinh\alpha y + C_2\alpha(1+\mu)\cosh\alpha y + C_3[\alpha y(1+\mu)\sinh\alpha y \\ & + (-1+\mu)\cosh\alpha y] + C_4[\alpha y(1+\mu)\cosh\alpha y + (-1+\mu)\sinh\alpha y]\} \\ & - Ax + F \end{aligned} \tag{4b}$$

where A, B, and F are constants of integration. These constants are associated with rigid body motions, and are taken equal to zero here.

The constants of integration are then evaluated from the boundary conditions. There is a stress function for each material layer, and there are four constants of integration for each stress function; therefore, there are twelve independent constants of integration for which there must be twelve independent boundary conditions to obtain the solution. The stress functions are denoted as ϕ_1, ϕ_2, and ϕ_3, where the subscripts correspond to the material layers indicated in Figure 1. The constants of integration are denoted as C_1^1, C_2^1, etc., corresponding to ϕ_1; C_1^2, C_2^2, etc., corresponding to ϕ_2, etc.

The boundary conditions are as follows. The superscripts denote the corresponding material layers as indicated in Figure 1.

$$\tau_{xy}^1(x, -y_1) = 0 \tag{5a}$$

$$\sigma_y^1(x, -y_1) = 0 \tag{5b}$$

$$\tau_{xy}^1(x, 0) = \tau_{xy}^2(x, 0) \tag{5c}$$

$$\sigma_y^1(x, 0) = \sigma_y^2(x, 0) \tag{5d}$$

$$u^1(x, 0) = u^2(x, 0) \tag{5e}$$

$$v^1(x, 0) = v^2(x, 0) \tag{5f}$$

$$\tau_{xy}^2(x, y_2) = \tau_{xy}^3(x, y_2) \tag{5g}$$

$$\sigma_y^2(x, y_2) = \sigma_y^3(x, y_2) \tag{5h}$$

$$u^2(x, y_2) = u^3(x, y_2) \tag{5i}$$

$$v^2(x, y_2) = v^3(x, y_2) \tag{5j}$$

$$\tau^3_{xy}(x, y_3) = 0 \tag{5k}$$

$$\sigma^3_y(x, y_3) = \Sigma a_n \sin \frac{n\pi x}{L} \tag{5l}$$

$$\int_A \sigma_x(0, y)dy = 0 \tag{6a}$$

$$\int_A \sigma_x(L, y)dy = 0 \tag{6b}$$

$$\int_A \sigma_x(0, y)ydy = 0 \tag{6c}$$

$$\int_A \sigma_x(L, y)ydy = 0 \tag{6d}$$

$$\int_A \tau_{xy}(0, y)dy = \text{reaction} \tag{6e}$$

$$\int_A \tau_{xy}(L, y)dy = \text{reaction} \tag{6f}$$

where the letter A associated with the integral sign denotes the total cross-sectional area of the beam.

Boundary conditions in Eq. 5 are used to evaluate the twelve constants of integration, whereas boundary conditions in Eq. 6 were identically satisfied by the choice of the stress functions. The load in Eq. 5l is represented by a Fourier sin series expansion, thereby allowing a large degree of freedom in the type of loadings that may be studied.

By applying the boundary conditions (Eq. 5), twelve simultaneous linear equations are obtained with the constants of integration as unknowns. The constants of integration are dependent on the parameter α which in turn depends on the value of the integer n. For this reason, the twelve equations must generally be solved for each value of n. The number of times the set of equations must be solved therefore depends on the number of terms of the series used to represent a given loading. Because of the large number of computations involved, numerical results are best obtained with the use of a digital computer.

DISCUSSION OF RESULTS

The examples of three material beams presented and discussed in this paper have central material layer depths that are small in comparison to the depths of the top and bottom layers. In addition, the total depth of the beam is small in comparison to its length. This type of beam simulates a composite beam consisting of two materials bonded together by a third material. The dimensions and material properties of the top and bottom layers are selected so that the neutral axis of the entire cross-section is located very near the top of the bottom layer.

Except for the depths of the central and top layers, the relative beam dimensions are the same for all the examples presented. In terms of the dimensionless ratio x/L, the span length is unity. The vertical coordinates and dimensions are expressed in terms of the ratio y/L. For a given beam, three y/L values are assigned, one for each material layer. For the examples presented here, the y/L values for the top and bottom edges are 0.016 and 0.050, respectively. The moduli of elasticity are expressed as ratios of a particular layer to the modulus of elasticity of the bottom layer; hence, the modular ratio of the bottom layer is always unity. The letter n is used to denote the modular ratios and the subscripts associated with the n's identify the associated material layer as shown in Figure 1. The associated subscripts should prevent any

confusion with the integer n used previously. The modular ratios of the top and central layers are varied; however, Poisson's ratios for all three layers are kept the same throughout this investigation. Poisson's ratios for the top, central, and bottom layers are 0.15, 0.20, and 0.25, respectively. The relative dimensions and material properties were chosen to simulate, as nearly as possible, composite beams used in practice.

For most of the examples presented, a single concentrated load of unity was applied at midspan. In some cases two concentrated loads symmetrically placed about midspan are used. The total magnitude of the loads is unity, and they are located at x/L equal 0.30 and 0.70. The concentrated loads are approximated by using a uniformly distributed load acting over approximately $1/100$ of the span length. All beams presented here were symmetrically loaded; however, the solution developed is not limited to symmetrically loaded beams. The solutions obtained are valid in the range of elastic behavior of the materials only.

The horizontal tensile stress σ and the vertical displacement v are obtained at midspan for simple end-supported beams in which the modular ratio of the top material layer and the modular ratio and depth of the central material layer are varied. The top layer modular ratios n_3 used are 4.0, 6.0, and 8.0. The central layer depth is varied from a y/L value of 0.0005 to 0.0020 in increments of 0.0005. The central layer modular ratio n_2 is assigned values of 1.0, 0.10, 0.01, 0.005, 0.001, and 0.0001. The horizontal stresses and vertical displacements (or deflections) at midspan of the beams are presented in terms of the ratios σ/σ_e and δ/δ_e, where σ and δ are obtained from the expressions for σ_x and v developed earlier and σ_e and δ_e are obtained by using the transformed section and the elementary beam formulas (Appendix). The stresses σ and σ_e are the tensile stresses in the lower fiber of the beam. This stress is used because the compressive stresses in the top fibers of the beam are not comparable. The concentrated load is applied at midspan; hence, the elasticity solution yields a local effect in the stresses at midspan, whereas the elementary solution does not. This local effect is not nearly as pronounced in the bottom fibers of the beam. Stress variation along the beam for the stresses in the top and bottom fibers of the beam are shown in Figure 2.

The deflection δ was computed at the top edge of the bottom layer (y = 0). This would not introduce a significant error because the neutral axis is very near this edge. The deflection δ_e was obtained at the neutral axis of the section as computed by elementary methods.

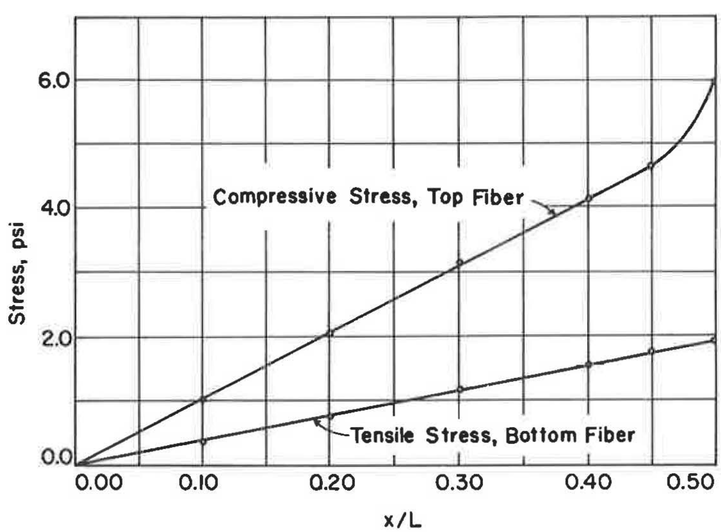

Figure 2. Stress variation along beam's length.

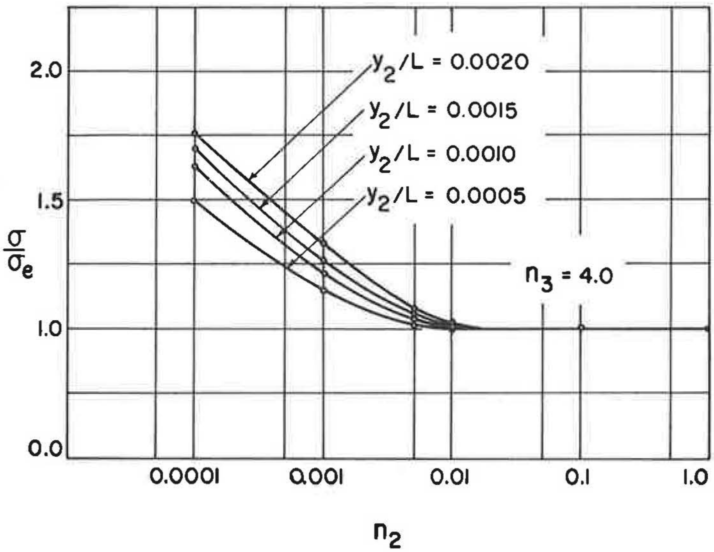

Figure 3. Stress ratio vs central layer modular ratio.

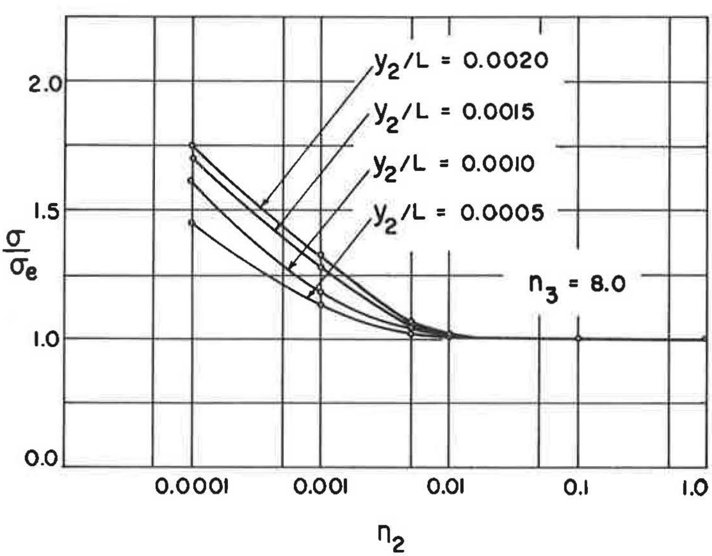

Figure 4. Stress ratio vs central layer modular ratio.

The results of this part of the investigation are shown in Figures 3 through 8 where the stress ratio σ/σ_e and the deflection ratio δ/δ_e are plotted against the modular ratio n_2 of the central layer. Figures 3 and 4 show that for central layer modular ratios between 0.01 and 1.0 the stress ratio is very nearly unity. The significance of this is that for this range of n_2 the stresses are predicted with sufficient accuracy by the use of the transformed section and the elementary beam stress formulas. Although evidence is not presented here, this statement holds true for both normal and shearing stresses, and also for any type of loading (8). The homogeneous beam would produce the largest shearing stress, and the shearing stress decreases as the central layer modular ratio decreases; hence, the elementary shear stress formula would yield slightly conservative results in the central modular ratio range listed previously.

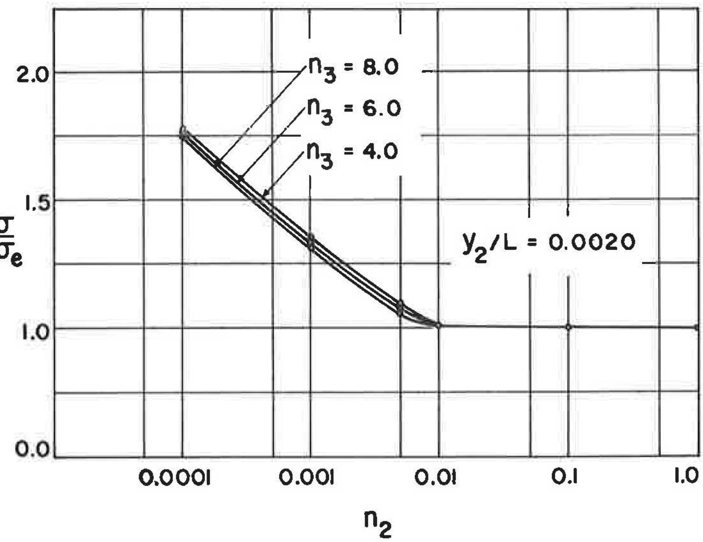

Figure 5. Stress ratio vs central layer modular ratio.

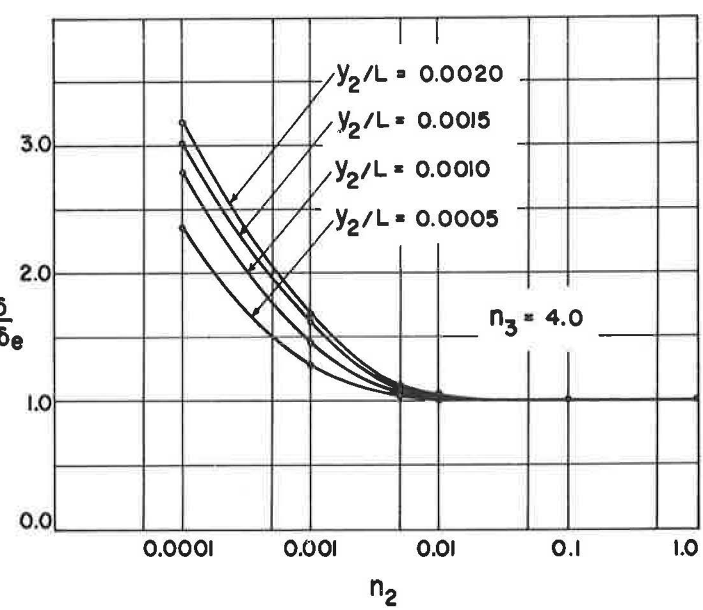

Figure 6. Deflection ratio vs central layer modular ratio.

For modular ratios less than about 0.01, the stress ratios increase; hence, the stresses computed with the elementrary stress formulas would be in error, the magnitude of the error increasing as the central layer modular ratio becomes smaller. Figures 3 and 4 show that the central layer depths considered did not have a significant effect on the stress ratios in the range of central layer modular ratios between 0.01 and 1.0. The central layer depths noticeably affected the stress ratios for modular ratios n_3 less than about 0.01. The stresses increase with increasing central layer depths. Figure 5 shows that the stress ratios are not greatly affected by the value of the modular ratio of the top layer within the range of values considered.

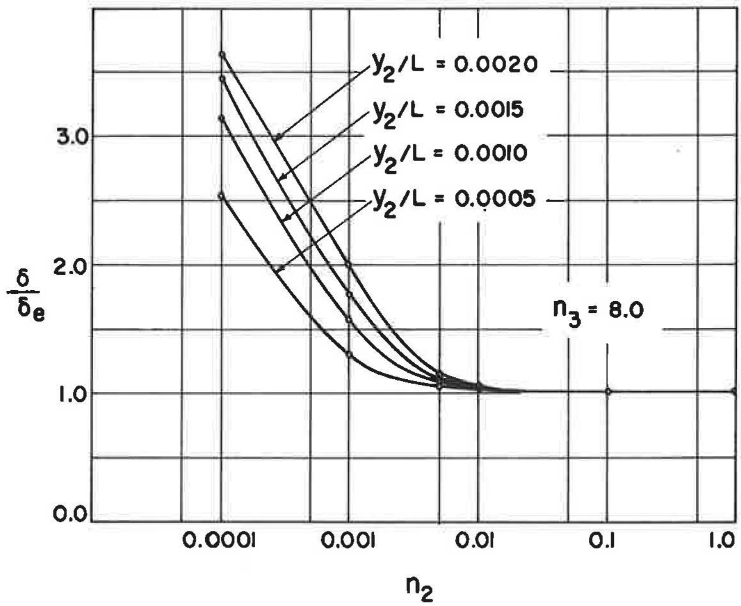

Figure 7. Deflection ratio vs central layer modular ratio.

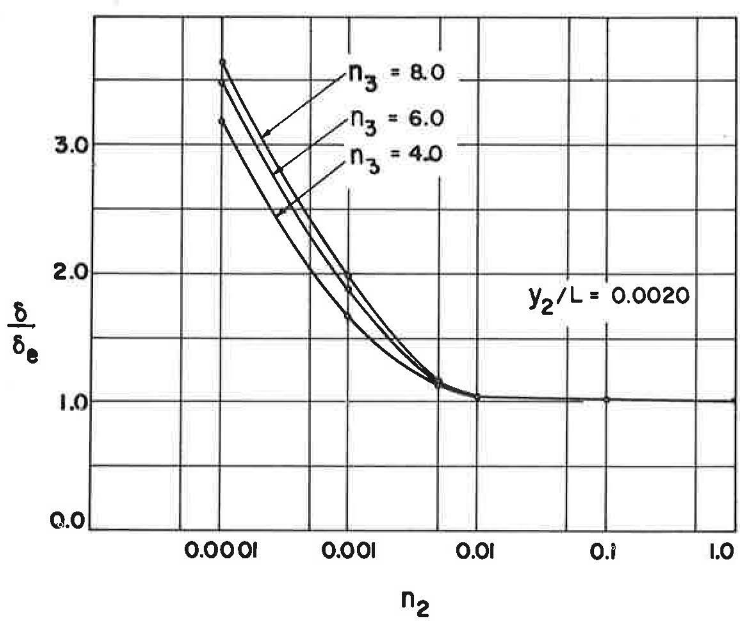

Figure 8. Deflection ratio vs central layer modular ratio.

The deflection ratios vs the central layer modular ratios are shown in Figures 6 and 7. In general, the foregoing discussion on the stress ratios also applies to the deflection ratios. Between the central layer modular ratios of 0.01 to 1.0, the elementary beam formulas for deflection may be used. The deflections are more sensitive to the central layer depths and top layer modular ratios than the stresses for central modular ratios of about 0.01 and less (Fig. 8). The deflection ratios increase as the top layer modular ratios increase.

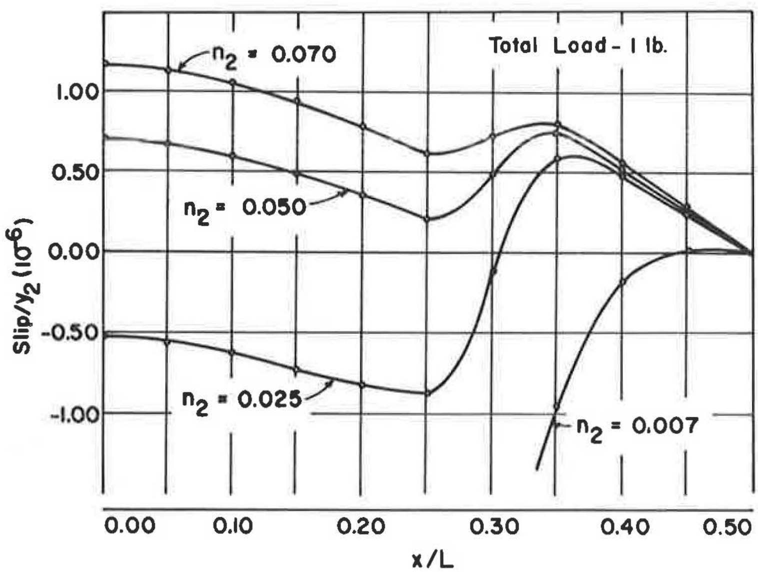

Figure 9. Slip curves for different central layer modular ratios.

The three material or composite beams presented here are considered to satisfy the assumptions on which the mathematical solution is based, i.e., the beams are in a condition of plane stress. Composite beams used in construction do not generally satisfy the assumption of a plane stress condition. Their cross-sections are neither rectangular shaped nor are their widths small in comparison to their depths; hence, the predominate normal stress is not uniformly distributed across the width of the cross-section. Even though T-beams cannot be accurately analyzed, in gaining an adequate insight into the behavior of continuously connected composite beams, the purpose of this investigation has been realized. Another investigation is being conducted to determine a more exact method of analyzing T-beams.

In previous investigations of composite beams (3, 5, 7) the relative displacement at a point between the upper and lower material layers was usually measured. This quantity, commonly referred to as slip, was usually measured at the ends of the beams, but in some cases it was also measured at several points along the span length.

Slip curves were obtained for several cases for the purpose of determining the quantities affecting the slip magnitudes and slip distribution along the span length. Here slip is presented as a ratio of the relative displacement between the top and bottom layers at a point to the depth of the central layer. For this study, the beams were loaded by two concentrated loads symmetrically placed about midspan.

Slip curves for beams having different central layer modular ratios n_2 are shown in Figure 9. The slips decrease algebraically as the central layer modular ratios decrease. The slips are positive (a point on the bottom edge of the top layer displaces to the right of a point on the top edge of the bottom layer) along half of the span for a central layer modular ratio of 0.050 and greater. The slips in the region between the end of the beam and the applied load (at $x/L = 0.30$) were negative for a central layer modular ratio of 0.025 and less. Some of the slips in the region between midspan and the applied load remained positive until the central layer modular ratios became very small. The foregoing discussion applied to the left half of the beam. The signs of the slips in the right half of the beam would be opposite to those in the left half of the beam. Positive slips do occur and have been observed by the authors.

The effect on the slips of changing the reaction locations is also investigated. The reactions are placed at x/L values of 0.00, 0.025, and 0.050. The applied loads are kept the same distance from the ends of the beams. Slip curves for the three reaction locations for beams loaded by two concentrated loads symmetrically

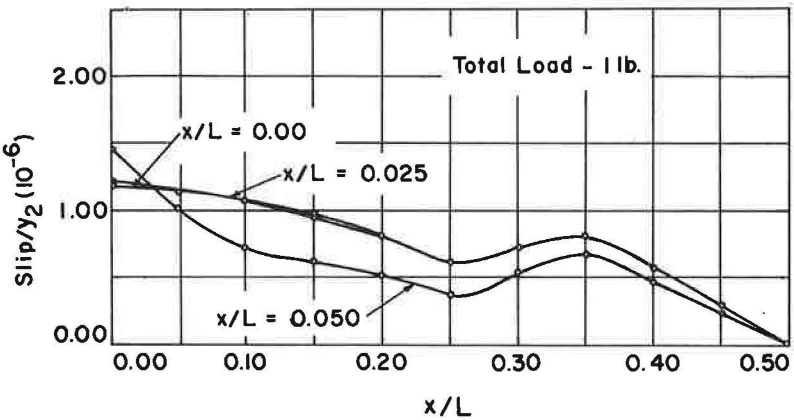

Figure 10. Slip curves for beams with different reaction locations.

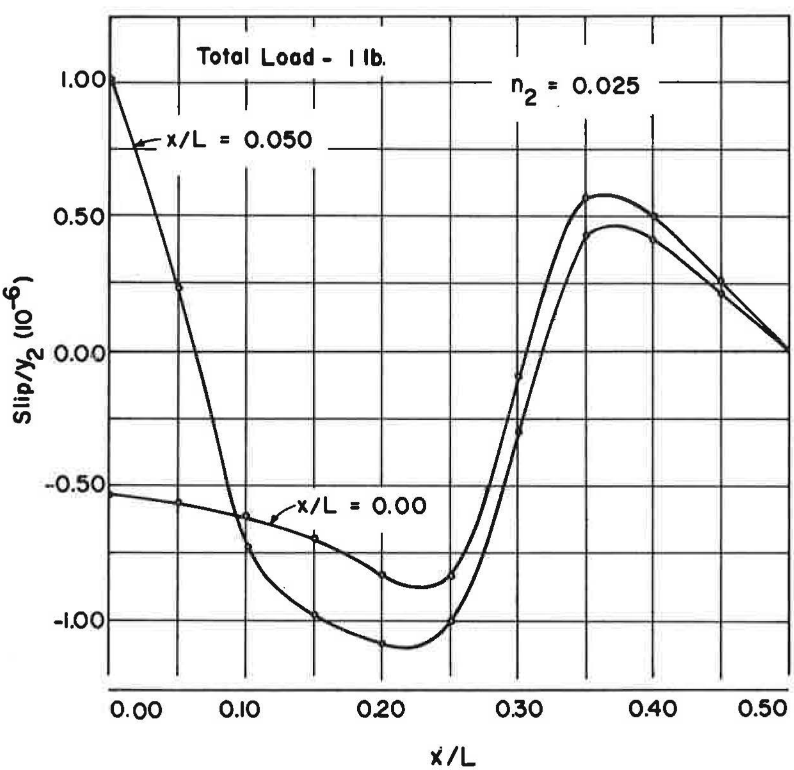

Figure 11. Slip curves for beams with different reaction locations.

placed about midspan are shown in Figure 10. The central modular ratio for the beam is 0.070. The maximum slip, which is positive, occurs at the ends of the beam, and the changing of the reaction location increases the end slips. The end slips increase as the reactions move away from the ends. The effect on the slips is small for the reaction location of x/L equal to 0.025, but becomes quite pronounced at x/L equal to 0.050.

Figure 11 shows the slips for the case of the reactions located at the ends were negative between the ends and the applied loads. This beam had a central layer modular ratio of 0.025. For the reaction located at x/L equal to 0.050, the slip curve had a completely different shape than the curve with the reactions located at the ends. The maximum slip did not occur at the ends. This had been verified by several investigators who performed tests on composite beams (3, 7).

EXPERIMENTAL TEST RESULTS

A three-layer beam consisting of aluminum, steel, and epoxy resin was constructed, instrumented, and tested for the purpose of showing the effectiveness of the mathematical solution. The test results were then compared to results predicted by the mathematical solution.

Steel and aluminum constituted the top and bottom layers, whereas the epoxy resin constituted the center layer. The cross-sectional dimensions of the steel, aluminum, and epoxy resin were ½ by ½ in., ¾ by ½ in., and ¼ by ½ in., respectively. The beam was ½ in. wide and 1½ in. deep. The total length of the beam was 48½ in. During testing the beam was simply supported with a span length of 48 in. The material constants (modulus of elasticity and Poisson's ratio) for the steel and aluminum were taken from a handbook, whereas the material constants for the epoxy resin were obtained by testing tensile specimens. The tensile specimens were molded at the time the beam was constructed, and kept in the same environment as the beam. The strains in the tensile specimens were obtained by means of electric strain gages. The results of the epoxy resin tensile tests are shown in Figure 12. The stress-strain relationship for the epoxy resin was essentially linear.

The model beam was simply supported on knife-edge supports 48 in. apart. The beam was positioned on the supports so that the steel constituted the top layer. Instru-

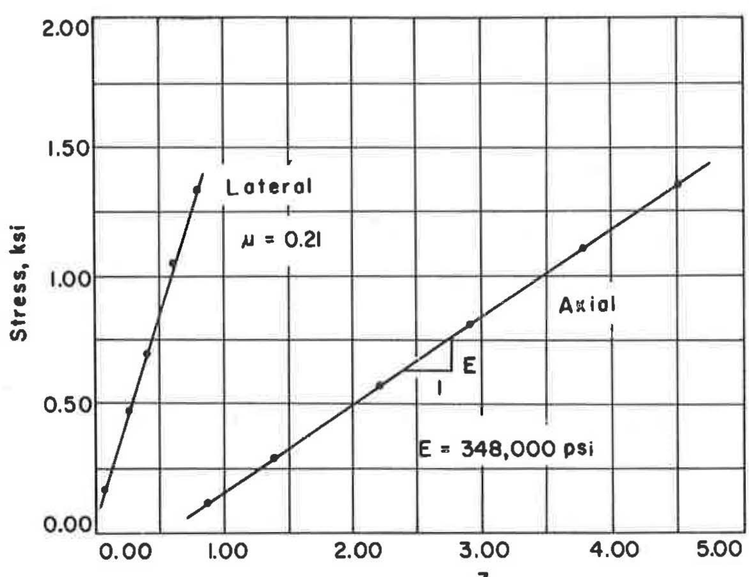

Figure 12. Stress-strain curves for epoxy resin.

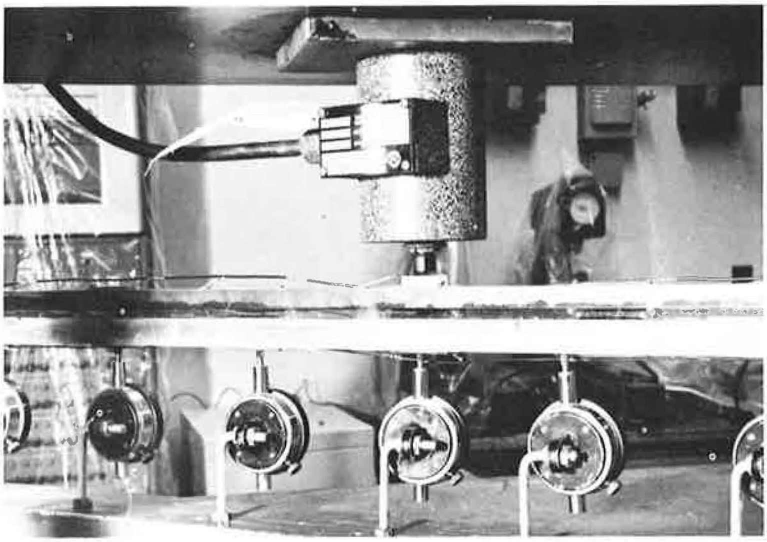

Figure 13. Model beam.

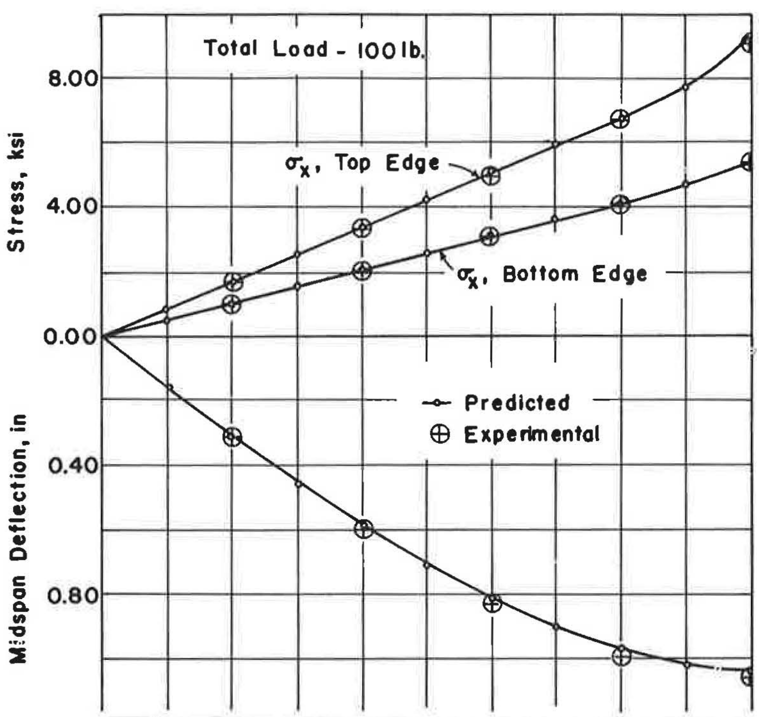

Figure 14. Comparison of predicted and experimental results–single load.

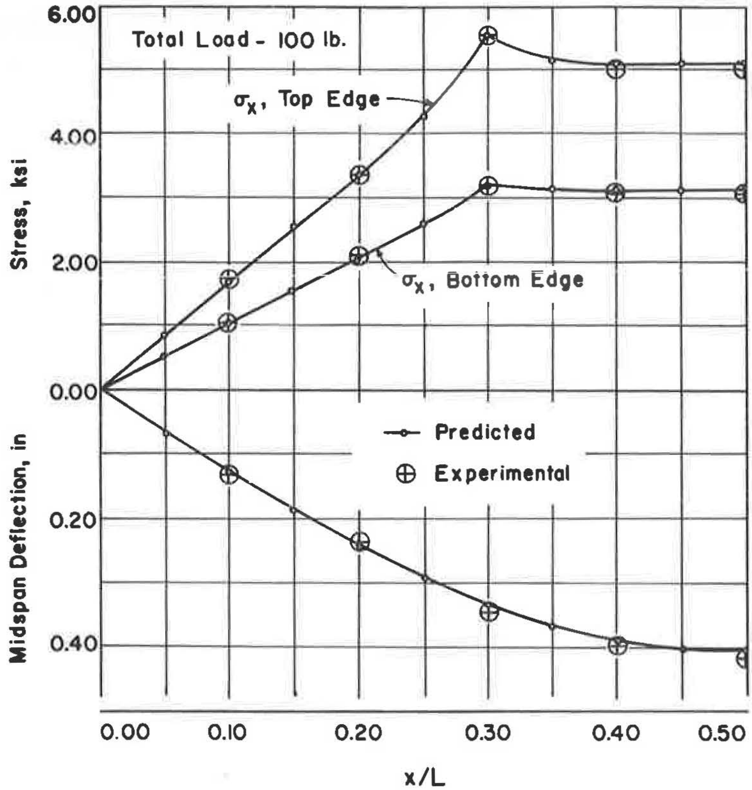

Figure 15. Comparison of predicted and experimental results—two point load.

mentation consisted of electric strain gages and 0.0001-in. dial deflection gages. The electric strain gages were placed on the top and bottom of both the steel and aluminum layers at every one-tenth point of the span (4.8 in. in this case). The dial deflection gages were located at every one-tenth point of the span length. The model beam was not instrumented to measure end slips, because of the lack of sensitive measuring devices. A single concentrated load at midspan and a symmetrically placed two point loading located at $x/L = 0.30$ and 0.70 was used. The load was measured to an accuracy of one pound by means of a load cell. The mathematical solution applied only in the elastic range; hence, during testing the maximum load applied was kept below the yield load. The model beam is shown in Figure 13.

The predicted and experimental horizontal stresses at the top and bottom edges of the model beam for a single concentrated load at midspan and for a symmetrically placed two point loading are shown in Figures 14 and 15, respectively. The values are for a total applied load of 100 lb. There is less than 3 percent difference between the maximum predicted values and the test values.

CONCLUSIONS

Studies of several examples of three material beams whose central layer depths were small compared to the depths of the other two material layers, and whose total depth was small compared to the beam's length have been presented. For thin central layers, the quantity that had the greatest effect on the predominate stresses and deflections was the central layer modular ratio. It was also shown that for a certain range of central layer modular ratios, the predominate stresses and deflections may be determined with sufficient accuracy by using the transformed section and the elementary formulas for stresses and deflections.

An investigation was conducted to determine the reliability of the relative displacement (slip) at a point between the top and bottom material layers as an indicator of the behavior of composite beams. The slips were sensitive to many factors, and therefore not a reliable indicator for use in experimental studies unless properly interpreted.

A three material model beam was constructed and tested. The test results were compared to results predicted by the mathematical solution. There was little difference between the test values and the predicted values; hence, the effectiveness of the mathematical solution was verified.

The foregoing conclusions are valid in the elastic range of behavior of the materials, and are also subject to the range of the variables considered.

REFERENCES

1. Viest, Ivan M. Review of Research on Composite Steel-Concrete Beams. Proc., ASCE, Vol. 86, J. Structural Div., June 1960.
2. Miklofsky, Haaren A., Brown, Martin R., Jr., and Gonsior, Michael, J. Epoxy Bonding Compounds as Shear Connectors in Composite Beams. Rensselaer Polytechnic Inst. Eng. Res. Series RR62-2, Troy, New York, Oct. 1962.
3. Kriegh, James D., and Endebrock, Elton G. The Use of Epoxy Resins in Reinforced Concrete Static Load Tests. Univ. of Arizona Eng. Res. Lab., Tucson, Arizona, Jan. 1963.
4. Stuessi, F. Zusammengesetzte Vollwandtraeger. Publications, Internat. Assoc. for Bridge and Structural Eng., Vol. 8, pp. 249-269, 1947.
5. Jaeger, K. Die Verbundwirkung zwischen Stahltraeger und Stahlbeton platte. Oesterreichischer Ingenieur - Archien (Wien), Vol. 3, No. 4, pp. 295-311, 1949.
6. Newmark, Nathan M., Siess, Chester P., and Viest, Ivan M. Tests and Analysis of Composite Beams with Incomplete Interaction. Proc., Soc. for Experimental Stress Analysis, Vol. 9, No. 1, pp. 75-92, 1951.
7. Hanson, Norman V. Horizontal Shear Connectors. Jour. of the Res. and Dev. Lab., Portland Concrete Assoc., Vol. 2, No. 2, May 1960.
8. Endebrock, Elton G. An Elasticity Solution of the Multi-Material Beam. PhD thesis, Univ. of Arizona, Tucson, Arizona, 1964.

Appendix

ELEMENTARY METHOD OF COMPUTING STRESSES AND DEFLECTIONS

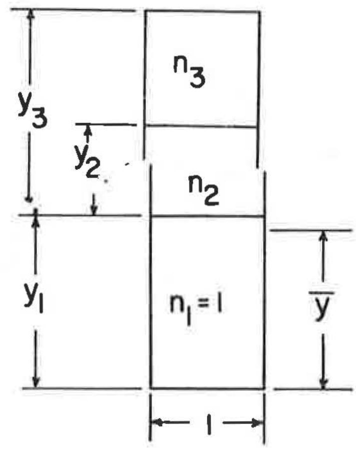

Figure 16.

Neutral axis:

$$\bar{y} = \frac{y_1^2/2 + y_2 n_2(y_2 + y_2/2) + (y_3 - y_2)n_3\left(y_1 + \frac{y_2 + y_3}{2}\right)}{y_1 + n_2 y_2 + n_3(y_3 - y_2)}$$

Moment of inertia:

$$I = \frac{1}{12} y_1^3 + y_1\left(\bar{y} - \frac{y_1}{2}\right)^2 + \frac{1}{12} n_2 y_2^3 + n_2 y_2 (y_1 + y_2/2 - \bar{y})^2$$

$$+ \frac{n_3(y_3 - y_2)^3}{12} + n_3(y_3 - y_2)\left(y_1 + \frac{y_2 + y_3}{2} - \bar{y}\right)^2$$

Stresses:

$$\sigma_e = \frac{M\bar{y}}{I}$$

Deflections:

$$\delta_e = \frac{PL^3}{48I}$$