PB95-210530

MISSOURI COOPERATIVE HIGHWAY RESEARCH PROGRAM

REPORT 89-2

EVALUATION OF WEARING SURFACE SYSTEMS FOR ORTHOTROPIC STEEL-PLATE BRIDGE DECKS

MISSOURI HIGHWAY AND TRANSPORTATION DEPARTMENT
UNIVERSITY OF MISSOURI, COLUMBIA
FEDERAL HIGHWAY ADMINISTRATION

REPRODUCED BY:
U.S. Department of Commerce
National Technical Information Service
Springfield, Virginia 22161

Technical Report Documentation Page

1. Report No. MD-FHWA-89-2	2. PB95-210530	3. Recipient's Catalog No.
4. Title and Subtitle: Evaluation of Wearing Surface Systems for Orthotropic Steel-Plate Bridge Decks		5. Report Date: 1989
		6. Performing Organization Code
7. Author(s): V.S. Gopalaratnam, J.W. Baldwin, Jr., Bryan Hartnagel, and R.A. Rigdon		8. Performing Organization Report No.
9. Performing Organization Name and Address: University of Missouri-Columbia, Columbia, MO		10. Work Unit No. (TRAIS): 4 DC
		11. Contract or Grant No.
		13. Type of Report and Period Covered
12. Sponsoring Agency Name and Address: Missouri Highway and Transportation Department and Federal Highway Administration		14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

Six wearing surface (~~we~~ WS) materials were evaluated as potential replacements for the existing WS on the Poplar Street Bridge in St. Louis. The materials were 2 asphaltic concrete, 3 epoxy concrete and a methyl-methycrylate concrete. Lab tests included fatigue tests (flexural) ~~a~~ 0°R and ~~to~~ a range of 0°-160° F. Concurrently, test sections of the same material were placed on the bridge and monitored for 1-2 years. Recorded strains were correlated with the laboratory fatigue tests.

One of the proprietary epoxy concretes was recommended, although none of them performed as well as desired.

at

17. Key Words: Steel, Bridges, Orthotropic Deck, Wearing Surface, Fatigue		18. Distribution Statement	
19. Security Classif. (of this report): N/A	20. Security Classif. (of this page): N/A	21. No of Pages: 72	22. Price

Form DOT F 1700.7 (8-72) (PF V2.1, 6/30/92)

Reproduction of completed page authorized

EVALUATION OF WEARING SURFACE SYSTEMS FOR ORTHOTROPIC STEEL-PLATE-BRIDGE DECKS

Prepared for

Missouri Highway and Transportation Department

by

Vellore S. Gopalaratnam
James W. Baldwin, Jr.
Bryan A. Hartnagel
and
Robert A. Rigdon

Department of Civil Engineering
University of Missouri-Columbia

in cooperation with

Federal Highway Administration

The opinions, findings, conclusions and recommendations expressed in this report are those of the authors and are not necessarily those of the Federal Highway Administration, the Missouri Highway and Transportation Department or the University of Missouri-Columbia.

1A

TABLE OF CONTENTS

		Page
EXECUTIVE SUMMARY		iii
ACKNOWLEDGEMENTS		iv
LIST OF TABLES		v
LIST OF FIGURES		vi
CHAPTER 1 - INTRODUCTION		1
1.1	Background	1
1.2	Vital Statistics of the Poplar Street Bridge	2
1.3	Related Work	4
1.4	Objective and Scope of the Study	9
CHAPTER 2 - DETAILS OF THE RESEARCH PROGRAM		11
2.1	Overall Experimental Program	11
2.2	Flexural Fatigue Tests	12
	2.2.1 Specimen Geometry and Fabrication	13
	2.2.2 Temperature Cycled Flexural Fatigue Tests	17
	2.2.3 Cold Temperature Flexural Fatigue Tests	19
	2.2.4 Joint-Specimens Subjected to Flexural Fatigue	20
	2.2.5 Test Apparatus and Procedures	21
2.3	Deck Strain Measurements	25
	2.3.1 Strain Measurement Locations	26
	2.3.2 Instrumentation and Data Acquisition Equipment	28
	2.3.3 Test Procedures	30
	2.3.4 Data Acquisition Software and its Features	31
2.4	Test Sections on the Bridge and their Monitoring	33
2.5	Other Tests	36
	2.5.1 Pull-out Tests	36
	2.5.2 Resistivity Tests	38
CHAPTER 3 - TEST RESULTS AND DISCUSSIONS		40
3.1	Flexural Fatigue Tests	40
	3.1.1 Temperature Cycled Flexural Fatigue Tests	41
	3.1.2 Cold Temperature Flexural Fatigue Tests	46
	3.1.3 Flexural Fatigue Tests on Joint Specimens	49
3.2	Deck Strain Measurements	50
	3.2.1 Automated Data Collection	50
	3.2.2 Strain Histories and Hourly Histogram Summaries	51
	3.2.3 Measured Peak Strain and Associated Frequency of Occurance	53
3.3	Performance of the Test Sections on the Bridge	53
3.4	Other Results	55
	3.4.1 Pull-out Tests	55
	3.4.2 Resistivity Tests	55
3.5	Stress Computations from the Deck Strain Measurements and their Relation to the Flexural Fatigue Tests	56
CHAPTER 4 - CONCLUSIONS AND RECOMMENDATIONS		60
4.1	Conclusions	60
4.2	Recommendations	62
REFERENCES		63

EXCUTIVE SUMMARY

Six wearing surface material systems were evaluated as potential replacements for the wearing surface on the Poplar-Street Bridge in St. Louis, Missouri. Two were asphaltic concretes, three were Epoxy Concretes and one was a Methyl Methacrylate Concrete. Laboratory tests included flexural fatigue tests at a constant temperature of 0°F, flexural fatigue tests under cyclic temperature variations from 0°F to 160°F, and ancillary tests to evaluate the condition of the surfaces before and after the fatigue tests. Concurrently with the laboratory tests, a test section of each material was placed on the bridge by its supplier and subjected to normal traffic for 1-2 years. These field test sections were observed regularly for evidence of rutting, shoving, or other signs of deterioration.

In addition to tests of the materials, deck strains on the bridge were monitored continuously for a period of six weeks. Strain histories from this monitoring were converted to stresses in the wearing surface and compared with fatigue stresses imposed during the laboratory tests. One of the proprietary epoxy concretes was recommended as the best of the materials tested, although none of the material systems exhibited as great a margin of confidence against cracking as would be desirable.

ACKNOWLEDGEMENTS

The work reported herein was conducted by the Department of Civil Engineering, University of Missouri-Columbia (MU). It was jointly supported by the Missouri Highway and Transportation Department (MHTD) and the Federal Highway Administration (FHA). Dr. James W. Baldwin, Jr., and Dr. Vellore S. Gopalaratnam were the Principal Investigators.

Laboratory technicians Delbert Morton, Rex Gish and Richard Adams were responsible for fabrication of all the test apparatus. Richard Oberto was responsible for the instrumentation and maintenance of electronic equipment. Graduate students Robert L. Rigdon, Jr., and Bryan A. Hartnagel were responsible for the flexural fatigue tests and the field strain measurements respectively. Additional assistance during various stages of the project from Julie Bax, Clarence Jones and Mark Magruder while they were undergraduate students at the MU Department of Civil Engineering are also acknowledged. Original data acquisition program written by undergraduate student Ellen Gandle for an earlier MU project sponsored by the MHTD was modified for use in the flexural fatigue tests reported here.

LIST OF TABLES

Page

Chapter 2

Table 2.1 Strain levels associated with the ten counters 33

Chapter 3

Table 3.1 Results from the temperature cycled flexural fatigue tests (Upper limit load : 1,450 lbs.) 41

Table 3.2 Results from the temperature cycled flexural fatigue tests (Upper limit load : 1,150 lbs.) 45

Table 3.3 Results from the cold temperature flexural fatigue tests 48

Table 3.4 Changes in counter values after processing the trace shown in Fig. 3.6 52

Table 3.5 Results from the resistivity tests on uncracked and cracked flexural fatigue test specimens 56

LIST OF FIGURES

Page

Chapter 1

Fig. 1.1 A side-view of the Poplar Street Bridge. 3

Fig. 1.2 Cross-section of the eastbound bridge showing structural details 3

Fig. 1.3 A cross-section of the third asphalt-based wearing surface on the Poplar Street Bridge showing rutting along the wheel path 5

Chapter 2

Fig. 2.1 Composite specimen showing the loading configuration used for the flexural fatigue tests 14

Fig. 2.2 Polymer-modified asphalt concrete being placed into the heated molds just prior to compaction 14

Fig. 2.3 The polymer concrete mix is poured into the mold 16

Fig. 2.4 Aggregate is being broadcast on top of the polymer concrete layer 16

Fig. 2.5 Typical temperature variation with time. Corresponding number of fatigue cycles at a rate of 5 Hz of uninterrupted testing is also shown 18

Fig. 2.6 Schematic of the test chamber and the air flow ducts 18

Fig. 2.7 Close-up of the test chambers showing the four specimens and associated instrumentation in place 20

Fig. 2.8 View of the underside of the deck plate inside the southernmost box girder 27

Fig. 2.9 View of the three-gage rosette and the bridge completion gages 27

Fig. 2.10 Close-up view of one of the strain gage rosettes 29

Fig. 2.11 Schematic showing the location of the gages with respect to the box girder webs 29

Fig. 2.12 Layout of the test sections placed in June 1989 [21] giving details of the paint system and wearing surface thicknesses used 34

Fig. 2.13 Layout of the test sections updated to show additional placements of June 1990 [16] 35

Fig. 2.14 Schematic of the core pull-out device 37

Fig. 2.15 Schematic of the test set-up for the resistivity tests 37

Page

Chapter 3

Fig. 3.1 Typical variation of specimen stiffness with temperature (polymer concrete)..... 44

Fig. 3.2 Temperature corrected relative stiffness versus the number of fatigue cycles (upper limit load : 1,450 lbs.). Thermal loading applied simultaneously with the 5 Hz fatigue loading is also shown..... 44

Fig. 3.3 Temperature corrected relative stiffness versus the number of fatigue cycles (upper limit load : 1,150 lbs.). Thermal loading applied simultaneously with the 5 Hz fatigue loading is also shown..... 46

Fig. 3.4 Relative stiffness versus the number of fatigue cycles in a cold temperature flexural fatigue test. Progressively increasing upper limit load is also shown in the figure..... 48

Fig. 3.5 Summary of results from the temperature cycled flexural fatigue tests on joint specimens. Symbols indicate when each crack detection wire failed on the different specimens..... 49

Fig. 3.6 Typical strain history recorded during a loading event (Gage 15)..... 52

CHAPTER 1 - INTRODUCTION

1.1 BACKGROUND

The Poplar Street bridge across the Mississippi river in St. Louis, Missouri is an orthotropic steel-plate deck bridge. Orthotropic steel-plate bridge decks consist of a continuous thin steel plate reinforced by a system of longitudinal stiffeners and transverse floor beams. This type of deck construction is most commonly used with box girders as the primary structural element as in the Poplar Street bridge, but can be used with a truss or arch type bridge as well. There are approximately thirty such bridges in the world including six in the United States [20].

When the bridge was constructed in 1967, the steel deck was covered with a wearing surface consisting of two layers of epoxy tack coat and 1-1/2 inches of rubberized asphalt concrete wearing surface. Stone chips were embedded in the second layer of epoxy as an anchor for the rubberized asphalt layer. This wearing surface system performed well until 1983.

When the first wearing surface was no longer serviceable, it was replaced after over 15 years of service. The wearing surface was removed completely to expose bare metal of the deck, and a replacement wearing surface was applied. This second wearing surface was intended to be identical to the original surface. The second wearing surface lasted less than three years. The eastbound lane was replaced for a third time in 1986. This third wearing surface incorporated a proprietary system that included a fiberglass reinforcing mat in the asphalt wearing surface layer. Despite the reinforcing mat, unacceptable amounts of rutting and shoving have necessitated the placement of a new wearing surface during the 1992 construction season. Possible traffic-related reasons for the rapid deterioration of the last two wearing surface systems include: (i) increased tire pressure, (ii) larger allowable gross truck weights, and, (iii) increased traffic volume. Another possible reason for the poor performance of the replacement wearing surfaces relates to conditions under which these wearing surfaces were placed. Compared to the original wearing surface which was placed on virgin metal deck under zero-traffic conditions (negligible stresses and vibrations

during placement), all replacement surfaces were placed on cleaned metal surface while the lanes other than that being placed were subjected to regular traffic.

Given the experience with the replacement wearing surfaces on the Poplar Street bridge, there clearly appears to be a need to identify suitable alternative materials that will meet the increased performance required from present-day wearing surfaces.

Of relevance to some of the discussions to follow are important statistics of the Poplar Street bridge. These are presented in the next section.

1.2 VITAL STATISTICS OF THE POPLAR STREET BRIDGE

The Poplar Street bridge carries three major interstate highways, I-70, I-64, and I-55 across the Mississippi river at St. Louis, Missouri. Approximately 130,000 vehicles including about 15,000 large trucks use this bridge each day. The five-span bridge is 2,165-ft. long and consists of two independent bridges whose total cross-sectional width is 113-ft. The bridges are supported on a set of common piers (Fig. 1.1). Each bridge carries four lanes of traffic in one direction, eastbound or westbound, and is supported by two box girders which have a web spacing of 5-ft. 5-in. and are spaced at 32-ft. 6-in. centers. Box girder depths are 16-ft., except in the center span and over the two central piers, where they are 17-ft. and 25-ft., respectively. The deck plate thickness is typically 9/16-in. The deck plate is stiffened by closed trapezoidal stringers or ribs (Fig. 1.2). The 5/16-in. thick stiffeners on 13-in. centers are 11-in. deep and run along the length of the bridge. Load from the deck is also transferred to the box girders by transverse floor beams spaced on 15-ft. centers. Transverse frames brace the box girders at 60-ft. centers coinciding with every fourth floor beam location, providing the bridge additional torsional rigidity.

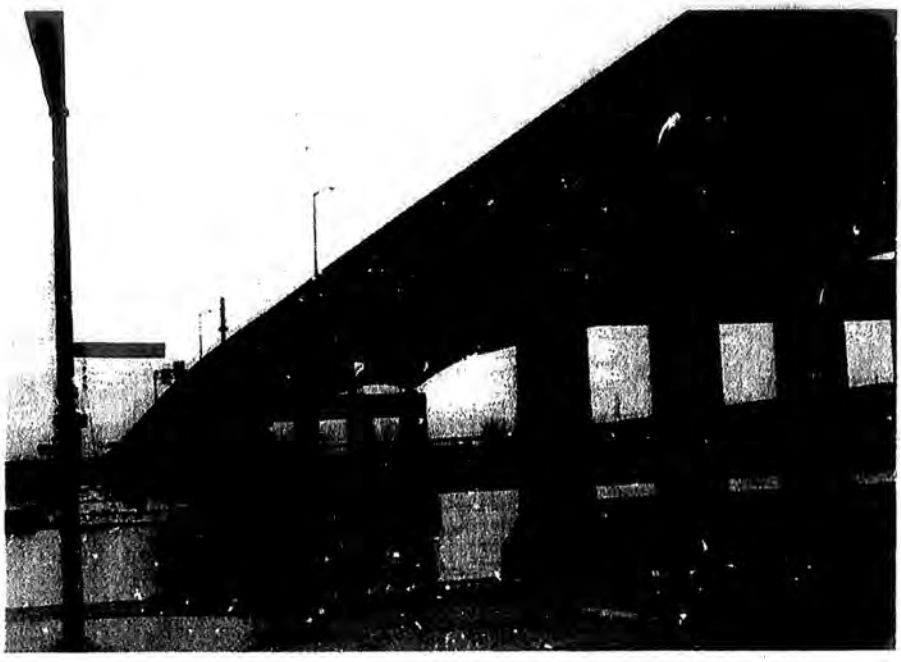


Fig. 1.1 A side-view of the Poplar Street Bridge.

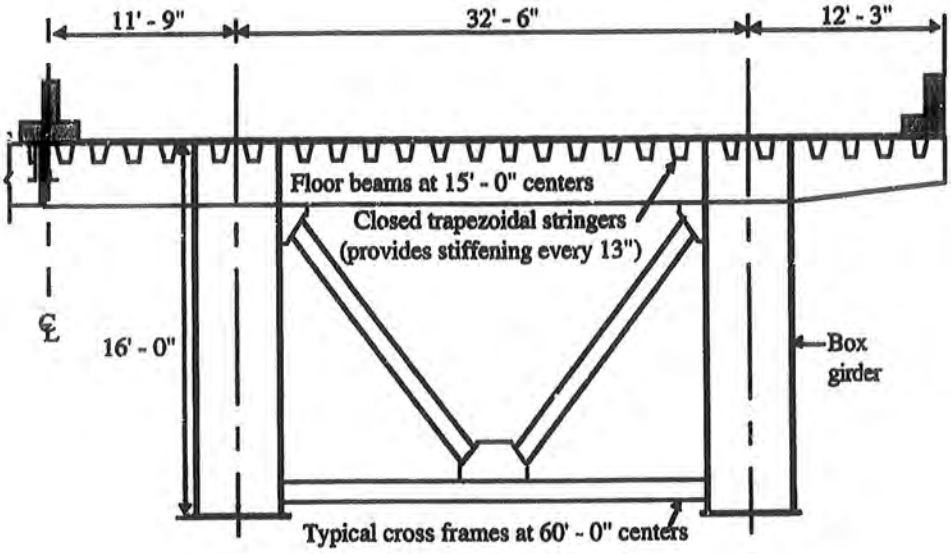


Fig. 1.2 Cross-section of the eastbound bridge showing structural details.

1.3 RELATED WORK

A limited number of articles and reports are available on the performance of wearing surface systems on orthotropic steel-plate bridge decks. Most of them deal primarily with asphalt-based wearing surfaces [4, 10, 12, 15, 19]. Only recently has there been interest on the use of polymer concrete wearing surfaces [2, 5, 6, 9, 11, 13, 14, 16, 17, 21, 22].

Philips [12] prepared an excellent overview of the in-service performance of the different asphalt surface systems that have been used. This first-hand field survey of 15 orthotropic bridges around the world was conducted as a part of the selection process for a wearing surface on the West Gate bridge in Melbourne, Australia. He observed that most (asphalt-based) bridge wearing surfaces which were less than 2 in. in thickness experienced extensive longitudinal cracks. He also concluded that the stiffness of the deck plate influenced the amount and location of these cracks. Longitudinal and lateral cracking was observed by Philips [12] on the virgin asphalt wearing surface on the Poplar Street bridge. Bild [4], notes that longitudinal cracks form at three locations: (i) at paving joints, (ii) over the webs of box girders, and (iii) over the longitudinal ribs. Lateral cracking is normally confined to the area above the piers on a bridge. In addition to longitudinal and lateral cracking, the first three asphalt surfaces on the Poplar Street Bridge experienced severe rutting and shoving. Fig. 1.3 shows a cross-section of the asphalt wearing surface on the Poplar Street Bridge photographed during the placement of field test sections of alternate wearing surfaces (described later in this report). Rutting along the wheel track in this photograph has reduced the thickness of the wearing surface from approximately 2-1/2 in. to 1/2 in. Rutting consists of lateral plastic deformation of the asphalt, while shoving is the longitudinal plastic displacement of the asphalt. Philips [12] believes that the lateral displacement of asphalt surfaces is due to the failure of bond between the steel deck and the wearing surface and not due to the instability of the asphalt mixture itself. However, in the case of the third asphalt surface (Fig. 1.3) on the Poplar Street Bridge, it appeared that bond failure was probably not the cause of the extensive rutting observed. Huber [7] has reported that excessive plastic deformation is likely to occur when the mix is over-

asphalted or has an inadequate aggregate skeleton. This observation is also supported by a review reported by Jones [8] undertaken during this investigation.

Fig. 1.3 A cross-section of the third asphalt-based wearing surface on the Poplar Street Bridge showing rutting along the wheel path

More recently Touran and Okereke [19] summarized the performance of asphalt-based wearing surfaces for orthotropic bridge decks in North America based on responses to survey questionnaires sent to agencies responsible for their maintenance. According to them poor performance of the wearing surfaces could be attributed to adverse climate (freeze-thaw, need for deicing etc.), excessive deck flexibility, excessive traffic loads, and poor paving procedures.

Metcalf [10] conducted laboratory flexural fatigue tests on wearing surface-steel composite specimens to aid in the selection of an appropriate wearing surface for the San Mateo bridge across the San Francisco bay. He observed that the elastic modulus of the wearing surface should be relatively large in order to limit the strain level in the wearing surface of the wearing surface-steel composite system. If the elastic modulus is small, the wearing surface material must have large

failure strain capacity. These observations based on simple mechanics of layered-composite action, were used to explain why conventional asphalt concrete wearing surface performed poorly in flexural fatigue compared to an experimental paving material containing a thermal-setting asphalt-modified resin binder (now known as epoxy modified asphalt or epoxy asphalt).

Siem [15] reports results from flexural fatigue tests conducted at Lehigh University on steel-epoxy asphalt composite specimens for the Luling Bridge in Louisiana. Although the specimen geometry used was similar to that used by Metcalf [10], the specimen dimensions were based on the dimensions of the orthotropic steel-plate deck system of the Luling Bridge. Flexural fatigue tests were conducted at room temperature and at 160°F. The results show that, as the pavement thickness increased from 2 in. to 3 in., the fatigue strength decreased dramatically. The combination of thicker pavement and 160°F test temperature was considered to be too severe a test condition for horizontal shear on the steel-epoxy asphalt interface. Bond failure as a result was a significant factor at these elevated temperatures. At room temperatures, fatigue failures were predominantly due to tensile cracking. The fatigue performance of 2 in. thick epoxy asphalt wearing surfaces made with sandstone aggregates was considered adequate (fatigue life of 5 million cycles without tensile cracking or delamination at the steel-epoxy asphalt interface) for the Luling Bridge.

It should be noted that the tests conducted by Metcalf [10] and those reported by Siem [15] both use loading configurations where the wearing surface is not loaded directly. Loads are applied to the steel plate of the composite specimen. This configuration, similar to the one used in this investigation, is ideally suited to study the potential of the wearing surface to resist tensile cracking and delamination at the steel-wearing surface interface when the composite specimens are subjected to flexural fatigue loads. It is however not suitable to study the shoving and rutting behavior typically observed in asphalt-based wearing surfaces. Plastic deformations of the asphalt-based wearing surfaces result from wheel loads directly applied on the wearing surface. In the present investigation the rutting and shoving behavior of the asphalt-based wearing surfaces were monitored as described later, based on the in-service performance of test sections on the bridge.

Polymer concrete has been used for approximately ten years to overlay or resurface bridge decks. Much of the experience has been with relatively stiff concrete decks. Primary concerns related to use of polymer concrete for (Portland cement) concrete overlay applications include the potential for bond failure, permeability, reflective cracking and long-term durability. Concerns with regard to use of polymer concrete on relatively flexible steel decks include tensile cracking, delamination, and interfacial stresses due to differential thermal expansion of the wearing surface and the steel base-plate. Another concern common with regard to use of polymer concrete overlays on both reinforced concrete and steel decks is the brittle behavior of polymer concrete at low temperatures (typical of winter nights in the midwestern United States). The only other full-scale application of polymer concrete wearing surface to a flexible orthotropic deck known to the authors is that of the Smithfield Street bridge in Pittsburgh, Pennsylvania. The new corrosion resistant aluminum orthotropic deck installed in 1967 used a 3/8 in. thick polyester-sand wearing surface. This wearing surface is reported to have performed well for approximately 16 years [2]. A new epoxy concrete replacement surface was recommended for the bridge after an extensive testing program which included the original wearing surface as well [2].

Average bond strengths measured by Sprinkel [18] on cores of general-purpose unsaturated polyester resins taken from several concrete bridges in Virginia equals approximately 800 psi in shear and approximately 275 psi in direct tension [1, 18]. Pasko [11] reports a tensile bond strength of 625 psi for a methyl-methacrylate-monomer layer bonded to steel.

Sprinkel evaluated the protection provided by the polymer concrete overlay to the deck using two methods. The first method was electric resistivity measurements as specified by ASTM [3]. Another method was used to determine the permeability to chloride ions of 4-in. diameter cores. Resistivity measurements were said to provide a good indication of the extent of cracking, while the permeability test provided information on chloride ion infiltration. The permeability data supported the electrical resistivity data. Both indicated a deterioration in the waterproofing characteristics of the overlay based on measurements made after 2, 24 and 47 weeks of service life.

ALCOA [2] conducted a study of wearing surfaces suitable for aluminum orthotropic bridge decks. The investigation of flexural fatigue performance of wearing surface-aluminum plate composite specimens was a small component of this study which also included tests related to wear resistance, skid resistance, freeze-thaw resistance, accelerated weathering, and corrosion resistance (salt spray and humidity tests). The room-temperature fatigue tests (18.3 Hz) were conducted to determine the wearing surface's susceptibility to cracking. Of the seven different wearing surface materials tested, two polymer (epoxy) concrete surfaces were observed to have the best overall performance indices. Other materials in the test program included an epoxy asphalt, a polyester, a polyester/urethane, a urethane/epoxy and a methacrylate. The urethane/epoxy and epoxy asphalt were judged the next best systems overall, although their fatigue performance was observed to be marginal and unacceptable respectively. To the best of the authors' knowledge, no published information is available on the fatigue performance of polymer concrete wearing surfaces on steel plates.

Polymer concretes and epoxy asphalts are known to exhibit temperature dependent mechanical response. It is hence essential that the fatigue performance of these types of wearing surfaces be evaluated at realistic in-service temperatures. With the exception of the limited fatigue test results reported by Siem [15] at a constant test temperature of 160°F, this kind of information is also not presently available for wearing surface-steel composite specimens. The test program described in the following chapter attempts to address these and other issues important to the choice of a suitable wearing surface system for the orthotropic steel-plate bridge deck of the Poplar Street Bridge.

The upper-limit load to be used in a fatigue test of the wearing surface-steel-plate composite specimen depends upon the maximum in-service stresses experienced by the wearing surface. Computation of actual service stresses experienced by the deck and the wearing surface of an orthotropic steel-plate bridge is complicated by the levels of redundancy in such a structure. According to Wolchuck [20], for design purposes it is common to break-up the orthotropic bridge structure into three interrelated systems: System I - The main bridge system, with the deck acting

as a part of the main load carrying members of the bridge. In the computation of stresses in this system the effective cross-sectional area of the deck (including the longitudinal ribs) is considered as flange. System II - The stiffened steel-plate deck (acting as a bridge floor between the main members) consisting of the ribs, the floor beams, and the deck plate as their common upper flange. System III - The deck plate, acting in local flexure between the ribs, transmitting the wheel loads to the ribs. The local stresses in the deck plate act mainly in the direction perpendicular to the supporting ribs. The stresses attributed to this third system that subjects the wearing surface to tensile stress over the longitudinal stiffeners is of primary interest in the present investigation. Stresses in the deck plate are calculated as for a continuous plate strip supported by ribs that are assumed to be rigid.

Ishikawa-Narima-Heavy Industries Co. Ltd. (steel fabricators for the Luling Bridge project) in the only investigation known to the authors where strain measurements were made on the deck plate of an orthotropic steel-plate deck, measured the strains in a fabricated girder segment of the Luling Bridge [15]. Strain gages and deflection gages were attached to the deck plate while the deck was subjected to five different types of loading. Maximum negative moment stress of approximately 21 ksi was recorded over the web plate. Fatigue tests conducted for the Luling Bridge used an upper limit load that produced the same level of stress as measured by Ishikawa-Narima Industries Co. Ltd. The relation of these five loading types to actual in-service loads is not described in Siem's report [15]. Also, this static test was conducted without the wearing surface in place. The composite action with the wearing surface in place is expected to reduce the maximum stress recorded. No previous published information is available on actual in-service strains in such orthotropic steel-plate bridge decks. One component of the present investigation addresses this issue [5, 6].

1.4 OBJECTIVE AND SCOPE OF THE STUDY

Research reported here was conducted for the Missouri Highway and Transportation Department (MHTD). The main objective of the investigation was to evaluate the performance of

several wearing surface materials for use on the orthotropic steel-plate deck of the Poplar Street Bridge. The wearing surface materials investigated were selected by the MHTD and include an epoxy asphalt concrete, a polymer-modified asphalt concrete, three epoxy concretes and a methacrylate concrete [13, 14, 21, 22]

While rutting and shoving are primary concerns with the asphalt-based wearing surfaces, the potential for cracking and delamination are primary concerns with the polymer concrete and methacrylate wearing surfaces. In line with these concerns, a five-component study was conducted by the University of Missouri-Columbia (MU) in collaboration with the Missouri Highway and Transportation Department (MHTD). Details of these components and results from the various tests conducted during this investigation are presented and discussed in the following chapters of this report.

CHAPTER 2 - DETAILS OF THE RESEARCH PROGRAM

2.1 OVERALL EXPERIMENTAL PROGRAM

A five-component experimental program was designed to accomplish the main objective of this investigation. These include (i) temperature cycled and cold temperature laboratory flexural fatigue tests on steel-plate wearing surface composite specimens, (ii) field measurements of in-service strains on the bridge deck, (iii) monitoring of test sections on the bridge subjected to normal service loads and weathering conditions, (iv) laboratory resistivity tests to evaluate the damage to the wearing surface before and after the fatigue tests, and (v) laboratory core pull-out tests to characterize the tensile bond between the steel plate and the wearing surface. The flexural fatigue tests constitute the only major laboratory component of the investigation. Two major field components include deck strain measurements and the monitoring of test sections on the Poplar Street Bridge. Resistivity tests and pull-out tests constitute two additional minor laboratory components that were used to develop support data for the overall study.

The wearing surface materials evaluated were selected by the MHTD and included: an epoxy-modified asphalt concrete (epoxy asphalt supplied by Adhesive Engineering Co.), a polymer-modified asphalt concrete (polymer-asphalt supplied by Elf Asphalt, Inc.), three epoxy concretes (Transpo T-48 epoxy binder system supplied by Transpo Industries, Flexolith epoxy binder system supplied by Dural International Corp., and Polycarb epoxy binder system supplied by Polycarb Industries), and a methyl methacrylate concrete (Degussa 330 MMA supplied by Degussa Corp.). With the exception of the asphalt-based wearing surface materials, the other materials were applied in conjunction with three different deck plate surface conditions. The surface conditions studied were (i) bare steel (sandblasted), (ii) sandblasted steel protected by a coating of water-based zinc paint (IC 531), and (iii) sandblasted steel protected by a coating of Carbozinc 11 paint. The asphalt materials were applied only to the two types of zinc coated plates.

The University of Missouri-Columbia (MU) was responsible for the first two components of the experimental program described above. Data and observations from the tests conducted by

the MHTD Division of Materials and Research [16, 17, 20, 21] are used while discussing the performance of test sections on the bridge. Resistivity and pull-out tests were conducted on laboratory fatigue specimens by MU researchers. Similar data was generated by the MHTD Division of Materials and Research for the test sections on the bridge [16, 17, 21, 22].

2.2 FLEXURAL FATIGUE TESTS

The flexural fatigue tests described here were used to evaluate the wearing surfaces with regard to their resistance to tensile cracking and delamination. These tests were ideally suited to study the performance of the polymer concrete wearing surfaces where cracking and delamination are the two most likely modes of failure. Additional and more relevant failure modes for the asphalt-based wearing surfaces included rutting and shoving, which these tests were not capable of simulating. These failure modes were however adequately captured in the test sections on the bridge subjected to actual loading and weathering.

Three types of flexural fatigue tests were conducted during the course of this investigation. These were: (i) Temperature cycled flexural fatigue tests, (ii) Cold temperature flexural fatigue tests, and (iii) Flexural fatigue tests on joint specimens.

The mechanical response of asphalt-based and polymer concrete materials are typically very sensitive to the testing temperature. In order to simulate actual service conditions, the main series of flexural fatigue tests was conducted in a temperature cycled environment. This series was termed "Temperature cycled flexural fatigue tests".

After the MHTD concluded that the asphalt-based wearing surfaces were unacceptable [21, 22] based on the performance of the test sections on the bridge, a series of flexural fatigue tests was conducted on the remaining polymer concrete wearing surfaces at a constant temperature of 0°F. These tests were necessitated by the fact that these materials behave in a very brittle manner at very cold temperatures. This series was termed "Cold temperature flexural fatigue tests".

The placement of large areas of polymer concrete wearing surface on a steel deck requires the use of construction joints. There was some concern about the fatigue performance of such

construction joints, particularly since fresh epoxy resin is known to bond very poorly with previously placed hardened epoxy. A series of temperature cycled flexural fatigue tests was designed using specially fabricated specimens to simulate construction joints in the wearing surface. This series of tests conducted only on one polymer concrete material was termed "Flexural fatigue tests on joint specimens".

2.2.1 Specimen geometry and fabrication: All the test specimens used for the laboratory flexural fatigue tests consisted of 4 x 15 in. steel plates 9/16 in. thick (same thickness as the bridge deck) with the requisite thickness of the surfacing material (2 in. for the asphalt materials and 3/8 in. for the polymer concrete and methacrylate materials) applied to one surface of the steel plate. Bearing plates, 6 x 2 x 1/2 in. were welded across the bottom of this plate at each end, on 13 in. centers so that they extended 1 in. on each side along the width of the base plate (Fig. 2.1). These extensions served as bearing surfaces for the four 4-1/2 in. rockers that supported the specimen. With this arrangement, the base plate simulated the section of the deck plate centered over a longitudinal stiffener. Load was applied through a 7/8 x 7/8 x 4 in. steel bar glued to the bottom of the plate at midspan. The pavement surface was, as a result, subjected to maximum flexural tensile stresses at midspan, simulating conditions similar to those the surface would experience from actual service loads.

The test plate surface on which the wearing surface was to be placed was sandblasted at the same time the field test sections on the bridge were sandblasted using the same equipment and procedures as used for the field test sections. The requisite number of plates were coated with water based zinc paint (IC 531) and Carbozinc 11 paint to the same specifications as recommended for the field test sections.

The epoxy asphalt surface was placed on the treated steel base plates by the manufacturer (Adhesive Engineering Co.) in their laboratory, using techniques identical to those used by Metcalf [10]. The polymer-asphalt surface was placed on the treated steel base plates by the MU research team using the asphalt laboratory of the MHTD Division of Materials and Research at Jefferson

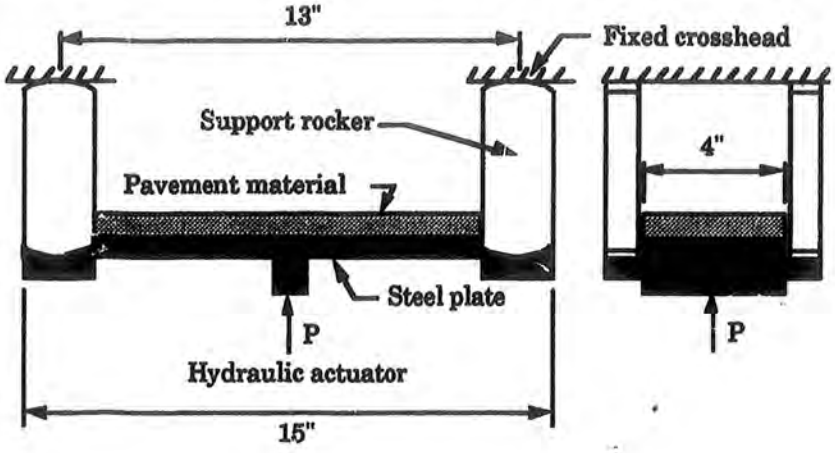


Fig. 2.1 Composite specimen showing the loading configuration used for the flexural fatigue tests.

Fig. 2.2 Polymer-modified asphalt concrete being placed into the heated molds just prior to compaction.

City, Missouri. Elf Asphalt, Inc. supplied the Styrelf 14-60 asphalt. MHTD furnished the aggregates. The aggregates were from the same batch as used on the test sections on the bridge. A tack coat of neat Styrelf asphalt was applied to the steel plate. The asphalt mix (as recommended by the manufacturer Elf Asphalt, Inc.), the steel plate with the tack coat, steel mold used to fabricate the composite specimen, and the loading platten used to compact the asphalt mix were heated in an oven to 300°F. After assembling the mold around the steel plate, the asphalt mix was placed in the mold (Fig. 2.2) and the steel loading platten was placed on top of it. A load of 200,000 lb. (stress level of 3,333 psi) was applied to the asphalt concrete and maintained for three minutes. The asphalt concrete-steel composite specimen was allowed to cool before demolding it.

Epoxy concrete and methacrylate concrete wearing surfaces were applied to the fatigue specimen base plates at the same time the field test sections were placed. Plastic molds fabricated at MU were used to place these wearing surfaces on the fatigue specimen base plate. All wearing surfaces were placed according to the respective manufacturer's specifications. Procedures used were identical to those used on the field test sections. Typically this involved providing the steel plate with a tack coat of the neat epoxy resin. The polymer concrete layer was placed when this coat was tacky (Fig. 2.3). Aggregates were broadcast over the polymer concrete when this layer was adequately stiff. This provided the traction needed from the wearing surface (Fig. 2.4). Loose aggregates were brushed off before a seal coat of methyl-methacrylate was applied to waterproof the wearing surface.

Polymer concrete specimens were also made to simulate two different types of construction joints. These specimens were generally cast using the same procedure described above for the regular polymer concrete specimens. The only difference was that these specimens were cast in two stages. The wearing surface was first applied to half the length of the steel base plate. A header used across the width of the specimen to contain the polymer concrete on one end was removed ten minutes after placement of the first section. At the time the header was removed, aggregates were broadcast on to the exposed edge of the wearing surface. This edge provided both shear

Fig. 2.3 The polymer concrete mix is poured into the mold.

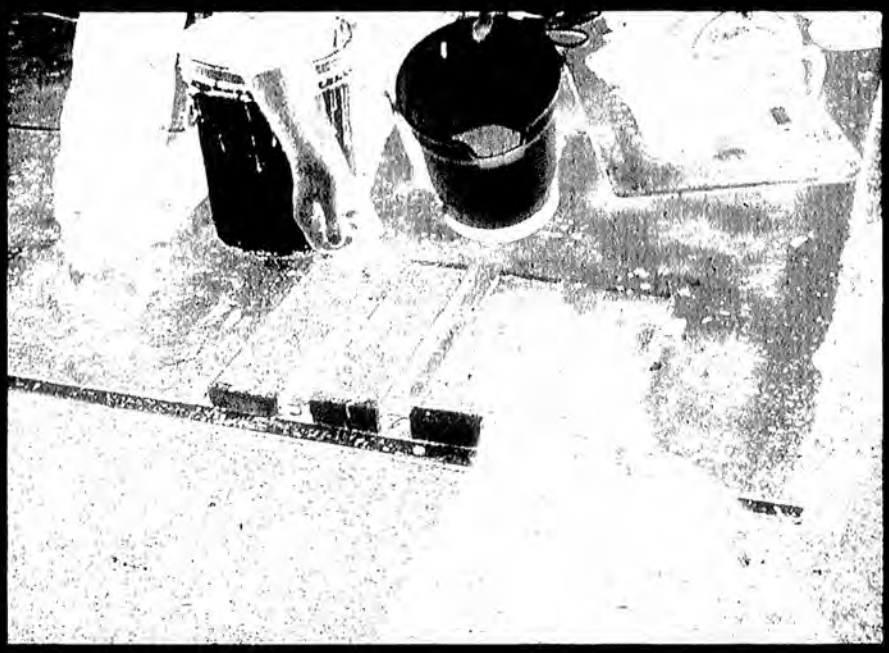


Fig. 2.4 Aggregate is being broadcast on top of the polymer concrete layer.

transfer capability and tensile strength across the joint. The wearing surface was allowed to cure for approximately one hour before placement of the second half. Two different approaches were used for preparing the joint. An epoxy tack coat was applied to this edge before placement of the second section of the wearing surface for one-half of the specimens. These joint specimens were appropriately labeled "TC" (joints with an epoxy tack coat). The other half of the specimens were not provided with this tack coat. These joint specimens are identified as "NC" (no coat) in the later discussions. All joint specimens for the laboratory fatigue tests were fabricated on the bridge deck at the same time field test sections for similar joints were placed.

2.2.2 Temperature cycled flexural fatigue tests: Temperature cycled flexural fatigue tests were designed to take into account the fact that the wearing surface materials are not only sensitive to the test temperature but are also sensitive to the history of the thermal loading they are subjected to. This is particularly true for the polymer concrete. Tests were conducted at a 5 Hz sinusoidal loading. The temperature in the test chamber was also simultaneously varied sinusoidally in real time to reflect the day-night summer and winter deck temperatures. The summer-time temperatures varied between 165°F and 45°F, and the winter-time temperatures varied between 70°F and 0°F. A periodic block of two-and-a-half day-night cycles of summer temperatures followed by two-and-a-half day-night cycles of winter temperatures was incorporated in the temperature control program. Fig. 2.5 illustrates the time variation of temperature during these flexural fatigue tests. Also included in the figure is the corresponding number of fatigue cycles assuming uninterrupted testing at a 5 Hz rate of loading. The fatigue specimens were typically tested until failure or for a maximum fatigue life of 5×10^6 cycles, whichever occurred first.

Two different upper limit loads were used during the course of this investigation. In the first group of tests, an upper limit load of 1,450 lbs. was used. Maximum in-service stress on the wearing surface for the Poplar Street Bridge was not known prior to the start of this test program. Since the deck plate design was comparable to that of the Luling Bridge, the upper limit load used for the fatigue tests conducted for the Luling Bridge [15] was initially used for this investigation as

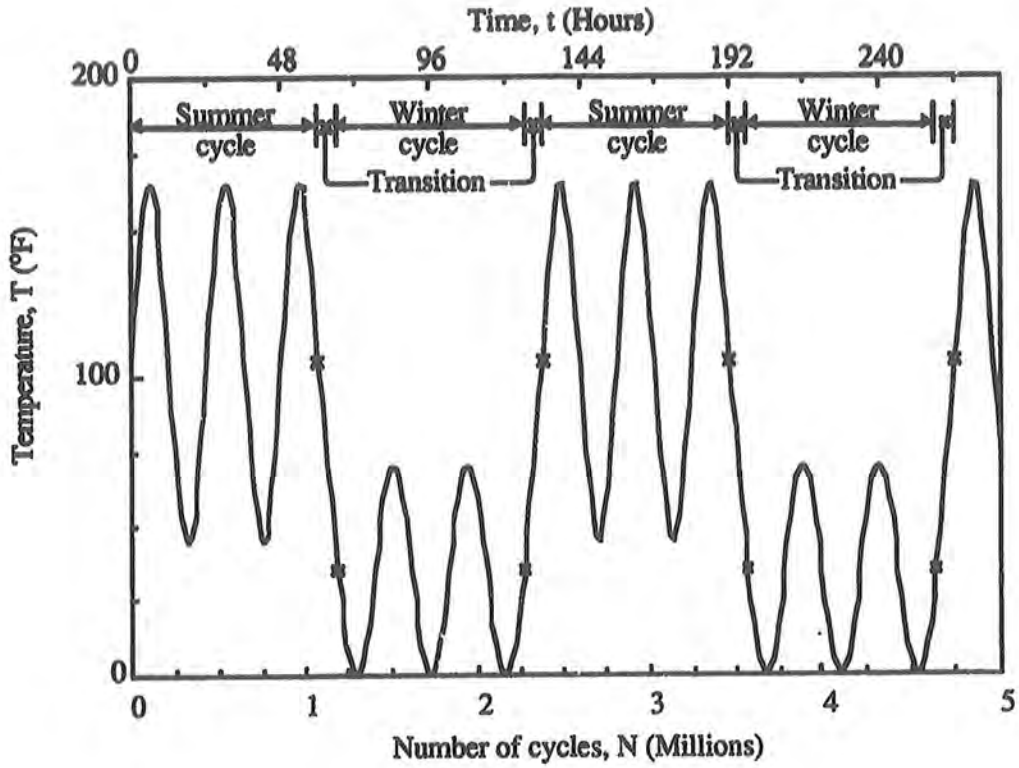


Fig. 2.5 Typical temperature variation with time. Corresponding number of fatigue cycles at a rate of 5 Hz of uninterrupted testing is also shown.

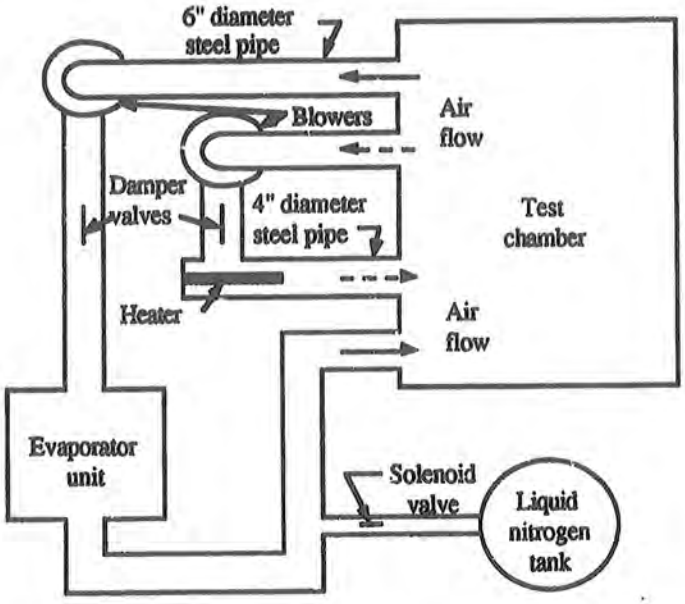


Fig. 2.6 Schematic of the test chamber and the air flow ducts.

well. The upper limit load of 1,150 lbs. used for the second group of temperature cycled flexural fatigue tests was based on the results from cold temperature flexural fatigue tests. A nominal lower limit load of 150 lbs. was used for all the flexural fatigue tests so as to retain the specimen and test fixtures in their places during unloading.

Fatigue specimens were tested in a 220 kip capacity servo-controlled electro-hydraulic testing machine. Special fixtures were fabricated for the set-up so that four specimens could be tested simultaneously. The entire loading fixture was enclosed in an insulated test chamber that allowed tests to be conducted using temperatures in the range 0° to 165° F. The heating and cooling duct-work connected to the temperature controlled test chamber was equipped with blowers, and damper valves. An electric heating element was provided in the heating circuit while an evaporator unit and a tank of liquid nitrogen were used in the cooling circuit (Fig. 2.6). A solenoid valve was provided to control the flow of liquid nitrogen. Two personal computer (PC) -based systems were developed for this investigation. One system served to control the temperature in the testing chamber during the two-week long fatigue tests. The other system facilitated automation of data acquisition and processing for the fatigue test specimens. Details of these systems and associated software developed are described in Section 2.2.5.

Load, slip between the steel plate and the wearing surface layer at the two ends of the composite specimen, specimen midspan deflection on each side of the specimen, and wearing surface temperature were monitored for each specimen during the fatigue tests. Fig. 2.7 shows a close-up of the test chamber with the specimens in place. In addition to the above measurements, temperature in the test chamber and the heating/cooling duct-work were measured at five different locations. These additional measurements allowed closed-loop temperature control using the PC-based system.

2.2.3 Cold temperature flexural fatigue tests: These tests were conducted at a constant temperature of 0°F. The lower limit load for these tests was the same as used for the temperature cycled flexural fatigue tests. Starting at 300 lbs., the upper limit load was incremented by 150 lbs.

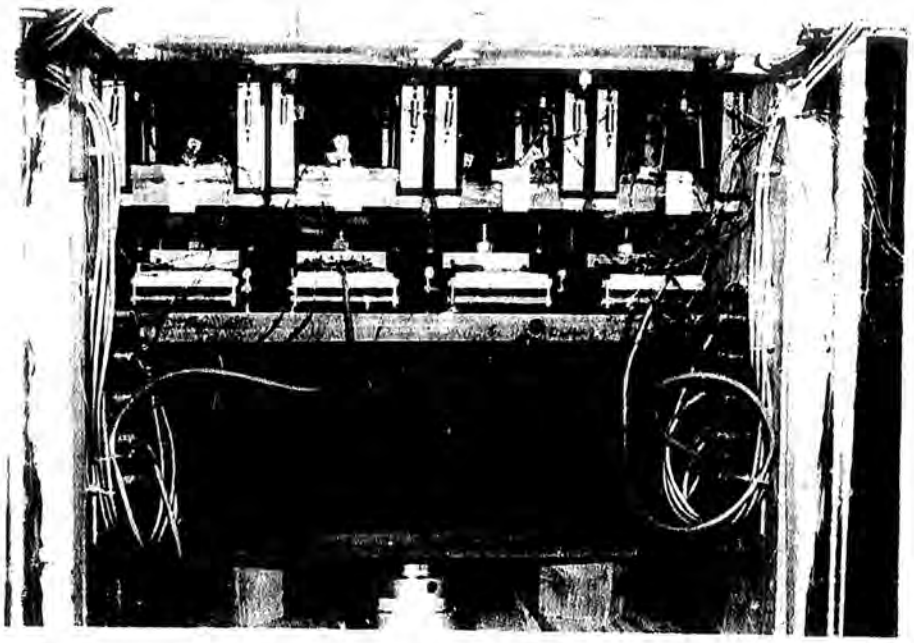


Fig. 2.7 Close-up of the test chambers showing the four specimens and associated instrumentation in place.

every 70,000 cycles until the specimens failed. Parameters monitored during these tests were identical to those monitored during the temperature cycled flexural fatigue tests. The tensile stress in the wearing surface at which the wearing surface first failed (cracked and/or delaminated) when being simultaneously subjected to a 0°F environment was computed from these tests. Cold temperature fatigue tests were conducted only for the three epoxy concrete and one methacrylate materials. As in the temperature cycled flexural fatigue tests, four specimens, two each of two different types of wearing surface materials were tested simultaneously.

2.2.4 Joint specimens subjected to flexural fatigue: Temperature cycled flexural fatigue tests were conducted on two TC and two NC type joint specimens in each of two rounds of testing. Parameters measured were identical to those measured in the other two types of flexural fatigue tests described earlier. In addition, a special procedure was developed to detect cracking in these specimens. Two wires were glued along the length of each specimen on top of the wearing

surface (approximately 3/4 in. away from each side along the specimen width). The electrical continuity of these wires was monitored continuously, and break in the continuity indicated cracking. This method was more effective than detecting cracking solely through stiffness changes in the specimens.

2.2.5 Test apparatus and procedures:

Test Procedures and Instrumentation: The fatigue specimens were tested in a 220 kip capacity servo-controlled electro-hydraulic testing machine. Special fixtures were fabricated for the set-up so that four specimens could be tested simultaneously in parallel. A relatively flexible aluminum load cell was installed in series with each of the four specimens. Thus it was possible to approximately simulate a load control mode of loading for each specimen while actually using a ram displacement controlled mode of loading. Displacement amplitude of approximately 0.1 in. produced the upper limit stress level desired. A 5-Hz command signal was used for all the flexural tests.

The simply supported beam-type load cells were made of 7075-T6 aluminum plates. They were designed to be soft so as to minimize changes in load application with changes in specimen stiffness. This also ensured high resolution for the load measurements. Designed as a full-bridge circuit, each load cell was calibrated statically prior to the fatigue tests.

Two end-slips (one on each end of the specimen) and two midspan deflections (one on each side of the specimen at midspan) were monitored for each specimen. These measurements were made using strain gage based clip-on gages specially fabricated for this purpose. During the later part of the investigation, a modified spring-loaded beam-type fixture was used to measure midspan deflection. This change was necessitated by the fact that the clip-on gages tended to bounce out of alignment during fatigue loading. All of the deflection measurement devices used a full-bridge circuit to provide large signal output.

Iron-constantan thermocouples with an ice-water reference junction were used for all the temperature measurements. Besides temperature measurements on the wearing surfaces of the four specimens, test chamber temperature was measured at several locations as described later.

Two extra-fine wires were glued to the top of the wearing surface of the joint specimens. By monitoring for a break in the electrical continuity of these wires, it was possible to determine when the construction joint cracked. Each wire on the specimen was connected in series to a small resistor. The two wire-resistor pairs were then connected in parallel. By applying a small current to the circuit and monitoring the voltage across each resistor with a digital storage oscilloscope, the exact time when either of the wires lost continuity could be established.

The Temperature Control System: Temperature in the test chamber was controlled using a dedicated personal computer based system. The computer was equipped with a general purpose data acquisition board (LabMaster by Scientific Solutions, Inc.). The analog to digital converter (A/D) and the digital input/output features of the board were used for the temperature control system. The board's functions were controlled using a compatible software driver (Labpac 2.0, also from Scientific Solutions, Inc.). The temperature control program developed for this study was written in Microsoft QuickBasic 3.0. This application program made calls to the Labpac subroutines that were memory-resident once Labpac was executed. These subroutines allowed analog sweep initialization, analog input, and digital output required for the temperature control program.

Air temperature at the inlet of the test chamber, outlet vent temperatures in the cooling and the heating circuits, temperature of the coils of the refrigeration unit, and temperature in the four specimen wearing surfaces were monitored using iron-constantan thermocouples. Analog signals from all the thermocouples were input into the A/D input terminals. The digitized voltage outputs were then converted to temperature signals based on the bilinear voltage-temperature relationship for iron-constantan thermocouples. Depending upon whether the test was in the cooling or the heating cycle, the voltage signal from one or the other thermocouple at the outlet vents was used as the feedback parameter for the closed-loop temperature control system. The temperature from the

thermocouple controlling the system was compared to the prescribed command temperature history once every fifteen seconds. Corrective action was taken to ensure that the feedback signal followed the prescribed command signal at all times. This action involved sending command signals to the various temperature control devices through the digital I/O lines. These devices include: two damper valves, two blowers, a heater, a refrigeration unit and a solenoid valve (Fig. 2.6).

The command function for temperature control was based on actual peak summer and winter air temperatures recorded at the Lambert International Airport (St. Louis), which is located approximately fifteen miles from the bridge. The air temperature values were used to estimate the variations in deck temperature. The command function comprised two-and-a-half real-time day-night temperature cycles (totally 60-hour, sinusoidal) representing a summer period, a six-hour linear transition, two-and-a-half real-time day-night temperature cycles (60-hour, sinusoidal) representing a winter period, and a six-hour linear transition, giving a 132-hour repetitive temperature control block. The lower and upper limit temperatures for the summer month were 45°F and 165°F respectively. Similar values for the winter month were 0°F and 70°F respectively. The hourly temperature values of the command function were stored in a tabular form (for 132 hours). Intermediate command temperature values were obtained from the hourly command function table by linear interpolation.

Special features of the program allowed for holding (pause feature) the temperature at desired levels through an interrupt facility, resuming from the pause feature, and restarting the temperature control at any point on the command function. These features allowed for inspection of the specimen and/or instrumentation during the fatigue test cycle.

In addition to providing temperature control of the test chamber during the fatigue test, this PC-based system also allowed the acquisition of the temperature histories of the various thermocouples at user specified intervals.

<u>The Data Acquisition System</u>: The data acquisition system was similar in hardware configuration to that used for the temperature control system. The personal computer was equipped with a LabMaster board. The application program was written in Advanced Basic mainly because

an existing program written for an earlier fatigue investigation at MU was modified to customize the program for this project. Labpac 1.0 was used in conjunction with the LabMaster board because this version (unlike that used for the temperature control system) provided some features that were felt necessary for the applications program that were not available in Labpac 2.0 (e.g. count of zero crossings to determine the actual frequency of the fatigue loading). The A/D converter and digital I/O port of the LabMaster board were used for the features incorporated in the applications program.

Load, midpoint deflection on both sides of the specimen, slip between the wearing surface and the steel plate at the two ends of the specimen, and wearing surface temperature were measured for each of the four specimens. For each specimen these six analog signal outputs were fed into a multiplexer box. One multiplexer box was dedicated to each of the four specimens. The output line from each of the multiplexer boxes was connected to one of the eight inputs to the data acquisition system. Multiplexer channels were selected by sending the binary bit value of the channel number through the digital output port of the data acquisition computer. Output lines from the port were daisy-chained so that all four multiplexer boxes selected the same channel number (for example, all multiplexers switched to connect the load signal to the A/D converter, or all multiplexers switched to connect the North midspan deflection signal to the A/D converter). One of the four load signals was designated as a master channel for reference frequency of the fatigue loading and for comparison purposes. Based on the frequency of the prescribed master channel signal, a sampling rate that would provide thirty points per fatigue cycle was determined. During any data acquisition step, the data sampling rate was first established. Each multiplexer channel was sampled thirty times before switching to the next multiplexer channel. A total of thirty points for each of the mechanical performance parameters monitored for each of the four specimens were stored as raw data during each data acquisition step. Data could be acquired by the operator at anytime during the fatigue test. Data acquisition procedure could also be executed at regular operator specified intervals using a special batch file. This automated mode of data acquisition was usually used for overnight data acquisition when the fatigue test was left unattended. Data acquisition intervals used

in such an automated acquisition mode were typically between two and four hours. An autoincrementing procedure allowed generating data filenames based on the file name used in the previous data acquisition step. Every raw data file stored additional information associated with the data which included test date, the time of data acquisition, cycle count for the fatigue test, temperatures at the various locations in the test chamber, information with regard to the master channel, and operator comments if provided.

Given the large volumes of data that can be generated from a fatigue test lasting approximately two weeks, an automated scheme of data reduction was also used to monitor results from the fatigue test immediately after data were acquired. The post-acquisition data reduction program fitted a sine curve to each of the thirty points recorded for each parameter in the data acquisition step. Values of the double amplitude of the sine curve, phase shift and bias with respect to the master signal were stored and printed out for each of the parameters monitored. Also printed out were stiffness values of the specimen (ratio of the double amplitude of the load signal to the average value of the double amplitude of the two midspan deflection signals). This condensed form of data presentation allowed for monitoring of the status of damage to the specimen (cracked and/or delaminated), and for taking appropriate follow-up action. This also allowed for prompt follow-up action in case of instrumentation malfunction.

2.3 DECK STRAIN MEASUREMENTS

The first group of temperature cycled flexural fatigue tests were conducted with an upper load limit of 1,450 pounds. This loading, as stated earlier, was chosen based on the upper limit load used for the fatigue tests conducted for the Luling Bridge project [15]. At this upper load limit the flexural fatigue specimens tested for the Poplar Street Bridge all failed at the first exposure to the extreme cold portion of the temperature cycle (0°F). It was not known if the load of 1,450 lbs. produced stresses representative of the actual maximum in-service stress in the wearing surface on the Poplar Street Bridge. Cold temperature flexural fatigue tests were developed, which as discussed later in Chapter 3, indicated that the maximum upper limit load sustainable at the

coldest temperature was 1,200 lbs. A second group of temperature cycled fatigue tests was then conducted with an upper limit load of 1,150 lbs. Subsequently, a series of field deck strain measurements were made to estimate the actual stresses to which the wearing surface would be subjected. These in-service stresses were then compared to stresses applied in the laboratory flexural fatigue tests to determine if the wearing surface would crack under service conditions. Results of this comparison are presented and discussed later in Chapter 3.

2.3.1 Strain measurement locations: Simple analyses and some preliminary experiments were performed to determine the critical locations where the strain gages should be placed to facilitate computation of the maximum stress levels in the wearing surface. The field strain measurements, as a result, were made inside the southernmost box girder on the eastbound lanes of the bridge. Fig. 2.8 is a photograph inside the box girder where the strain measurements took place. Strain gages were placed on three transverse gage lines located at or near the middle of the western-most box girder span. Three 3-gage rosettes were placed on each transverse line for a total of nine active gages per line. The first transverse line of gages was located at the midspan of the first box girder span. This line was 150 feet east of the west abutment and cemented within 1 inch of a transverse floor beam. A second line of gages was located 7 feet 6 inches east of the first transverse line. This line was affixed midway between two of the transverse floor beams. The second line was between the same two floor beams as the first line of gages. Thirty feet east of the second line was the third transverse line of gages. This line was also placed midway between two transverse floor beams.

Each gage on the rosette was oriented to measure strain in one of the following directions, longitudinal, transverse, or 45° (with respect to the longitudinal and/or transverse) Three passive strain gages were connected to each active gage in a Wheatstone bridge configuration. All active and passive strain gages used had foil gage elements and a nominal resistance of 120 ohms. Fig. 2.9 shows two strain gage rosettes fixed to the underside of the deck plate and the 18 passive gages which were cemented to small steel plates and connected to screw terminal strips. The

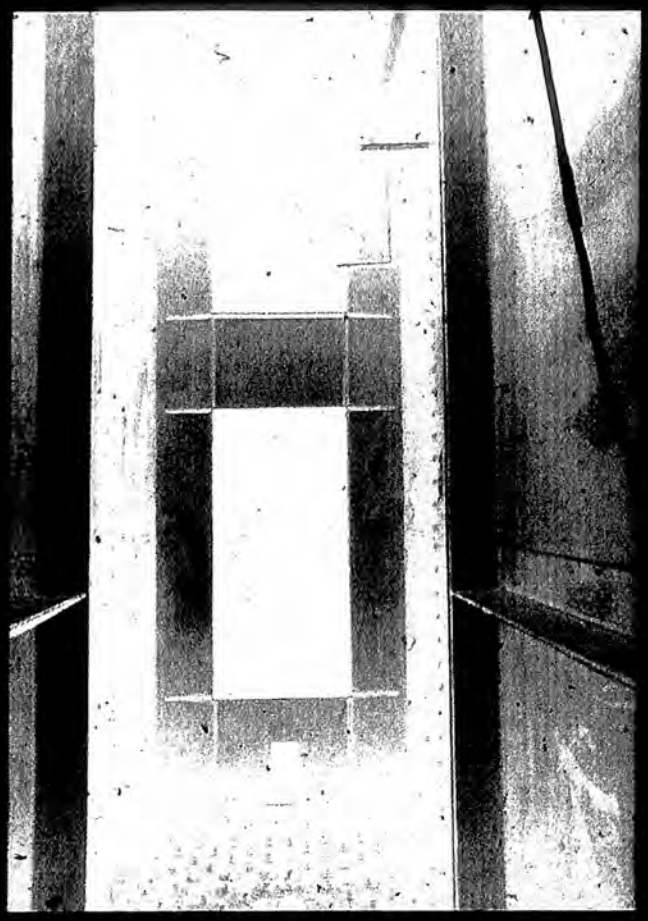


Fig. 2.8 View of the underside of the deck plate inside the southernmost box girder.

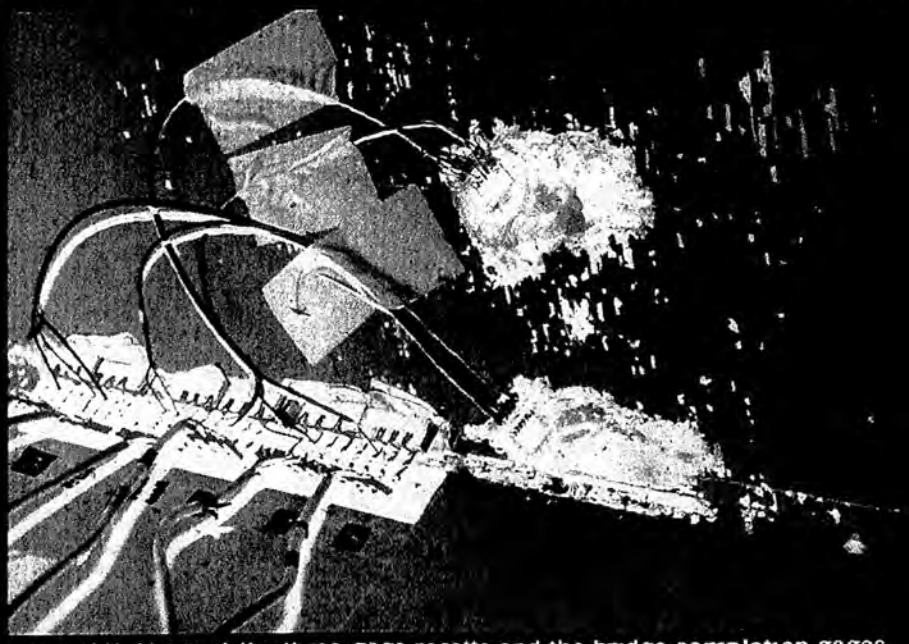


Fig. 2.9 View of the three-gage rosette and the bridge completion gages.

photograph in Fig. 2.10 shows a close up of one three-gage rosette. One rosette was placed midway between the inner box girder web and the first stringer. A second strain gage rosette was placed within one inch of the second stringer. The third strain gage rosette was cemented midway between the second and third stringer. Fig. 2.11 describes the location with respect to the box girder webs for each of the three rosette gages per gage line.

2.3.2 Instrumentation and data acquisition equipment:

Computer: Data were collected and processed using a PC-based automated test set-up. A portable PC formed the heart of the system. It was equipped with two 5-1/4 inch disk drives, one of which was a high density drive.

Hourly data files and Microsoft QuickBasic libraries were stored on the 1.2 MB high density diskette. The 360kB double density disk was used to store the program developed in this study. This program was used to acquire and process continuous strain histories from the service loads on the bridge. An eight channel differential input data acquisition board was used for the analog to digital (A/D) conversion. The A/D converter had a maximum sampling rate of 40 kHz (5 kHz per channel) and a resolution of 12 bits (LabMaster, Scientific Solutions, Inc.).

Strain Gages and Conditioning Equipment: Nine 3-element rosettes were used for the deck strain measurements. Each rosette had foil gage elements that were at 0°, 45° and 90° with respect to the axis of the rosette. All three of these gages had a nominal resistance of 120 ohms. The rosette arrangement allowed verification of the principal strain directions on the deck. Each of the 27 active gages was provided with a set of three external dummy gages (foil gages, 120 ohm nominal resistance) for bridge completion.

A ten-channel amplifier (Vishay 2100, Micro-Measurements, Inc.) unit was used to power the strain gages, provide balancing and calibration capability, and amplify the signal from the strain gages. A 4 V d.c. signal was used to power the full-bridge circuit. The amplifier provided convenient ± 10 V d.c. output signals from the gages

Fig. 2.10 Close-up view of one of the strain gage rosettes.

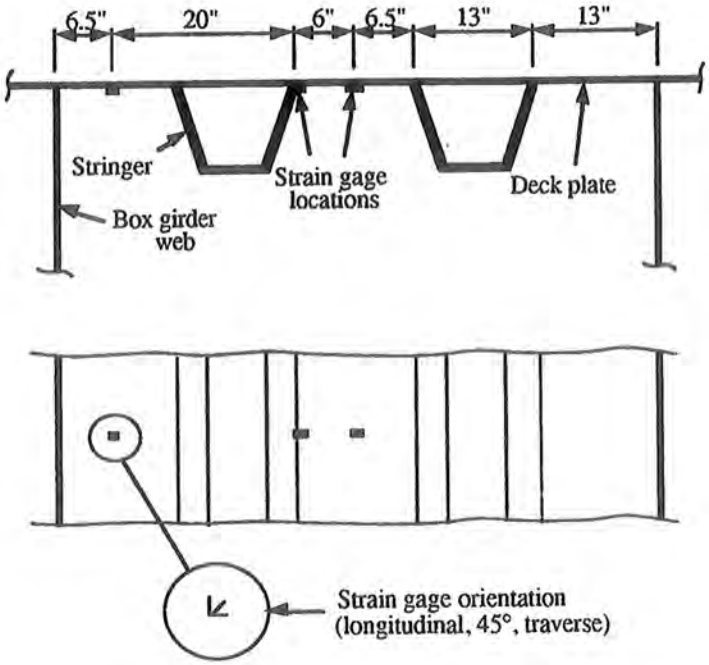


Fig. 2.11 Schematic showing the location of the gages with respect to the box girder webs.

2.3.3 Test Procedures: Simple analyses and some preliminary measurements with a digital storage oscilloscope allowed the determination of the critical locations at which the strain gages should be placed in order to facilitate the computation of the maximum stress levels in the wearing surface. A total of nine strain gage rosettes were glued to the underside of the deck-plate (Fig. 2.11). Three rosettes each were glued along three transverse sections along the length of the bridge. These sections were immediately under the polymer concrete test sections used for another component of this study. Two transverse sections were located midway between the floor beams (which were spaced on fifteen foot intervals) while the other transverse section was placed within one inch of the floor beam. The three gages on each rosette were oriented so as to measure strain in the longitudinal, transverse and 45° (with respect to the longitudinal and/or transverse directions) directions. After monitoring all 27 gages glued on the deck-plate with a digital storage oscilloscope, three gages were determined to be the most important for computing the maximum strains experienced by the wearing surface. These were the three transverse strain gages midway between two floor beams (Gages 12, 15 and 18).

Even though the three passive gages of the full-bridge arrangement used for each active gage were located in the proximity of the active gage (Fig. 2.9), the circuit did not provide complete temperature compensation. This was because the passive gages were not subjected to the same temperature as the active gage. The drift due to temperature variations during the day, as a result, had to be compensated for in the computer program.

Output signals from the strain gage conditioning unit were split into two lines so that the strain histories could be input to both, the digital storage oscilloscope and the PC. This was done for the early tests so that the data processing features of the computer program could be validated. Gage calibrations were accomplished using the shunt resistor built into the strain gage conditioning unit. The instrumentation and amplification used provided a system resolution of approximately 1 $\mu\varepsilon$ (1 V = 244 $\mu\varepsilon$).

A battery powered clock was installed to ensure that correct time was stored along with the hourly file written on the diskette even if the power failed. A batch file allowed the initialization of the data acquisition board and restarting of the program in the event of a power failure.

2.3.4 Data acquisition software and its features:

The program was written using Microsoft QuickBasic (Version 3.0) and Labpac subroutines (Scientific Solutions, Inc.). Labpac enabled the QuickBasic program to interface with the LabMaster data acquisition board (also by Scientific Solutions, Inc.). The basic functions of the program included: collection of strain gage data, sorting of the data, smoothing of the waveforms, finding local maximums for each sweep, incrementing the counters used to identify the maximum strain levels associated with each maximum, checking to see if an hour has elapsed since last data storage, and, if so, to store the current counters and initiate a new sweep.

Data Collection: First, the program initialized Labpac subroutines and all the variables used. The starting time of the program was read next and stored. Once the data acquisition board was initialized, collection of data could begin. Three strain gage channels were sampled every four milliseconds. Data from this sweep was stored using a single variable name for purposes of optimizing both the speed of acquisition and the storage requirements. This single variable comprised a long string of numbers from the three gages. Data from gage 1 (Gage 12) for example were stored in the first, fourth, seventh and $3n + 1$ th locations in this string. Similarly for gages 2 (Gage 15) and 3 (Gage 18) data were stored in the $3n + 2$ th and $3n + 3$ th locations respectively ($n = 0, 1, 2, 3...$). Sampling continued until the 64kB memory buffer was full (after approximately 19 seconds). The computer program was then designed to sort the single data string into three vectors, one for each gage location of interest.

Temperature Drift and Noise Correction: One of the problems encountered was the drift of the Wheatstone bridge output as the temperature of the deck-plate changed throughout the day. On occasion the computer also picked up irregular (of no measurable frequency and significantly different from the strain gage signals) electrical noise from an unknown source which caused

erroneous maximum counts. This noise was observed on the oscilloscope and the PC-based system during the early stages of the program development. In order to remedy these two problems, a subroutine was added to the program which smoothed each of the three waveforms using a complete quadratic polynomial. The number of data points to be used for the smoothing operations was among the input data read at the start of the program. Initial trials showed that a ten-point smoothing worked best. The curve-fitting technique served well to both minimize the effects from the stray noise, as well as to correct for the drift due to temperature variations. Corrected data were written over the raw data acquired earlier.

Data Processing for Identifying Maximums and Associated Strain Levels: The maximum strain levels were obtained using a simple algorithm that initially set the maximum equal to a floor limit (the lowest strain level of interest). Each data point was then compared to this temporary maximum. If the data point was greater than the temporary maximum, it was designated as the new temporary maximum. However, it was not recorded as a maximum until it was determined to be the absolute maximum of that peak. This was determined by checking to ensure that no subsequent data points exceeded this value until the waveform dipped below the prescribed floor level. At this stage the temporary maximum was declared an absolute maximum, and an associated counter was incremented to identify the strain level of this maximum. Once this procedure was completed for all the three strain gage signals obtained in the current sweep, the program was designed to check if the current counters had been stored in a disk file within the last hour. The program was designed to write the ten counter values for each of the three gages once every hour. A new data file name was generated automatically after each such writing operation. The strain levels used for the ten counters were user adjustable. During the first week of testing, the strain levels for the counters were refined to get the best resolution for the highest maximum events. Final strain levels used for the ten counters are presented in Table 2.1.

Counter number one is a strain level associated with a regular two-axle truck or an empty tractor-trailer truck. The other counters are associated with loaded tractor-trailer trucks. Passenger vehicles were normally not observed to cause strain levels to cross the floor threshold.

Table 2.1 Strain levels associated with the ten counters

Counter Number	Strain Level ($\mu\varepsilon$)	Counter Number	Strain Level ($\mu\varepsilon$)
1	20-50	6	200-225
2	50-100	7	225-237
3	100-150	8	237-250
4	150-175	9	250-262
5	175-200	10	262-999

Computations for smoothing and data processing operations took a long time compared to the time required to acquire enough data to fill the memory buffer. The program was actually sweeping the strain gage signals for about 12 percent of the time. Before the smoothing routine was added to the program, the computer was sweeping the gages approximately 80 percent of the time. Since the noise and drift corrections were essential to obtaining reliable strain data, the smoothing routine was retained. It should be pointed out that, since the strains were monitored over a six-week period, the 12 percent window of data acquisition still provides reliable and statistically significant data.

2.4 TEST SECTIONS ON THE BRIDGE AND THEIR MONITORING

Test sections of the various wearing surface materials being evaluated for use on the Poplar Street Bridge were placed at four different times during the course of this investigation.

The first placement was undertaken in June 1989. Five different materials were placed during this application. These included: (i) Styrelf 14-60 asphalt concrete, (ii) Epoxy asphalt concrete, (iii) Transpo T-48 epoxy concrete, (iv) Flexolith epoxy concrete, and (v) Degussa 330 methyl methacrylate concrete. The location of each test section, the steel plate treatment (paint system), and wearing surface thickness details are included in Fig. 2.12. Further details on

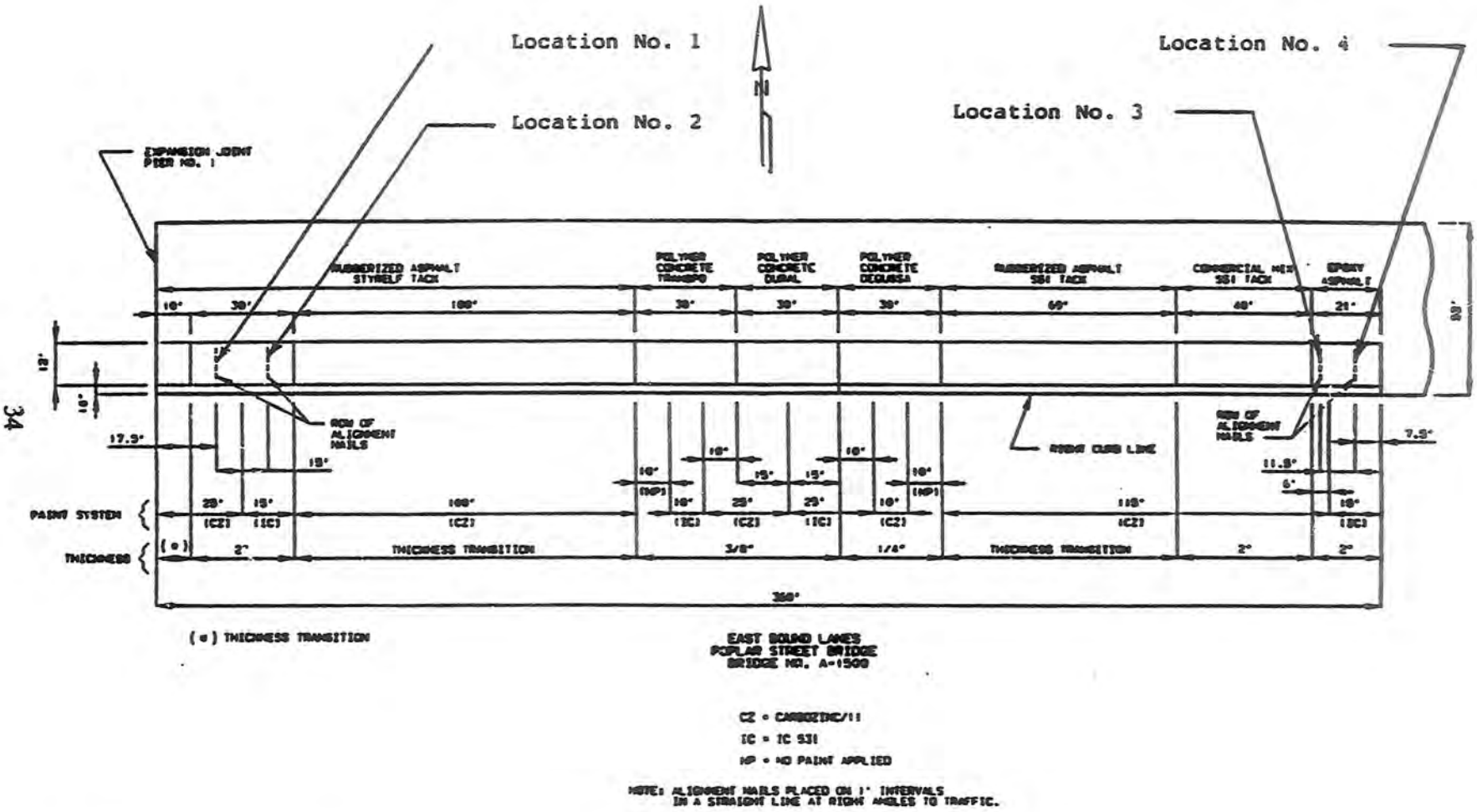
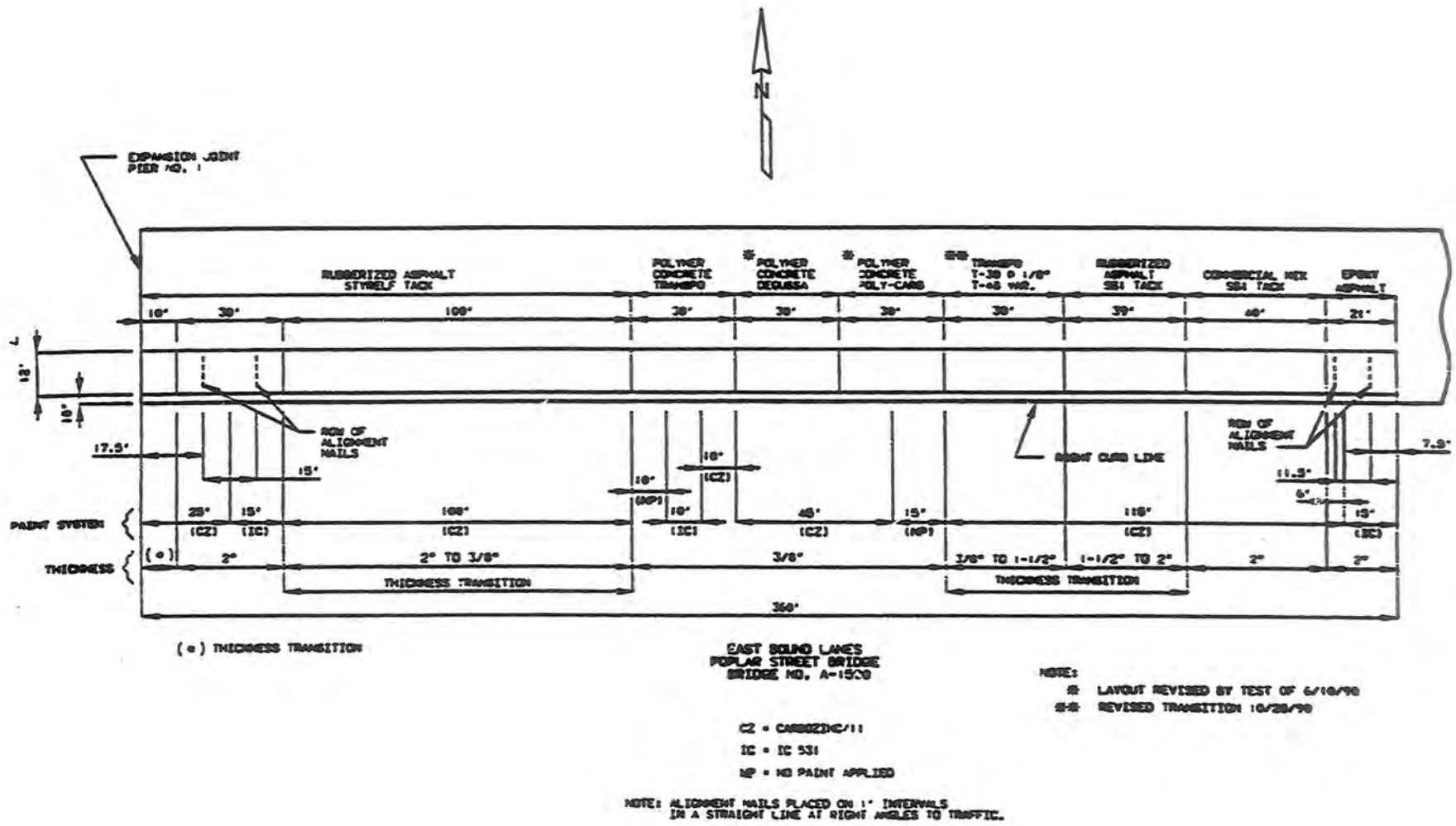


Fig. 2.12 Layout of the test sections placed in June 1989 [21] giving details of the paint system and wearing surface thicknesses used.



(a) THICKNESS TRANSITION

EAST BOUND LANES
POPLAR STREET BRIDGE
BRIDGE NO. A-1500

NOTE:
* LAYOUT REVISED BY TEST OF 6/10/90
** REVISED TRANSITION 10/28/90

CZ = CARBOZINC/11
IC = IC 531
NP = NO PAINT APPLIED

NOTE: ALIGNMENT NAILS PLACED ON 1' INTERVALS
IN A STRAIGHT LINE AT RIGHT ANGLES TO TRAFFIC.

Fig. 2.13 Layout of the test sections updated to show additional placements of June 1990 [16].

removal of the old wearing surface and placement procedures are available in the interim report issued by the MHTD's Division of Materials and Research [21].

The second placement was undertaken in June 1990. Two failed wearing surface systems (Degussa 330 methyl methacrylate concrete and Flexolith epoxy concrete) were removed and in their places (i) a reapplication of Degussa 330 methyl methacrylate concrete and (ii) Polycarb epoxy concrete were applied [16]. The Degussa system was reapplied because the manufacturer felt that the first application was carried out under less than ideal temperature conditions. Fig. 2.13 provides details of these new test sections and their particulars.

The third placement in October 1990 involved placement of a test section of Transpo T-48 epoxy over a 1/8 in. thick layer of Transpo T-30 methyl methacrylate for thickness transition from 3/8 in. to 1-1/2 in [16].

The fourth placement in May 1991 involved placement of Transpo T-48 epoxy system to test the in-service performance of construction joints in the wearing surface. It was during this time that the joint specimens for laboratory flexural fatigue tests were also fabricated.

Performances of these test sections are discussed in detail in several reports issued by the MHTD's Division of Materials and Research [16, 17, 21, 22]. A summary of these results is also included later in Chapter 3 for completeness of the discussions related to the choice of a wearing surface for the Poplar Street Bridge.

2.5 OTHER TESTS

2.5.1 Pull-out tests: Pull-out tests were performed on the flexural fatigue test specimens in accordance with ACI 503R (Appendix A) [1]. This test was used to determine the tensile bond strength between the polymer concrete and the steel plate. Similar tests were also conducted by MHTD on the Poplar Street bridge to determine the tensile bond strength between the wearing surface and the steel deck plate in the test sections. The testing device consisted of a load frame (Fig. 2.14) made of three 1-in. all thread rods, two 1/2-in. plates and a square bar that was threaded on one end. The square bar was placed through the two plates and a large nut was placed

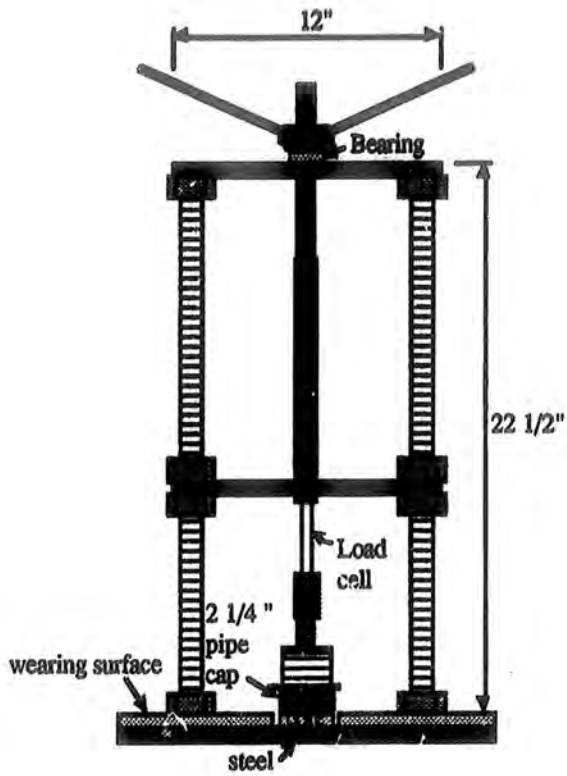


Fig. 2.14 Schematic of the core pull-out device

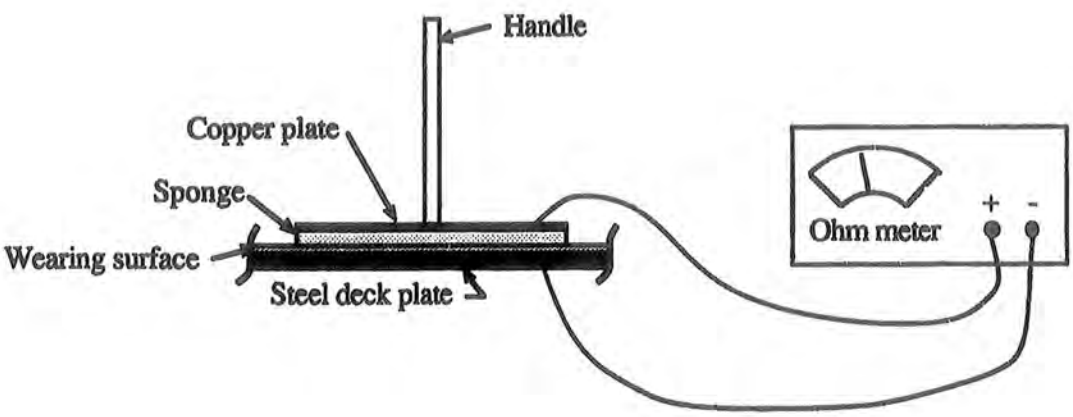


Fig. 2.15 Schematic of the test set-up for the resistivity tests.

on the threaded end. When the nut was turned the bar moved upward thus applying a tensile force on a pipe cap. The pipe cap was glued to the cored pull-out specimen. A 1/2-in. diameter aluminum strain gage-based load cell was connected between the pipe cap and the square bar. The signal from this load cell was recorded using a digital oscilloscope. Fig. 2.14 presents details of the pull-out device. Pull-out tests were conducted on some of the laboratory specimens after they had been subjected to flexural fatigue. A limited number of previously unloaded specimens were also tested for pull-out strength.

2.5.2 Resistivity tests: Resistivity tests on the laboratory flexural fatigue specimens were performed in accordance with ASTM D3633-88 [3]. These tests determined the severity of wearing surface cracking in the flexural fatigue specimens (Fig. 2.15). Similar tests were performed by the MHTD on the test sections on the Poplar Street bridge to determine the extent of cracking in the wearing surface under service conditions.

On laboratory specimens the test consisted of placing the specimen mold back on the specimen with silicone rubber caulking around the edges to prevent water from reaching the steel plate. Once the caulk had set up, a soap and water mixture was placed on the surface of the specimen. Soap was added to minimize the surface tension, thus allowing the water to penetrate fine cracks which might be present in the specimen. A thin copper plate 3-in. x 6-in. was attached to a sponge and allowed to soak in the soap water mixture. This "probe" was attached to one lead of an ohm meter, the other lead was attached to the steel plate of the specimen. When a reading was desired, the probe was taken from the soap water and placed on top of the specimen and the ohm meter was read immediately. Readings were taken at 1 minute, 5 minutes, 10 minutes, 15 minutes, 30 minutes, 1 hour, 1.5 hours and 2 hours after the specimen had been saturated with the soap water mixture. As time passed, the measured resistance usually decreased. Results from this test provided a qualitative estimate of the severity of cracking in the wearing surface material. Low resistance (less than 10,000 ohms) indicated the specimen had a large crack through the thickness of the wearing surface. A minimum resistance of 750,000 ohms was specified as the lowest

acceptable value for this resistance. Tests were conducted on each of the four types of wearing surfaces tested in the flexural fatigue tests. Two specimens of each wearing surface material were tested. One of these was subjected to flexural fatigue loading prior to conducting the resistivity test. The other was not subjected to any loading prior to the resistivity test.

CHAPTER 3 - TEST RESULTS AND DISCUSSIONS

3.1 FLEXURAL FATIGUE TESTS

In the absence of direct visual access to the specimens under test, cracking and/or delamination was monitored during the flexural fatigue test by looking for abrupt changes in the stiffnesses of the specimens. This detection of a change in specimen stiffness was complicated by two facts.

First, the elastic modulus of the wearing surface was significantly lower than that of the steel base-plate. Also, for the polymer concretes specimens, the thickness of the wearing surface was smaller than that of the steel base-plate. As a result the contribution of the wearing surface to the stiffness of the composite specimen was small for these materials. Any change in the stiffness resulting from cracking or delamination of the specimen was small. Detecting small changes in the stiffness is a challenging task that requires accurate measurement of loads and deflections. For the asphalt materials which had wearing surface thicknesses of 2-in., the contribution of the wearing surface to the stiffness of the composite specimen was more significant than in the case of the polymer concretes.

Second, the elastic modulus of polymer concrete materials is very sensitive to test temperature. While this did not pose any problem for the flexural fatigue tests conducted at constant temperature (0°F), it was necessary to normalize the specimen stiffness for temperature each time the stiffness was measured during the temperature cycled flexural tests. This normalization was necessary before stiffness changes due to tensile cracking in the wearing surface or delamination of the wearing surface from the steel plate could be detected.

In a later component of the study dealing with fatigue response of simulated construction joints in the wearing surface, a more elaborate system of crack detection which comprised fine wires glued on to the top of the wearing surface was used. This system worked very well in the timely detection of cracks.

3.1.1 Temperature cycled flexural fatigue tests: A total of nine specimens was tested in the first group of temperature cycled flexural fatigue tests. An upper limit load of 1,450 lbs. was used for these tests. As stated earlier, this value was chosen based on the load levels used in the fatigue tests done for the Luling bridge using comparable specimen geometry and size. Results from these tests are summarized in Table 3.1.

Table 3.1
Results from the temperature cycled flexural fatigue tests (Upper limit load : 1,450 lbs)

Spec. No.	Wearing Surface	Coating	Mode of Failure	Failure at (# of cycles)
1	Epoxy asphalt	IC531	Bond failure (tack coat-zinc primer)	1×10^6*
2	Epoxy asphalt	IC531	No failure	over 4.75×10^6
1	Flexolith epoxy	Bare steel	Tensile cracking	1×10^6*
2	Flexolith epoxy	Bare steel	Tensile cracking	1×10^6*
1	Rubberized asphalt	Carbozinc	Tensile cracking	1×10^6*
2	Rubberized asphalt	Carbozinc	Tensile cracking	1×10^6*
3	Rubberized asphalt	Carbozinc	Tensile cracking	1×10^6*
1	Transpo T-48 epoxy	Bare steel	Tensile cracking	1×10^6*
2	Transpo T-48 epoxy	Bare steel	Tensile cracking	1×10^6*

*First exposure to 0°F

Asphalt-based wearing surfaces: Flexural fatigue tests were used to investigate the potential for cracking and delamination of the asphalt-based wearing surfaces. The potential for rutting and shoving which are perhaps more relevant to the failure of such wearing surfaces was studied by observing the test sections on the bridge.

Two epoxy asphalt and three rubberized asphalt specimens were tested in flexural fatigue while simultaneously being subjected to varying temperatures. One of the epoxy asphalt specimens exhibited bond failure. This was detected by the sudden increase in one of the end-slip measurements. Adhesive Engineering Co. which fabricated the specimen indicated prior to testing

that the zinc primer on one of the two specimens delivered to them had been applied thicker than recommended. Sawing the wearing surface to investigate the plane of delamination indicated that the failure was for the most part between the water-based zinc primer and the epoxy asphalt tack coat. A few spots where the zinc primer debonded from the steel plate were also observed. This failure occurred when the specimen was first exposed to 0°F. The second epoxy asphalt specimen did not exhibit cracking or delamination based on the stiffness or end-slip measurements. After the test was stopped (> 4.75 x 10^6 cycles) the specimen was examined visually for cracks when subjected to approximately 1,000 lb. static load. No evidence of cracks or delaminations were observed. It has been observed in previous fatigue tests that asphalt concrete specimens with low air-void contents (percentage of total volume) performed better in fatigue than those with high air void content [15]. The epoxy asphalt specimens numbered 1 and 2 (Table 3.1) had air void contents of 3.8% and 2.5% respectively. Density measurements indicated that there were significant differences between the epoxy asphalt material on the test specimens furnished by Adhesive Engineering Co. and the material that was placed by Adhesive Engineering Co. on the bridge test section (air void content 5% - 7%). Before additional fatigue tests could be made at comparable air void contents, the field test sections had failed due to rutting and shoving [21] and all further laboratory testing of epoxy asphalt was suspended.

The air void contents of the rubberized asphalt wearing surfaces on the three fatigue specimens were in the 11% - 13% range and were representative of the material placed on the bridge test section (11.3%). These values were well above typical air void contents in conventional asphalt materials (5% - 7%) which is the likely reason these specimens failed by tensile cracking when first exposed to 0°F temperature and fatigue loading. Again, before additional tests at lower air void content could be undertaken, the bridge test sections had failed due to rutting and shoving.

Polymer concrete: Four epoxy concrete specimens, two of Transpo T-48 epoxy and two of Flexolith epoxy were tested in the first group of fatigue tests conducted at an upper limit load of 1,450 lbs. (Table 3.1). Since stiffness changes resulted not only from wearing surface failure

(cracking and/or delamination) but also from temperature changes during the fatigue tests, it was necessary to isolate the two effects. This was conveniently done by plotting stiffness as a function of temperature. Fig. 3.1 shows the temperature dependence of the stiffness of the composite specimens (Transpo T-48 epoxy - Steel composite) for temperatures below 100°F. Based on data from these tests, it was observed that for test temperatures higher than 100°F the stiffnesses of the composite specimens were essentially the same as the stiffnesses of the bare steel plates (approximately 35,000-39,000 lb./in, depending upon the actual dimensions of the steel base-plate). For each fatigue specimen, a linear line similar to that shown in Fig. 3.1 for a Transpo T-48 epoxy specimen was used to determine the uncracked stiffness of the specimen at any temperature. Stiffness measured during the fatigue test were then divided by this uncracked stiffness at the test temperature to compute a relative stiffness. At temperatures below 100°F the relative stiffness of a cracked specimen would essentially be less than 1.0. At temperatures above 100°F this relative stiffness would essentially be 1.0 even for a cracked specimen, because above that temperature the wearing surface does not make a significant contribution to the specimen stiffness.

Fig. 3.2 shows a plot of the temperature corrected relative stiffness versus the number of cycles of fatigue loading for a Transpo T-48 epoxy specimen subjected to an upper limit load of 1,450 lbs. Also shown in the figure is a plot of the variation of the test chamber temperature in relation to the fatigue loading cycles. It can be observed that cracking can be first detected about the time the specimen is subjected simultaneously to the fatigue loading and the coldest temperature (0°F, Fig. 3.2).

Based on test results like that presented in Fig. 3.2 it was determined that the upper limit load used for the initial series of the temperature cycled fatigue tests (1,450 lbs.) was higher than the maximum levels of stress sustainable by the wearing surface at the minimum temperature of 0°F. This upper limit load was based on preliminary computations of the stresses expected in the wearing surface and on results reported from deck strain measurements on the Luling bridge [15] which had a deck geometry comparable to that of the Poplar Street bridge. Subsequently,

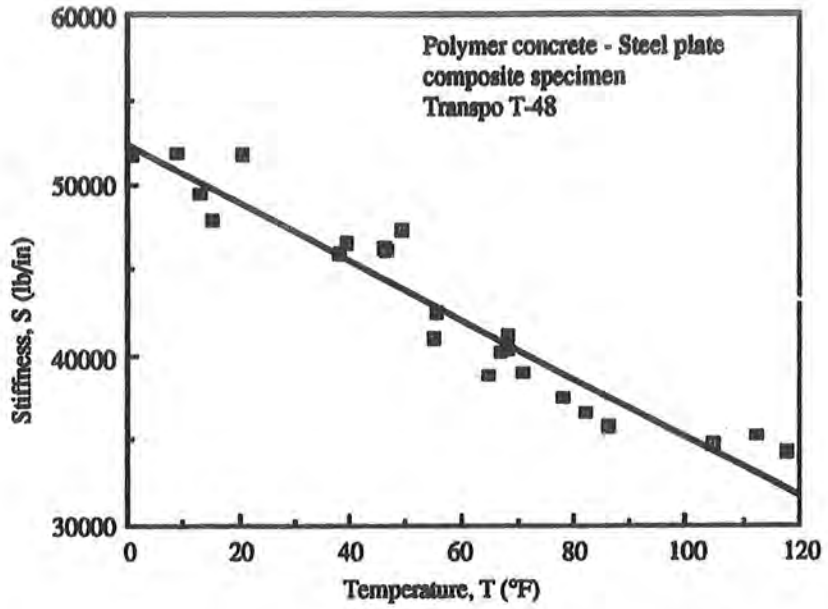


Fig. 3.1 Typical variation of specimen stiffness with temperature (polymer concrete)

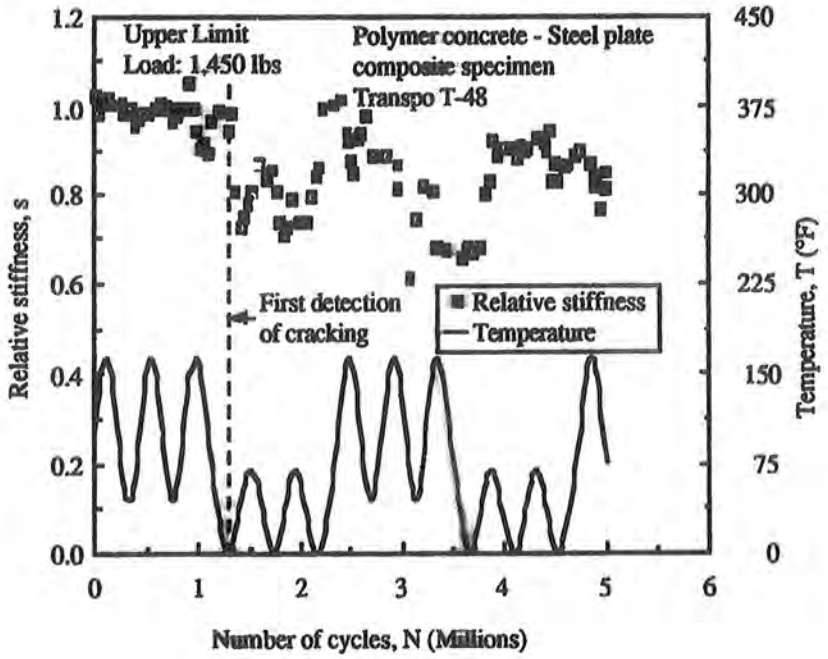


Fig. 3.2 Temperature corrected relative stiffness versus the number of fatigue cycles (upper limit load : 1,450 lbs.). Thermal loading applied simultaneously with the 5 Hz fatigue loading is also shown.

supported by results from the cold temperature flexural fatigue tests, the upper limit load was adjusted to 1,150 lbs.

The second group of temperature cycled flexural fatigue tests was conducted on six epoxy concrete and two methyl methacrylate concrete specimens using an upper limit load of 1,150 lbs. The results from these tests are summarized in Table 3.2. Even at this reduced upper limit load, the methyl methacrylate (Degussa 330 MMA) concrete and one epoxy concrete (Polycarb) (two specimens of each) failed when first exposed to 0°F. Two other epoxy concrete materials did not fail until approximately 2 x 10^6 cycles of fatigue loading had been applied. This represented approximately 750,000 fatigue cycles of exposure to winter temperatures. Of these cycles over 100,000 cycles were applied in conjunction with temperatures between 5°F and 0°F.

Table 3.2
Results from the temperature cycled flexural fatigue tests (Upper limit load: 1,150 lbs)

Spec. No.	Wearing Surface	Coating	Mode of Failure	Failure at (# of cycles)
1	Degussa-330 MMA	Carbozinc	Tensile cracking	1 x 10^6*
2	Degussa-330 MMA	Carbozinc	Bond failure (tack coat-zinc primer)	1 x 10^6*
1	Flexolith epoxy	Bare steel	Tensile cracking	~2 x 10^6**
2	Flexolith epoxy	Bare steel	Tensile cracking	~2 x 10^6**
1	Polycarb epoxy	Carbozinc	Tensile cracking	1 x 10^6*
2	Polycarb epoxy	Carbozinc	Tensile cracking	1 x 10^6*
1	Transpo T-48 epoxy	IC531	Tensile cracking	~2 x 10^6**
2	Transpo T-48 epoxy	Bare steel	Tensile cracking	~2 x 10^6**

*First exposure to 0°F
**Due to temperature control instrumentation malfunction

Fig. 3.3 shows a plot of the temperature corrected relative stiffness versus the number of cycles of fatigue loading for the Transpo T-48 epoxy - steel plate composite specimen subjected to an upper limit load of 1,150 lbs. Even at 2 x 10^6 fatigue cycles, the failure of these wearing surfaces was precipitated by an equipment malfunction. At this stage into the test, the valve on the

liquid nitrogen line froze in the 'open' position. The temperature in the test chamber dropped to -85°F before the equipment malfunction was detected. This resulted in the tensile cracking failure of the four epoxy concrete specimens (Transpo T-48 epoxy and Flexolith epoxy).

The 100,000 cycles of fatigue life at temperatures between 0°F and 5°F at the upper limit load of 1,150 lbs. used in further discussions in this report should hence be viewed as a conservative estimate of the fatigue life of these materials at temperatures in the 0°F -5°F range.

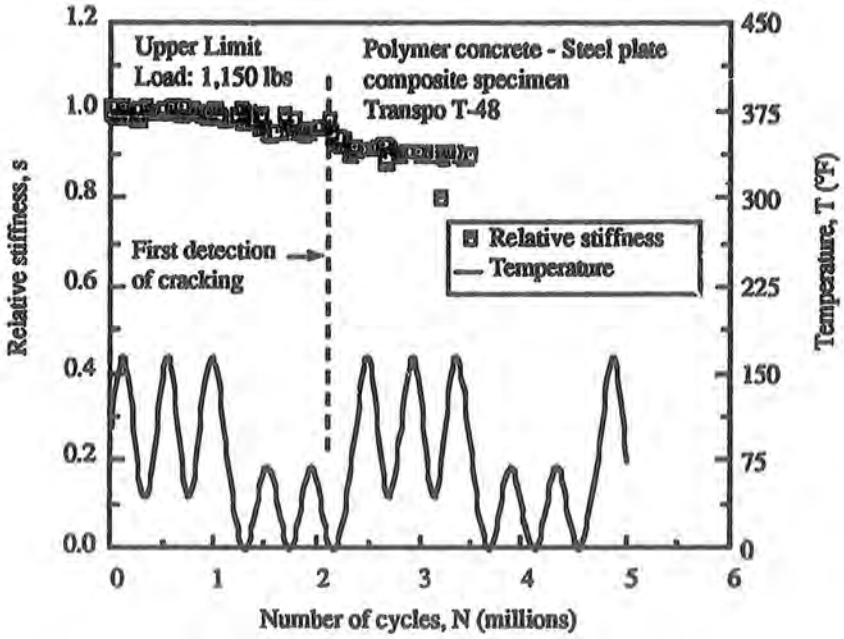


Fig. 3.3 Temperature corrected relative stiffness versus the number of fatigue cycles (upper limit load : 1,150 lbs.). Thermal loading applied simultaneously with the 5 Hz fatigue loading is also shown.

3.1.2 Cold temperature flexural fatigue tests: After the results from the first group of temperature cycled flexural fatigue tests were analyzed, it was observed that all the polymer concrete materials tested cracked at the first exposure to 0°F. These materials could not sustain the simultaneous combination of the strains in the wearing surface due to an upper limit load of 1,450 lbs. and the brittleness resulting from the 0°F environment. Since the cold temperatures used are

typical for a cold winter night on the deck, it was necessary to determine the maximum fatigue stress level sustainable by the polymer concrete materials at these temperatures. Specimens were tested in flexural fatigue at 5 Hz while being subjected to a constant temperature environment of 0°F. The initial lower and upper limit loads used were approximately 150 lbs. and 300 lbs., respectively. The upper limit load was incremented by approximately 150 lbs. after every 70,000 fatigue cycles until the specimens failed. Although these increments were measured exactly for each specimen, they differed somewhat depending upon the individual specimen stiffness because four specimens were tested simultaneously under displacement controlled conditions. Unlike the temperature cycled flexural fatigue tests where the stiffness was influenced both by the test chamber temperature and by specimen failure, in the cold fatigue tests specimen stiffness was influenced only by specimen failure. No temperature-dependent stiffness correction was needed to detect specimen failure from the stiffness data recorded. Fig. 3.4 shows a plot of the relative stiffness (ratio of the current stiffness to the stiffness of the uncracked specimen - both measured at 0°F) versus the number of fatigue cycles. Fig. 3.4 clearly demonstrates that the critical combination of the upper limit load and a temperature of 0°F occurs when the upper limit load is between 1,200 - 1,350 lbs. which corroborates well with earlier observations from the flexural fatigue tests.

Table 3.3 presents a summary of all the results from the cold temperature flexural fatigue tests conducted during this investigation. The methacrylate concrete (Degussa 330 MMA) and one of the epoxy (Polycarb) concrete materials failed at relative low levels of upper limit load (900 lbs. and 650 lbs. respectively). The other two epoxy concrete materials performed well. Transpo T-48 epoxy performed marginally better than the Flexolith epoxy specimens. However, given the fact that only two specimens were tested for each of these two materials, the difference in performance is not considered to be significant. What is significant however is the fact that both of these materials failed due to a system of fine distributed cracking as opposed to a single large crack as in the case of Polycarb epoxy. This is important because a single large crack is more likely to permit water/chloride penetration and to precipitate local delamination.

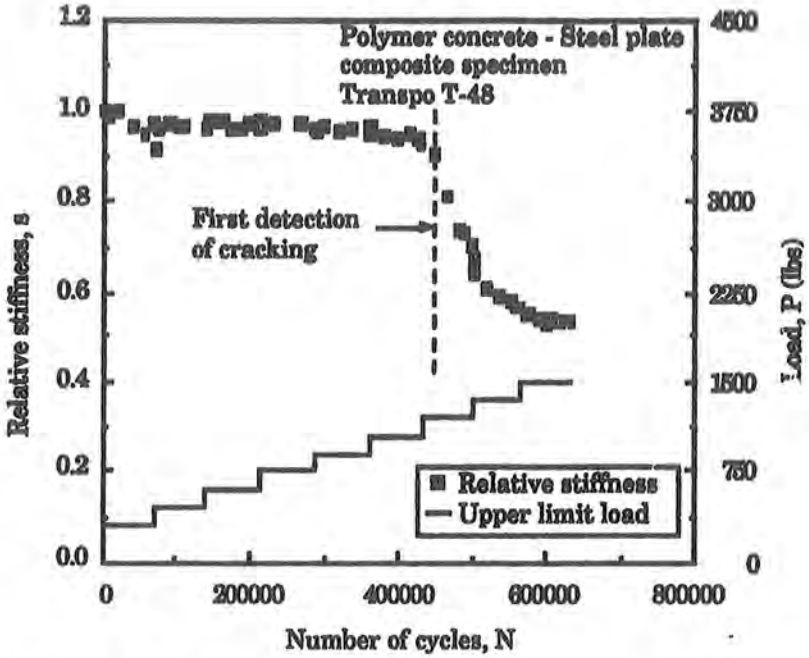


Fig. 3.4 Relative stiffness versus the number of fatigue cycles in a cold temperature flexural fatigue test. Progressively increasing upper limit load is also shown in the figure.

Table 3.3
Results from the cold temperature flexural fatigue tests

Spec. No.	Wearing Surface	Cracking Load (lbs)	Cracking stress* in the wearing surface (psi)	Remarks
1	Degussa-330 MMA**	900	1,588	Multiple fine cracks
2	Degussa-330 MMA**	900	1,585	Multiple fine cracks
1	Flexolith epoxy***	1,200	2,000	Multiple fine cracks
2	Flexolith epoxy***	1,050	1,704	Multiple fine cracks
1	Polycarb epoxy**	650	1,147	Bond failure (primer to steel)
2	Polycarb epoxy**	600	1,040	Large single crack
1	Transpo T-48 epoxy***	1,200	2,027	Multiple fine cracks
2	Transpo T-48 epoxy***	1,200	2,105	Multiple fine cracks

*Based on measured wearing surface thickness for each specimen and a modular ratio n = 14.5
**Primer coating - Carbozinc
***Primer coating - IC531

3.1.3 Flexural fatigue tests on joint specimens: Results from the temperature cycled flexural fatigue tests on the specimens simulating construction joints in the wearing surface are illustrated in Fig. 3.5. It shows the cycle count at which each of the two crack detection wires on each of the eight specimens failed. From these results it appears that the earliest failures in the joint specimens occurred after 1.33×10^6 cycles of fatigue loading (with the exception of specimen NC4 which was damaged during fabrication).

Of the 1.33×10^6 cycles, at least 100,000 cycles were at temperatures below 10°F. It should also be noted that the specimens labeled TC (tack coat applied on the joint) in Fig. 3.5 performed marginally superior to those labeled NC (no tack coat applied).

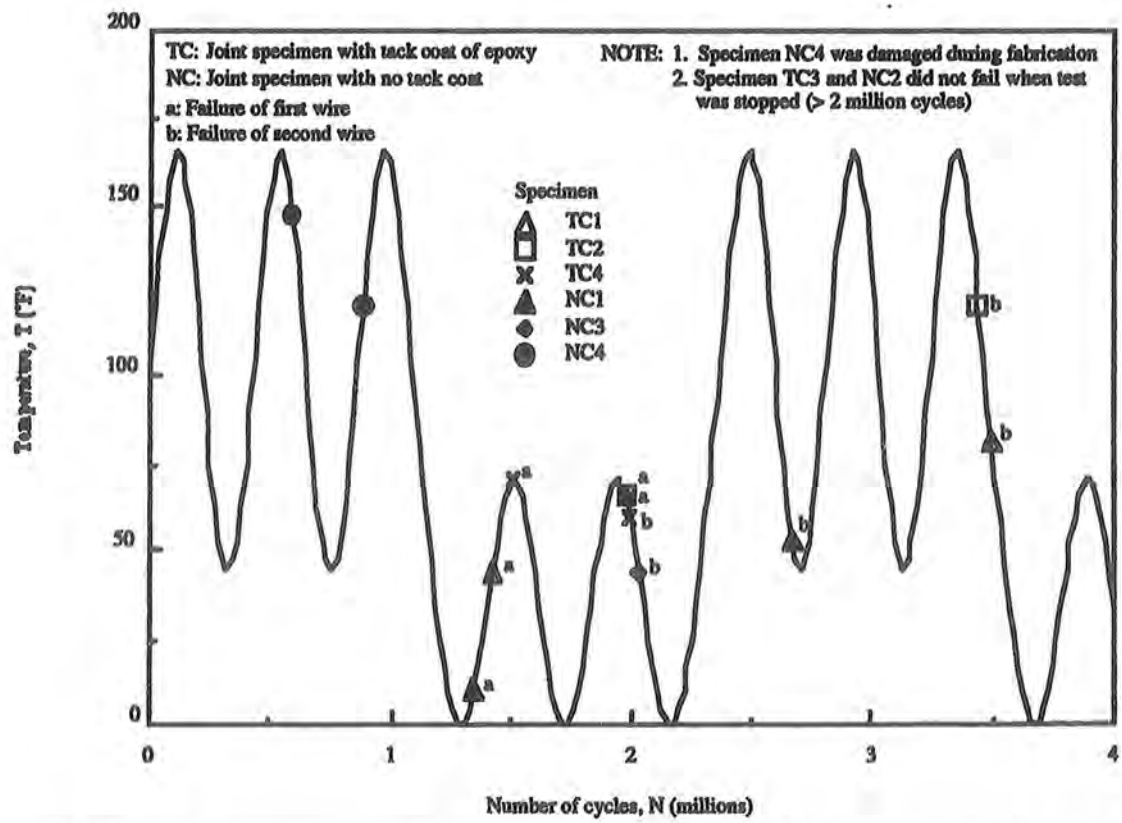


Fig. 3.5 Summary of results from the temperature cycled flexural fatigue tests on joint specimens. Symbols indicate when each crack detection wire failed on the different specimens.

Some differences may exist between the laboratory performance and field performance of construction joints in the wearing surface. While it was relatively easy to keep the joint in the laboratory specimen clean, control of dust and other impurities on the deck may not be as good, particularly since construction is to be undertaken with regular traffic in adjoining lanes. Impurities in the joint will result in poor bonding between two successive placements of the wearing surface. The steel plate in the laboratory specimen was unstressed during fabrication of the joint specimen. The deck plate on the bridge will be dynamically stressed during placement of the wearing surface. Also the joints in the laboratory specimen were not loaded directly. Vehicle tire loads will be applied directly to the construction joints on the bridge. Local unevenness at the construction joint due to thickness differences between two different placements of the wearing surface may cause additional local stresses.

3.2 DECK STRAIN MEASUREMENTS

This section presents results that were collected from the deck strain measurement program, discusses procedures used to determine the peak stress experienced by the wearing surface, and provides an explanation of how the measured peak strain relates to the laboratory flexural fatigue tests both in terms of its magnitude and frequency of occurrence.

3.2.1 Automated data collection: Over 700 data files were collected from the strain gage recording program described in Section 2.3 during a six-week period in January and February 1991. Of these files, the last 400 are believed to be reliable and statistically significant. The first 300 data files were used to refine the strain gage counter level increments and to debug and refine the program. As explained earlier in Chapter 2, each data file contained ten counters for each of the three transverse strain gages monitored. Each counter represents a range of strain levels (Table 2.1). The number in each counter represents the number of times the strain gage experienced maximum strain within the limits of the level associated with that counter during the hour in which the data file was created. Although the primary motivation for undertaking the deck strain

measurements was to establish the magnitude and frequency of the maximum tensile strain event to which the wearing surface was subjected, with little additional effort it was also possible to record hourly traffic patterns on the Poplar Street Bridge. Data files were hence saved to a disk every hour. This also provided a safeguard against power outages. If the power supply was cut-off at any time during the monitoring period, data from the last hour before the outage would be the only data lost. When the power supply was restored, the system was designed to reboot the computer and restart the data collection program. However, during the six-week data collection period, the system did not experience any power outage.

3.2.2 Strain histories and associated hourly histogram summaries: A typical signal obtained from Gage 15 of the transverse strain gages is presented in Fig. 3.6. This strain history was obtained using a digital storage oscilloscope and is similar to the strain gage signals processed by the automated data acquisition system. The computer program processed three similar strain histories simultaneously. On this particular strain history there are three global sub-events. Each sub-event corresponds to a group of axles on a tractor-trailer unit. The first sub-event is associated with the front axle of the tractor, while the second sub-event has two strain reversals as each of the two axles on the rear of the tractor pass directly over the strain gage. The third sub-event does not have the strain reversals, but there are dips in the trace as the two rear axles of the trailer pass over the strain gage. In this sub-event the trace does not drop below the floor level in the first dip, so only two maximums of a level 20-50 $\mu\varepsilon$ are recorded. After processing this particular signal, results for the counter values will show the changes listed in Table 3.4. Counter numbers and associated strain intervals were earlier presented in Table 2.1. The change in value represents the number of maximums within the interval represented by the counter, which resulted from the event shown in Fig. 3.6. The main purpose of the strain recording program was to reduce the complicated strain history of Fig. 3.6 to the six counts shown in Table 3.4. The data acquisition system processed large amounts of data and extracted usable information on-line to save disk storage space.

Depending on where the tires pass over the test section, a mirror image about the zero strain axis of Fig. 3.6 could be experienced by the strain gage. The program was written so that only maximums that caused tensile strains in the wearing surface were recorded. For Gage 12 and Gage

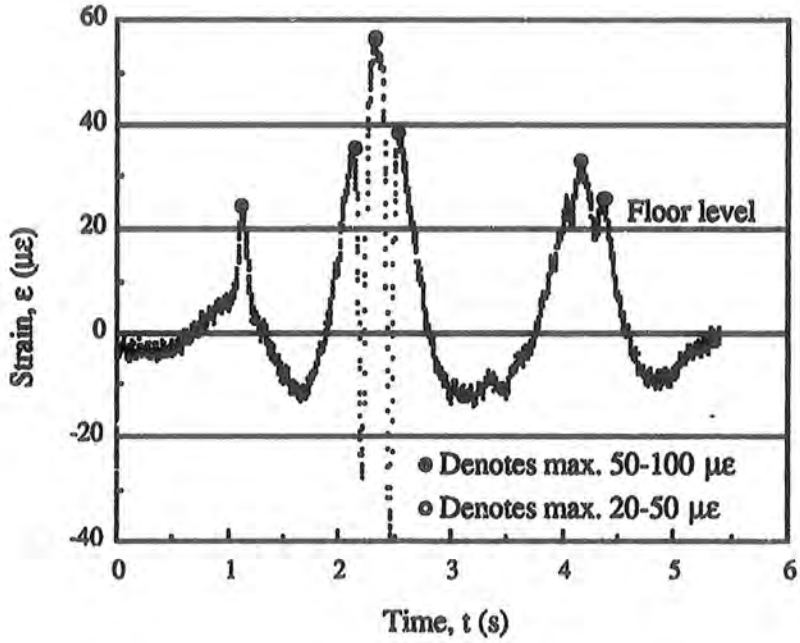


Fig. 3.6 Typical strain history recorded during a loading event (Gage 15)

Table 3.4
Changes in counter values after processing the trace shown in Fig. 3.6.

Counter Number	Change in Value	Counter Number	Change in Value
1	5	6	0
2	1	7	0
3	0	8	0
4	0	9	0
5	0	10	0

18 (both of which were midway between stiffeners) only maximum tensile strains were recorded. Tensile strain on the bottom of the steel deck midway between stiffeners was translated to tensile strain in the wearing surface above the stiffeners. Only maximum compressive strains monitored on Gage 15 (located within one inch of a stiffener) were stored. Since Gage 15 is located next to a stiffener, compressive strains on the bottom of the deck plate next to the stiffener would translate to tensile strains in the wearing surface above the stiffener.

3.2.3 Measured peak strain and associated frequency of occurrence: The peak strain measured during the monitoring period was recorded at Gage 12. The peak strain events were associated with strain counter number 8. This would place the peak strain between 237 $\mu\varepsilon$ and 250 $\mu\varepsilon$. A total of four such events were recorded during the three weeks the program was collecting reliable data. However, the program was only sweeping the strain gages about 12 percent of the time. For the remaining 88 percent of the time the program was processing the strain histories into associated histograms as described earlier. Given the randomness of the "12 percent time intervals" during the three week period, it is reasonable to assume that approximately 32 such peak strain events (strain level between 237 $\mu\varepsilon$ and 250 $\mu\varepsilon$) would have occurred during the entire three week period of reliable program operation. This corresponds to approximately 11 events in this strain range per week.

Statistical analysis of the hourly traffic patterns are not of particular interest as far as this investigation is concerned and hence are not discussed here. Information on this aspect is available in reference [5].

3.3 PERFORMANCE OF THE TEST SECTIONS ON THE BRIDGE

As discussed in Section 2.4, the wearing surfaces under consideration were applied to the test sections of the bridge during four different times. Reports [16, 17, 21, 22] detailing the results from these field performance studies were prepared by MHTD's Division of Materials and

Research responsible for monitoring these test sections. A summary of their observations is presented here for completeness of the discussions to follow.

Rutting of the rubberized asphalt test section ranged from 1/8 - 1/4-in. after the first 30 days and 1/8 - 5/16-in. after 60 days. Rutting stabilized after the first two months. Longitudinal movement was observed after two months and continued through the evaluation period. Random cracks developed in the rubberized asphalt test section after approximately 6 months.

The epoxy asphalt test section showed rutting in the early stages after placement and the depth of rutting increased from 1/16-in. to 3/8-in. during the evaluation period. Longitudinal movement was also observed. Rutting stabilized after 4-1/2 months, but longitudinal movement continued through the evaluation period. Longitudinal and transverse cracks were observed in the wheel paths. The MHTD's Division of Materials and Research [21] believes that heating and placement problems during construction of the epoxy asphalt test section may not have provided a valid test of this material. It should be noted that an Adhesive Engineering Co. placed epoxy asphalt wearing surface on the San Mateo Hayward bridge in San Fransisco requires minimal maintenance [19] even after 23 years of service.

The Degussa 330 methyl methacrylate concrete system experienced no horizontal or vertical movement. However it did exhibit some aggregate loss and resulting slick glass appearance. Pull-out tests on the field test section revealed low tensile bond strength values compared to the other polymer concrete wearing surfaces tested. The first test section of the methyl methacrylate concrete placed in June 1989 failed by delamination.

The Flexolith epoxy concrete system placed in June 1989 initially performed satisfactorily except at the longitudinal construction joint. This joint appeared to open up in the cooler temperatures. In March 1990 this test section failed due to delamination. Although not definitive, it appears that the delaminations may have been initiated at the failed longitudinal construction joint. Successful applications of the same material elsewhere [21] suggests that this test section may not have been properly constructed.

The Polycarb epoxy test section placed over Carbozinc 11 failed by delamination. Another Polycarb epoxy test section placed over bare steel exhibited unacceptable resistivity values after 16 months of service. The thickness transition section where Transpo T-48 epoxy was placed over a base layer of Transpo T-30 methyl methacrylate consistently failed in pull-out tests at the MMA-epoxy interface. This type of a transition layer was not recommended for use on the Poplar Street bridge [16].

The Transpo T-48 epoxy concrete test section performed well. According to the evaluation of the MHTD's Division of Material and Research, the T-48 placed on bare steel and that placed on Carbozinc 11 were considered acceptably wearing surface materials [17]. The construction joints in the Transpo T-48 epoxy concrete field test section also performed well.

3.4 OTHER RESULTS

3.4.1 Pull-out tests: Numerous pull-out tests were performed on two of the polymer concrete surfaces (Transpo T-48 epoxy concrete and Flexolith epoxy concrete) using several different kinds of materials (methyl methacrylates and epoxies) to glue the pipe cap from the pull-out device (Fig. 2.15) to the core. In all of these tests the tensile failure occurred at the glue line between the pipe cap and the core. In all cases the stress at the time of failure was in excess of 500 psi. In some instances the stresses were as high as 800 psi. Some of the cores were on specimens that had failed by tensile cracking in the flexural fatigue tests. These values are higher than those observed in the pull-out tests conducted on the field test sections (for example 223-445 psi for Transpo T-48 epoxy concrete on bare steel). This may be attributed to the wear due to traffic as well as weathering on the test section. The relationship of results from this test to the actual performance of the wearing surface is not entirely clear for surfaces that have not failed by delamination. As a result, pull-out testing on the additional materials was not conducted.

3.4.2 Resistivity tests: Results of the resistivity tests performed on the laboratory flexural fatigue specimens in accordance with ASTM D3633-88 [3] on the four types of polymer concrete

wearing surfaces are presented in Table 3.5. These tests were conducted both on specimens that had already failed in flexural fatigue and those which were as yet to be subjected to any loading. All uncracked wearing surfaces exhibited resistance values in excess of 15 MΩ 1-1/2 hours after saturating the surfaces with the soap-water mixture. In the cracked category, Transpo T-48 epoxy concrete exhibited the highest resistivity - 700,000 Ω after 1 minute dropping to 75,000 Ω after approximately 1-1/2 hours. Wearing surface systems like Polycarb epoxy concrete that had single large crack exhibited low resistivity values compared to those that had a fine distributed system of cracks (Transpo T-48 epoxy concrete).

Table 3.5
Results from the resistivity tests on uncracked and cracked flexural fatigue test specimens

Specimen Type	Condition	Resistivity after 1 minute	Resistivity after 1 hour	Resistivity after 1.5 hours
Degussa 330 MMA	Cracked	1,000 ohms	1,000 ohms	800 ohms
	Uncracked	∞	∞	∞
Dural Flexolith	Cracked	3,000 ohms	2,500 ohms	2,200 ohms
	Uncracked	∞	15,000,000 ohms	15,000,000 ohms
Polycarb Epoxy	Cracked	600 ohms	200 ohms	200 ohms
	Uncracked	∞	20,000,000 ohms	17,500,000 ohms
Transpo T-48 Epoxy	Cracked	700,000 ohms	200,000 ohms	75,000 ohms
	Uncracked	∞	20,000,000 ohms	15,000,000 ohms

It is relevant here to provide some comparison of resistivity values with similar overlays placed on concrete. In tests conducted by MHTD,* a 1/4 - 3/8 in. thick virgin polymer concrete overlay (Transpo Industries Inc.'s T-30) on 3 in. thick reinforced concrete specimens with cover thickness of 1-1/2 in. exhibited resistivity values of 500 MΩ. Resistivity values dropped to 325,00 Ω after the specimens were subjected to 50 cycles of freeze-thaw testing using salt water ponding.

* Girard, R., Personal communication, November 1993.

3.5 STRESS COMPUTATIONS FROM DECK STRAIN MEASUREMENTS AND THEIR RELATION TO THE FLEXURAL FATIGUE TESTS

The reason for measuring the peak strain experienced by the bridge deck was to ascertain whether the wearing surface materials in the laboratory flexural fatigue tests were subjected to realistic levels of upper limit stress. By comparing the worst case conditions experienced in service with the conditions causing the wearing surface materials to fail (crack or delaminate from the steel plate), it is possible to get useful information on the potential field performance of the wearing surface.

The second round of flexural fatigue tests was conducted at an upper limit load of 1,150 lbs. The tensile stress in the wearing surface at this load level can be computed based on the modular ratio, n (ratio of the elastic modulus of the steel plate to that of the wearing surface), the thicknesses of steel plate, the thickness of the wearing surface, and the loading configuration used. Modular ratios were calculated using the apparent stiffness of the composite specimen during the flexural fatigue tests. For example, an average value of the modular ratio, n, was found to be 14.5 when the polymer concrete material was cold (0°F - 35°F) and 29 when the material was hot (100°F - 165°F). Using a wearing surface thickness of 3/8-in., steel plate thickness of 9/16-in., specimen width of 4-in., and a span of 13-in., the tensile stress values associated with the upper limit load of 1,150 lbs. are 1,991 and 1,171 psi for modular ratios of 14.5 and 29 respectively. Clearly the cold temperature produces higher tensile stress in the wearing surface for the same external loading and hence further discussions are based on a modular ratio of 14.5.

Strain measurements were made on the bridge to provide an estimate of the maximum tensile stresses in the wearing surface. Since the maximum tensile stress in the wearing surface over the stiffners could not be measured directly, stress analyses were performed to determine this value. An approach similar to the one recommended by Wolchuck [20], where the deck plate acts in local flexure between the ribs, was used. Stresses in the deck plate were calculated by assuming it to act like a continuous plate strip supported by ribs that are assumed to be rigid and non yielding supports.

Using the measured peak strain of 250 $\mu\varepsilon$ ([5] and Section 3.2.3) and the effective width (distance along the bridge over which the load from a single wheel is distributed) as 8.5-in. [13], the tensile stress in the wearing surface was computed. Analyses of four different idealized loading configurations determined that the largest ratio of the moment over the stiffener to the moment at midspan was approximately -1.8 [13]. For n = 14.5 and using an effective width of 8.5-in., the section modulus for stress in the bottom fiber is 0.5793 in^3. The moment at midspan (between stiffeners) is then given by the product of the measured strain, elastic modulus of steel, and the bottom section modulus of the composite section and equals:

$$= (250 \times 10^{-6} \text{ in./in.})\ (29 \times 10^{6} \text{ psi})\ (0.5793 \text{ in}^3) = 4{,}200 \text{ lb in.}$$

The moment over the stiffener then is 4,200 x (-1.8) = -7,560 lb in. The tensile stress in the wearing surface using a top section modulus for the composite section of 0.2750 $in.^3$, and a modular ratio of 14.5 is 1,895 psi. Performing the same calculations with a modular ratio of 29 yields a wearing surface stress of 995 psi.

Comparing the maximum stress in the wearing surface computed from the measured peak strain with that applied in the second group of flexural fatigue tests suggests that for a modular ratio of 14.5 the overload factor used in the flexural tests is 1.05. For a modular ratio of 29 the overload factor is 1.18.

By itself, this small overload factor of 1.05 does not offer as comfortable a margin of safety as would be desirable. However, consideration of the analysis which produced this overload factor gives reason for greater optimism. In computing the stress in the wearing surface over the stiffeners from field strain measurements, a value of -1.8 was used as the ratio of the moment over the stiffener to the moment at midspan where the gages were located. This value is the worst case scenario of four possible loading conditions which are likely to occur under normal traffic conditions, but it was used for conversion of all events. The events themselves were measured to be between 237 $\mu\varepsilon$ and 250 $\mu\varepsilon$, but the maximum value of 250 $\mu\varepsilon$ was assumed for all events. Thus, the maximum stresses that actually occur may be somewhat lower than those calculated. Even with the worst case assumption, the maximum stress event was estimated to occur only about

11 times per week. A very conservative assumption that the temperature of the deck is below 5°F for five weeks per year leads to the conclusion that the critical combination of stress and temperature occurs only 55 times per year. The laboratory tests indicated that fatigue life under the 5% overload and temperatures below 5°F is at least 100,000 cycles. The fact that test sections on the Poplar Street Bridge survived two winters of normal traffic without any signs of significant deterioration or cracking also adds confidence that the selected material will perform satisfactorily. Other factors such as stresses from differential thermal expansion between the deck and the wearing surface and the effects of large numbers of very low level stress events could not be evaluated within the scope of this investigation.

CHAPTER 4 - CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

Recommendation of a wearing surface for the Poplar Street bridge is very difficult because service conditions seem to have become so severe that the customary asphalt concrete wearing surface will no longer perform satisfactorily. The first surface of asphalt concrete which was applied in 1967 was quite satisfactory, with a life of 16 years. The life of a replacement surface which was identical was lees than 20 percent of the first surface. The life of a slightly different third asphalt concrete wearing surface was only slightly longer. Rutting and shoving were the primary failure modes.

There are several possible reasons for these early failures, but it has not been possible to identify one of them as the primary cause. The traffic volume was much higher on the replacement surfaces but probably not five times as high as the average over the first 16 years to justify the short service lives of the replacement surfaces. Truck weights have increased, but may not be as significant as gradual increases in tire pressure. Increased tire pressure translates directly into increased pressure on the road. Finally, quality control during application of replacement surfaces under traffic can never be as good as that on a new bridge where there is no traffic. The early failures of the replacement surfaces were probably caused by a combination of these factors.

Based on both temperature cycled flexural fatigue tests and cold temperature flexural fatigue tests, the best performance of a polymer concrete was that of Transpo T-48 epoxy concrete. Laboratory flexural fatigue performance of Flexolith epoxy concrete was a close second. Given the small numbers of specimens tested, the difference between these two materials was not significant. However, the Flexolith surface suffered from extensive bond failures at a very early age in the field test section. Degussa 330 MMA concrete and Polycarb epoxy concrete both cracked at significantly lower loads in the cold temperature flexural fatigue tests. These materials also failed in the field test sections.

One of the early fatigue tests indicated that the epoxy asphalt might have superior cracking characteristics, but density measurements indicated there were significant differences between the material in the test specimens furnished by the supplier (Adhesive Engineering Co.) and the material which was placed in the field test sections on the deck. Before additional test could be made, the field test sections of the epoxy asphalt concrete and the rubberized asphalt concrete had failed from rutting and shoving, and fatigue testing of both asphalt concretes was suspended.

When temperature cycled flexural fatigue tests of polymer concrete materials with an upper limit load of 1,450 lbs. caused cracking during the first exposure to 0°F, additional fatigue tests were conducted at an upper limit load of 1,150 lbs. Cracking in these specimens developed only after over 100,000 cycles of fatigue loading with temperatures below 5°F. In order to compare the fatigue test strain levels with those that actually occur on the bridge deck, strain measurements were made on the deck plate of the Poplar Street bridge. Data acquired over a six-week period indicate that the actual maximum stresses in the wearing surface may only be approximately 5% less than those produced by the 1,150 lbs. load in the laboratory.

This suggests that the reserve cracking resistance of the polymer concrete wearing surface is considerably less than one would like to have. However a statistical analysis of the data indicates that the maximum stress event occurs approximately 11 times per week. A very conservative assumption that the temperature of the deck is below 5°F for five weeks per year leads to the conclusion that the maximum stress event occurs only 55 times per year. It would thus take many years to accumulate the several thousands of cycles which would be required to cause cracking.

Based on the results of the limited number of temperature cycled flexural fatigue tests it appears that the construction joints in the Transpo T-48 epoxy concrete material perform as well as does the continuous material itself. It also appears that application of an epoxy tack coat to the hardened material just before the new material is placed produces slightly better results.

It should be pointed out that the number of fatigue tests conducted during this study is far less than the number needed to accurately characterize the behavior of polymer concrete materials at even one temperature, let alone the wide range of temperatures experienced by the bridge deck.

Unfortunately, a comprehensive fatigue study would be impractical given the time constraints before a decision must be made on the choice of a suitable wearing surface material.

Resistivity tests on the specimens after fatigue testing indicated higher resistivity in the Transpo T-48 specimens than any other specimens. This is consistent with resistivity results from the tests on the field test sections. This is probably a further indication that cracks, if they exist, in this material are well distributed and small in size.

Pull-out tests indicated that uniform, thin primer coats do not adversely affect the bonding of the wearing surfaces. However, there was considerable evidence that thick coats of primer do substantially reduce bond strength. Thus, the use of primer would increase the risk of debonding while actual in-service benefits of using a zinc-based primer are unknown.

4.2 RECOMMENDATIONS

Considering all the information presented earlier in this report, the authors recommend Transpo T-48 epoxy concrete on bare steel as the best choice at this time for the wearing surface of the Poplar Street bridge. Cracking is of concern, particularly at low temperatures.

Since fresh epoxy does not adhere well to hardened epoxy, construction joints should at least be minimized if not totally avoided. Edges of construction joints should be kept free of traffic so that the "aggregate keys" on these edges which serve to transfer shear and tensile stresses across these joints do not get dislodged and/or contaminated with oil, rubber or other impurities.

The authors believe that application techniques and quality control are extremely important.

REFERENCES

1. ACI Committee 503, "Use of Epoxy Compounds with Concrete," ACI Document 503R-89, Appendix A - Test Methods, ACI Manual of Concrete Practice, 1992, pp. 503R 30-32.

2. ALCOA, "Wear Surfaces for Aluminum Orthotropic Bridge Decking," ALCOA Laboratories Technical Report, 21 pp.

3. ASTM, " Standard Test Method for Electrical Resistivity of Membrane-Pavement Systems," ASTM Standard D 3633-88, ASTM Annual Book of Standards, Vol. 04.03, 1992, pp. 449-451.

4. Bild, S., "Durability Design Criteria for Bituminous Pavements of Orthotropic Steel Bridge Decks," Canadian Journal of Civil Engineering, Vol. 14, No. 1, February 1987, pp. 41-48.

5. Hartnagel, B. A., "Long-Term Deck Strain Measurements on an Orthotropic Steel Plate Bridge Deck," Master of Science Thesis, Department of Civil Engineering, University of Missouri - Columbia, May 1993, 64 pp.

6. Hartnagel, B.A., Gopalaratnam, V.S., and Baldwin, J.W., "Challenges in Long-Term Deck Strain Measurements on Orthotropic Steel-Plate Bridge Decks," Proceedings of the 9th National Conference on Microcomputers in Civil Engineering, Lake Buena Vista, Florida, October 1991, pp. 101-105.

7. Huber, G.A., "Deformation? We've Already Got the Solution!," Asphalt, Fall 1989, pp. 9-10,13.

8. Jones, C, "Review of Previously Used Methods of Pavement Testing and Evaluation," C.E.300 Honors Report, Department of Civil Engineering, University of Missouri-Columbia, March 1989, 30 pp.

9. Magruder, M., "Evaluation of Wearing Surfaces for Orthotropic Steel Plate Bridges," C.E.300 Honors Report, Department of Civil Engineering, University of Missouri-Columbia, March 1991, 34 pp.

10. Metcalf, C.T., "Flexural Tests of Paving Materials for Orthotropic Steel Plate Bridges," Highway Research Record No. 155, 1967, pp. 61-81.

11. Pasko Jr., T.J., "Polymer Concrete Used in Redecking a Major Bridge," Public Roads, Vol. 49, No. 3, December 1985, pp. 78-89.

12. Phillips, B. L., "Surfacing of Orthotropic Steel Decked Bridges," Report prepared for the Lower Yarra Crossing Authority, Melbourne, Australia, 1973, 139 pp.

13. Rigdon, R. L., "Wearing Surfaces for Orthotropic Steel Plate Bridges," Master of Science Thesis, Department of Civil Engineering, University of Missouri - Columbia, August 1990, 136 pp.

14. Rigdon, R. L., Gopalaratnam, V. S., and Baldwin, J. W., "Automated Temperature Varying Flexural Fatigue Tests for Steel-Polymer Concrete Composite Specimens, "Proceedings of the 9th National Conference on Microcomputers in Civil Engineering, Lake Buena Vista, Florida, October 1991, pp. 242-246.

15. Seim, C., "Laboratory Testing of the Epoxy Asphalt Pavement for the Luling Bridge," Report prepared for the State of Louisiana Department of Transportation and Development, August 1982, 37 pp.

16. Shelton, M., "Final Report - Poplar Street Bridge Test Sections," Materials Field Office, MHTD Division of Materials and Research, Jefferson City, Missouri, January 1991, 23 pp.

17. Shelton, M., "Final Report of Test Results - Poplar Street Bridge Test Sections," Materials Field Office, MHTD Division of Materials and Research, Jefferson City, Missouri, November 1991, 7 pp.

18. Sprinkel, M.M., "Thin Polymer Concrete Overlays for Bridge Deck Protection," Transportation Research Record No. 950, 1984, pp. 193-201.

19. Touran, A., and Okereke, A., "Performance of Orthotropic Bridge Decks," Journal of Performance of Constructed Facilities, ASCE, Vol. 5, No. 2, May 1991, pp. 134-148.

20. Wolchuk, R., "Steel-Plate-Deck Bridges," Section 19 in Structural Engineering Handbook, (Gaylord and Gaylord, editors), McGraw-Hill Book Company, New York, 1990, pp. 19-1 - 19-28.

21. Zieger, J.R., "Interim Report - Poplar Street Bridge Test Sections," Materials Field Office, MHTD Division of Materials and Research, Jefferson City, Missouri, December 1989, 59 pp.

22. Zieger, J.R., "Second Interim Report - Poplar Street Bridge Test Sections," Materials Field Office, MHTD Division of Materials and Research, Jefferson City, Missouri, February 1990, 39 pp.