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16. Abstract

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Materials for surface treatment of concrete bridge decks were tested as deterrents to freeze-thaw scaling damage. The most effective materials were identified, and methods and schedules of applications were developed. Air entrained concrete was found to be the most resistant to freeze-thaw scaling, and a mixture of linseed oil and kerosene was found to be effective in combating freeze-thaw damage at a relatively low cost.

Materials and methods for repairing decks with 1½ in. to 2 in. thick concrete overlays were developed, tested, and proved in field installations. It was found that a 2 in. thick plain concrete overlay bonded with grout to a well prepared base provides a durable overlay which adds to the stiffness of the slab and provides a good riding surface.

Materials and methods for structural repair of cracked or fractured concrete with epoxy resin was studied in the laboratory. Information and techniques developed from the laboratory investigation was demonstrated to Texas Highway Department personnel in four Districts.

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concrete, epoxy resin, latex modified concrete, steel fiber reinforced concrete, bridge decks, scaling, spalling, concrete deterioration, delamination, delamination detector, acoustical velocity, Windsor Probe, Schmidt Rebound Hammer, (below)	

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direct tensile test, bond strength epoxy injection, crack repair, deck repair, deck overlay.

BRIDGE DECK DETERIORATION

A SUMMARY OF REPORTS

by

Howard L. Furr

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Research Report 130-11F

A Study of Reinforced Concrete Bridge Deck Deterioration:
Diagnosis, Treatment, and Repair
Research Study 2-18-68-130

Sponsored by

The Texas Highway Department
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TEXAS TRANSPORTATION INSTITUTE
Texas A&M University
College Station, Texas

PREFACE

This is the eleventh, and final, report issued under Research Study 2-18-68-130, A Study of Reinforced Concrete Bridge Deck Deterioration: Diagnosis, Treatment and Repair. The previous ten are as follows:

1. "A Study of Concrete Bridge Deck Deterioration: Repair," by Raouf Sinno and Howard L. Furr, Research Report 130-1, Texas Transportation Institute, March, 1969.

2. "Reinforced Concrete Bridge Deck Deterioration: Diagnosis Treatment and Repair - Part I, Treatment," by Alvin H. Meyer and Howard L. Furr, Research Report 130-2, Texas Transportation Institute, September, 1968.

3. "Freeze-Thaw and Skid Resistance Performance of Surface Coatings on Concrete," by Howard L. Furr, Leonard Ingram and Gary Winegar, Research Report 130-3, Texas Transportation Institute, October, 1969.

4. "An Instrument for Detecting Delamination in Concrete Bridge Decks," by William M. Moore, Gilbert Swift and Lionel J. Milberger, Research Report 130-4, Texas Transportation Institute, August, 1970.

5. "Bond Durability of Concrete Overlays," by Howard L. Furr and Leonard L. Ingram, Research Report 130-5, Texas Transportation Institute, April, 1971.

6. "The Effect of Coatings and Bonded Overlays on Moisture Migration," by Leonard L. Ingram and Howard L. Furr, Research Report 130-6, Texas Transportation Institute, June, 1971.

7. "An Investigation of the Applicability of Acoustic Pulse Velocity Measurements to the Evaluation of the Quality of Concrete in Bridge Decks," by Gilbert Swift and William M. Moore, Research Report 130-7. Texas Transportation Institute, August, 1971.

8. "Concrete Resurfacing Overlays for Two Bridge Decks," by Howard L. Furr and Leonard L. Ingram, Research Report 130-8, Texas Transportation Institute, August, 1972.

9. "Detection of Bridge Deck Deterioration," by William M. Moore, Research Report 130-9, Texas Transportation Institute, August, 1972.

10. "An Investigation of Concrete Quality Evaluation Methods," by Rudell Poehl, Gilbert Swift and William M. Moore, Research Report 130-10, Texas Transportation Institute, November, 1972.

This research was conducted at the Texas Transportation Institute as part of the cooperative research program with the Texas Highway Department and the United States Department of Transportation, Federal Highway Administration.

The authors wish to acknowledge their gratitude to all members of the staff of Texas Transportation Institute who contributed to this research.

The support given by the Texas Highway Department personnel is also greatly appreciated, especially that of Mr. M. U. Ferrari and Mr. Don McGowan who provided advice and assistance throughout the study and that of the maintenance personnel of the Districts who helped in the bridge deck investigations.

ABSTRACT

Reports on 3 separate phases of a study of bridge deck deterioration are summarized. Full details are not given in this report, but they are given in the 10 reports which are referred to in the body. The 3 phases, diagnosis, treatment, and repair deal with the problems of scaling, spalling, and delamination.

A detector was developed which maps the delaminated area of concrete on a bridge deck. It is an electronically controlled instrument which interprets acoustical signals taken from the concrete as the instrument cart rolls along the bridge. A prototype was constructed and conditioned for field service.

Materials for surface treatment of concrete bridge decks were tested as deterrents to freeze-thaw scaling damage. The most effective materials were identified, and methods and schedules of applications were developed. Air entrained concrete was found to be the most resistant to freeze-thaw scaling, and a mixture of linseed oil and kerosene was found to be effective in combating freeze-thaw damage at a relatively low cost.

Materials and methods for repairing decks with 1 1/2 in. to 2 in. thick concrete overlays were developed, tested, and proved in field installations. It was found that a 2 in. thick plain concrete overlay bonded with grout to a well prepared base provides a durable overlay which adds to the stiffness of the slab and provides a good riding surface.

Materials and methods for structural repair of cracked or fractured concrete with epoxy resin was studied in the laboratory. Information and techniques developed from the laboratory investigation was demonstrated to Texas Highway Department personnel in four Districts.

Key Words: Concrete, epoxy resin, latex modified concrete, steel fiber reinforced concrete, bridge decks, scaling, spalling, concrete deterioration, delamination, delamination detector, acoustical velocity, Windsor Probe, Schmidt Rebound Hammer, direct tensile test, bond strength, epoxy injection, crack repair, deck repair, deck overlay.

SUMMARY

A five year study of reinforced concrete bridge deck deterioration: Diagnosis, Treatment, and Repair is summarized in this report.

The research has as its broad objective the development of methods for detection, treatment, and repair of deteriorated concrete. It is divided into three parts, the individual objectives of which are given below.

1. Part 1, detection and evaluation of deterioration in reinforced concrete bridge decks, will seek to:
   (a) Develop a method or methods, a device or devices, and a procedure that can be used by maintenance personnel in carrying out operations named in items (b), (c), and (d) below.
   (b) Discover deterioration that might exist in an existing reinforced concrete bridge deck.
   (c) Identify the nature of the deterioration, such as vertical cracking or horizontal cracking.
   (d) Determine the extent of deterioration.
2. Part 2, treatment of reinforced concrete bridge decks, will seek to:
   (a) Identify materials for application as surface treatments that will stop or attenuate deterioration that has not begun or is in its early stages.

(b) Develop techniques and patterns of treatment.

(c) Determine the degree of effectiveness of treatments against deterioration.

3. Part 3, development of effective overlays and patches for repair of badly deteriorated reinforced concrete bridge decks, will seek to:

   (a) Determine an effective and practical treatment of a deck in need of major repairs to receive and to bond to an overlay of concrete.

   (b) Determine the mix proportions of portland cement concrete and of other materials that appear to be promising that will serve most effectively as an overlay material of thickness as might be required for the repair.

   (c) Determine how the overlay can be effectively bonded to an old concrete.

   (d) Determine the effective thickness that can be used as a portland cement concrete overlay.

   (e) Develop a technique for reinforcing the overlay if reinforcing is required.

Details of tests performed and developments of equipment and techniques to meet the objectives of the study are given in 10 published reports (1-10). Implementation statements are provided, where applicable, in those 10 reports. Because of additional information developed after those reports were written, implementation statements are given in this report in each major section. Each of the major sections represents one of the 4 particular divisions of the study.

One phase of the study was directed toward diagnosing deterioration in concrete bridge decks. The first section of this report summarizes the work of that phase. A device was developed for detecting delamination, and studies were made of devices for evaluating the quality of concrete in the service structure.

In the second phase of the work materials and methods for treating concrete surfaces to prevent freeze-thaw scaling were studied. The study reconfirmed the value of air entrainment for concrete exposed to freeze-thaw action. It also confirmed findings by others that treatments of the surface of new concrete decks with a mixture of 50 percent kerosene with 50 percent boiled linseed oil or 50 percent tung oil are effective in delaying, but not completely preventing, serious freeze-thaw scaling. Skid resistance measurements with the British Portable Tester showed that surface treatments reduce the skid resistance, but the reduction was smallest when the linseed oil mixed with kerosene was used.

The third major effort of the study was concerned with bridge decks that have deteriorated to the point of requiring major repairs. Patches and overlays of resinous concrete and portland cement concrete were studied. It was found that thorough removal of deteriorated concrete and thorough cleaning of the sound concrete surface are requirements for providing a durable patch or overlay. Overlays 1 1/2 to 2 in. thick can be bonded to a clean sound concrete surface with cement-sand grout or by epoxy resin. Bond strengths of overlay were tested by shear tests and tensile tests. The lowest shear bond strengths of overlay placed on new concrete were 245 psi with epoxy bonding, 388 psi with no agent placed and 592 psi with grout bonding (Reference 8, Table VI). The low tensile bond strengths were 105 psi for epoxy, 105 for no bonding agent and 263 psi with grout. Repeated loading of overlaid beams to 2 million load applications developed no bond failures. Freeze-thaw tests of portland cement concrete overlays developed no bond failures.

The final phase of the study on repair was concerned with repair of structural cracks and spalls caused by impact loads. The objectives of this phase of the program were as follows:

1. Review the available literature which deals with work of this type and study the methods which appear to be the most promising.
2. Study materials and equipment to determine which system appears to be best suited for repair of each type of structural crack.
3. Study methods of crack preparation.
4. Determine the extent of penetration of epoxy into cracks by injection methods.
5. Study strength of repaired concrete in static and repeated load tests.
6. Make freeze-thaw tests to determine if freeze-thaw action influences the repair.
7. Make repairs to damaged structures of typical bridge beams and piers.

8. Monitor the repaired structures.

A review of the available literature was made to determine methods and equipment available for repairs of this nature. Equipment was selected for injection of epoxy resin under pressure for crack repairs. It was found that epoxy repairs can restore structural integrity but care must be exercised in selecting the proper material for the intended use in order to insure a lasting repair.

It was found that repairs can be made effectively by bonding dislodged pieces together and/or by pressure injection. Flexural tests on repaired laboratory beams showed that the bond between pieces glued together after being broken apart was stronger than the concrete. In repeated load tests it was found that the flexural stiffness can be restored to laboratory beams and that the 2 million repeated load cycles did not cause any bond failures. In freeze-thaw tests it was found that the repaired and unbroken prisms were affected about the same by 50 freeze-thaw cycles.

IMPLEMENTATION

This report summarizes research performed in detection, treatment, and repair of deteriorated concrete. It is divided into four parts, each part treating a certain phase of the research. Because each section of the report is separate and not directly related to the other sections, an implementation statement appears under each of the separate sections. Those statements are located as follows:

Section 1, page 13.

Section 2, page 33.

Section 3, page 47.

Section 4, page 81.

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SECTION 1. DIAGNOSING BRIDGE DECK DETERIORATION
by William M. Moore and Gilbert Swift

Introduction

This section summarizes the research efforts in the phase of the study which was directed toward diagnosing deterioration in concrete bridge decks. Principal emphasis in this phase has been placed on techniques for evaluating a) delamination and b) poor quality concrete. These two factors were considered to be of primary importance in bridge decks. Research Report 130-4, entitled "An Instrument for Detecting Delamination in Concrete Bridge Decks," described the development and preliminary testing done in the "delamination" portion of the research. Research Report 130-7, entitled "An Investigation of the Applicability of Acoustic Pulse Velocity Measurements to the Evaluation of the Quality of Concrete in Bridge Decks" described the development and initial testing directed toward the detection of poor quality concrete. In both of these research efforts, primary emphasis was placed on non-destructive testing techniques for diagnosing deterioration. In later research, some slightly destructive concrete evaluation techniques were investigated. The several techniques for detection of bridge deck deterioration are described and the results obtained are summarized in Research Report 130-9. The detailed results obtained with four concrete quality evaluation methods are given in Research Report 130-10.

Each of the diagnostic techniques is discussed briefly in the following paragraphs.

Delamination Detection

Probably the most serious form of deterioration commonly found in reinforced concrete bridge decks is delamination which ultimately results in large scale spalling. This type of failure occurs most frequently where salt is used for winter deicing and is believed to result chiefly from salt induced corrosion of the reinforcing steel (11, 12, 13, 14). The Delamination Detector, which is a device for locating this type of failure, was developed in this study (4). (See Figure 1). It is roughly the size and shape of a power lawn mower and has been found to be reliable and easy to operate.

Verification of the ability of the Delamination Detector to detect delamination was made by coring two locations in each of ten bridge decks, one in an indicated and another in a non-indicated area. Complete agreement was obtained. The delaminations in these bridge decks varied in depth from 1/2 to 4 1/2 in. Several of the decks had asphaltic surfacing layers which varied in thickness from 1/4 to 3 1/2 in. These successful tests were thought to be very significant because the distinctive "hollow sound" produced by conventional sounding techniques is greatly diminished when delaminations in a deck become deep or when a deck is resurfaced with an asphaltic layer.

The instrument has been used by maintenance personnel in several Texas Highway Department Districts. Probably the most extensive use was by the maintenance personnel in the El Paso District who surveyed about 130 bridges, most of which had asphaltic concrete or epoxy

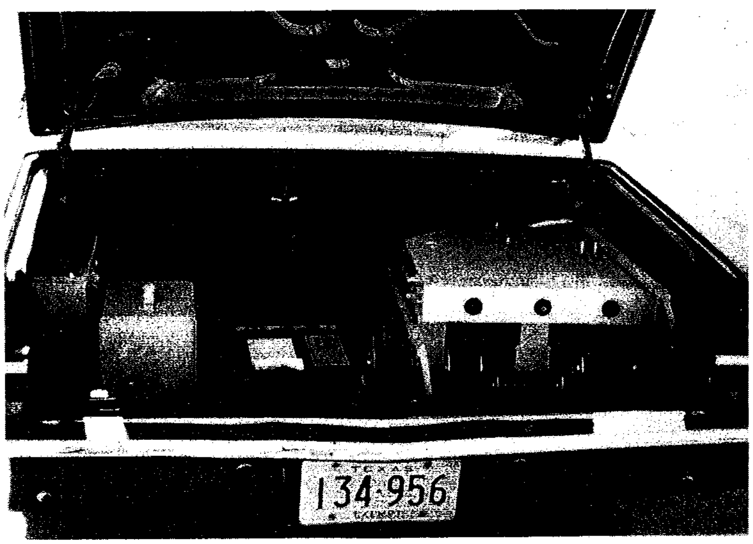


FIGURE 1: Instrument developed to detect delamination on bridge decks. It can be disassembled and stowed in an automobile trunk.

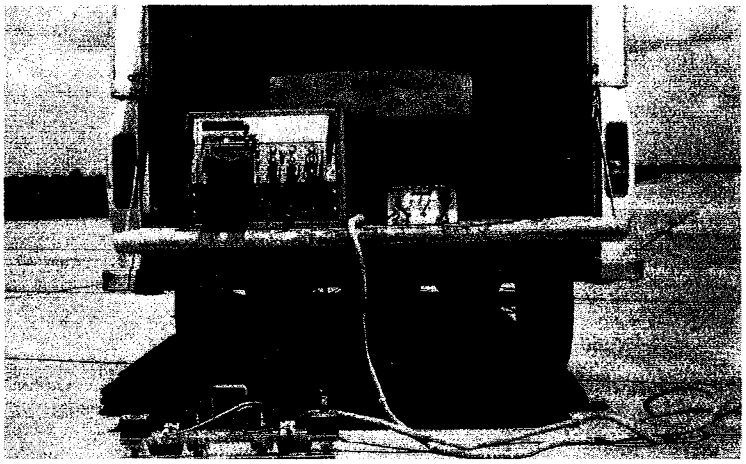
overlays. Maintenance personnel found the instrument to be a reliable and effective means for delamination detection. It appears particularly valuable for evaluating many overlayed bridges upon which conventional sounding techniques have been found to be ineffective.

Acoustic Velocity

An investigation into the suitability of acoustic velocity measurements for determining concrete quality indicated that the compressional wave velocity could be measured from the accessible upper surface of a bridge deck (7). It was also learned that the elastic modulus of concrete could be reliably estimated from the concrete's compressional velocity and unit weight.

A portable field-type velocity measuring instrument was developed in this study for use on bridge decks (Figure 2). This device employs a probe which places an array of four acoustic transducers into contact with the concrete. Velocity is measured, using the "timing along" technique, by observing the time of travel of the acoustic waves between two identical receiving transducers. Waves are produced and propagated successively in opposite directions and the two time-intervals are measured and averaged in order to cancel coupling delay errors.

Cores were taken from many different slabs which had been measured with the velocity meter. The measured velocities were found to be slightly correlated with the measured core compressive strengths. In a linear regression to determine core strength from velocity the coefficient of variation was found to be 19.9 percent. Thus the measured velocity can be used to estimate the core strength, to within 20% or better, two thirds of the time.



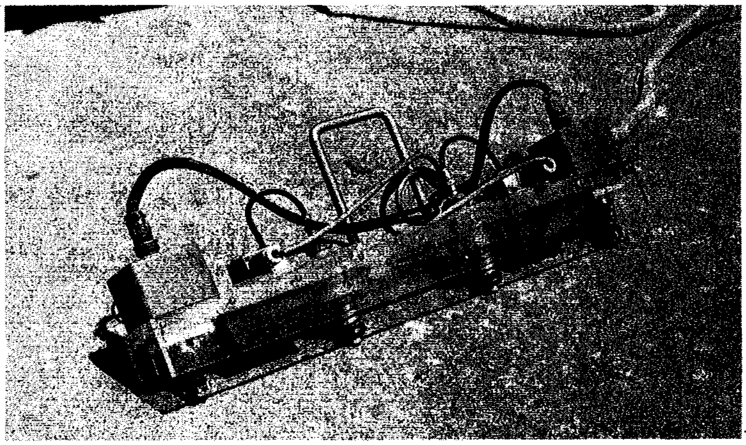


FIGURE 2: The control unit of the field-type velocity measuring instrument is conveniently operated from a pickup tail gate as the probe unit is moved to various points on the deck.

It was found on many bridges which had been in service for several years that it is difficult to measure the acoustic velocity due to the attenuation introduced by numerous surface cracks. Often these surface cracks were visible only after moistening the surface. This problem was not encountered on the newer bridge slabs.

This instrument is believed to be a useful research tool when it is desirable to non-destructively estimate the elastic modulus of the concrete. It is not believed to be practical for routine bridge deck inspections, because too many bridges are difficult to measure, even for a highly trained operator.

Windsor Probe

The Windsor Probe Test System has been used in field investigations to estimate the in-situ strength of concrete in pavements, bridges, walls, pipes, etc. (15, 16, 17). The device is easy to use and requires no surface preparation prior to testing. Basically the tests consist of shooting a standard probe into the concrete with a standard explosive cartridge. The depth of penetration is determined by measuring the height of the exposed probe. A special gun or driver unit is provided for shooting the probes (Figure 3). Gage plates are also provided to facilitate the measurement of the average height of the exposed probes in a standard group of three shots. The higher the probes, i. e., the more resistant to penetration, the stronger the concrete.

Cores were taken from many different slabs which had been measured with the Windsor Probe. The average probe heights were found to be weakly correlated with the measured compressive core strengths. In a linear

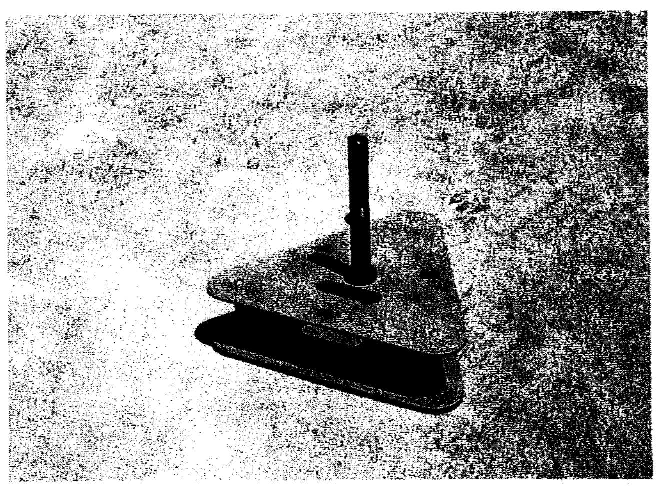


FIGURE 3: A standard stud is shot into concrete with the Windsor Probe Test System to determine concrete quality. The average penetration resistance of three shots is measured using a depth gage and special templates.

regression to estimate core strength from the probe height the coefficient of variation was found to be 20.3 percent.

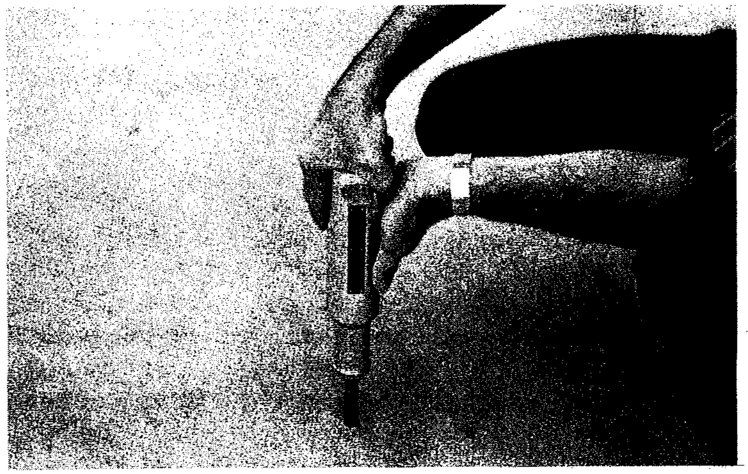
The system is believed to be practical for bridge deck survey measurements to locate deteriorated areas. The test is slightly destructive. In addition to the small hole made by the probe penetration, a spall about 6 in. in diameter and up to 3/4 of an in. deep at the center is often produced by the test.

Schmidt Rebound Hammer

The Schmidt Rebound Hammer is a very widely used instrument for estimating the quality of in-situ concrete. Basically, the test consists of striking a rod, in contact with the concrete, with a standard hammer and measuring the height of the hammer rebound. The higher the rebound the stiffer (and better quality) the concrete.

Several authors have suggested that the Schmidt rebound hammer can be used to estimate the compressive strength of concrete in-situ (18, 19, 20). They agree that the type of coarse aggregate, surface condition of the concrete, its moisture condition, etc., have a pronounced effect on the results of the relationship between rebound reading and strength. Also there is common agreement that the instrument can be used to determine the uniformity of concrete and thus is an effective tool for locating weak spots. A Soiltest Model CT200 rebound hammer was used for investigation in this study (Figure 4).

The results from this test were found to be slightly correlated with the compressive strengths of cores taken from the many different slabs.



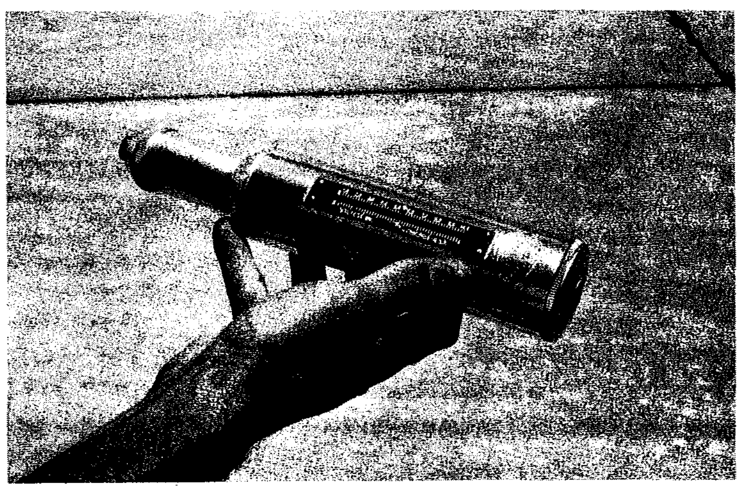


FIGURE 4: A Soiltest Model CT200 rebound hammer is used to estimate concrete strength. Prior to testing the surface is smoothed with a hand grinder.

In a linear regression to estimate core strength from rebound value the coefficient of variation was found to be 21.2 percent. The instrument is fast and easy to use and is believed to be practical for bridge deck measurements to locate deteriorated areas.

Direct Tensile Test

An important characteristic of concrete, which is seldom considered in field evaluations, is its tensile strength. This characteristic is highly significant in quality bridge deck construction.

In 1956, the Shell Chemical Corporation introduced a "Highway Tensile Tester". This tester was developed for evaluating the quality of resinous cement overlays and to pre-evaluate the surfaces upon which they were to be applied. A device similar to the Shell tester was fabricated in this study (Figure 5). The chief modification was that a hydraulic cylinder, instead of a screw, was used to apply tension in order to eliminate the possibility of horizontal forces on the screw handle being converted into unwanted tension. To measure the in-place tensile strength of the concrete, a 2 in. diameter cylinder is bonded to the surface of the bridge deck with an epoxy adhesive. After the adhesive has cured, the cylinder is pulled from the deck causing a tensile fracture to occur in the concrete.

The results from this test were somewhat better correlated with the compressive strength of cores taken from different slabs than any of the other techniques investigated. In a linear regression to estimate core strength from the average tensile strength the coefficient of variation was found to be 17.4 percent.

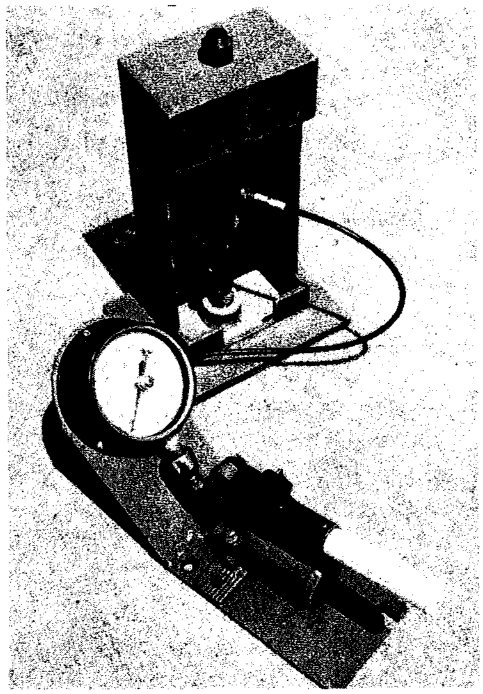


FIGURE 5: Two-inch diameter cylinders are epoxied to a smooth clean concrete surface. After about 90 minutes for curing, tension is applied to pull out a dome shaped piece of concrete.

The tensile test is somewhat time consuming. It requires a curing time of about 90 minutes for the epoxy to harden prior to testing. On a warm day about 40-50 tests could be made in an 8-hour day. The test is probably a better indicator of the general quality of the concrete in the deck than the other techniques and it is believed to be practical for bridge deck measurements.

Conclusions: Diagnostic Phase

1. The technique utilizing the delamination detector developed in this study has been found to be practical and effective for determining the extent of delamination in bridge decks.
2. The acoustic velocity meter developed in this study was found useful to estimate the dynamic modulus and the chord modulus of concrete in-situ. Other instruments investigated were deemed more suitable for the routine detection of poor quality concrete by maintenance personnel.
3. The Schmidt hammer, the Windsor probe, a tensile test device fabricated in this study, and the acoustic velocity meter were each found to be applicable to a survey procedure for detecting weak spots, although each instrument responds to a somewhat different set of properties of concrete. In the opinion of the authors, the Schmidt rebound hammer provided the most rapid and economical technique for detecting deteriorated or low quality concrete. The tensile test, though slow, is believed to be the one which best indicates the quality of the concrete.
4. It was concluded that core compressive strength is not a direct measure of environmental deterioration or loss of quality in concrete.

5. Attenuation of acoustic waves was found to be greatly influenced by environmentally induced micro-cracking of the concrete. It appears that an instrument could be designed to measure the acoustic attenuation of bridge decks and might prove to be a superior detector of deterioration.

Implementation: Diagnostic Phase

The delamination detector has been found to be a practical and fieldworthy tool for use by highway maintenance personnel. Research has demonstrated that multiple-path automatic detection is practical.

The Direct Tensile Test, Velocity Meter, Rebound Hammer, and Windsor Probe are each designed to measure a different characteristic property of in-situ concrete. All of them have merit and any one of them can be used to locate weak spots or deterioration in bridge decks. These tests can supply valuable data to the engineer faced with the problem of possible major maintenance of a bridge deck.

SECTION 2. SURFACE TREATMENT
by Howard L. Furr and Leonard L. Ingram

Introduction

This phase of the research is concerned with preventing deterioration of the surface of the bridge slab. The deterioration is assumed to be surface scaling caused by freeze-thaw action and the use of deicing salts. Materials that may be sprayed or brushed on the surface as coatings and penetrants are studied in the laboratory for effectiveness, method of application, rate and schedule of application, resistance to abrasion, and costs. The performance of air entrained concrete is compared with that of non-air entrained concrete in the studies. The skid resistance of some of the treated surfaces are compared through laboratory tests.

Tests conducted in this phase of the program are divided into 3 groups which are listed below along with their general objectives.

1. Water absorption tests on concrete specimens coated with various materials to determine the effectiveness of the coatings in sealing out water.
2. Skid tests to determine the effect of coating materials on the skid resistance of concrete.
3. Freeze-thaw tests to determine the effectiveness of coating materials in preventing surface scaling of concrete.

Details and results of these tests are given in research reports 130-2, 130-3, and 130-6 (2, 3, 6). The information contained in those reports is summarized in sections that follow.

The individual tests series of this phase of the program are listed in Table 1. A brief statement of objectives and results of each series of tests are also given. The detailed tests listed in the table are reported primarily in Reference 3 and secondarily in References 2 and 6.

This section gives only a summary of the material that has been reported earlier in References 2, 3, and 6. The reader should refer to the full reports if more complete details are desired.

Test Summaries and Results

1. Water absorption tests:

A study was made to determine the effectiveness of various materials in preventing entry of water into concrete. It was reasoned that if entry of water could be prevented, surface scaling due to freeze-thaw action would be minimized.

Non-air entrained concrete blocks 3 in. x 3 in. x 4 in. served as test specimens. They were moist cured 7 days and air dried a minimum of 21 days. They were then coated on 5 sides and dried at room temperature. They were placed in approximately 2 in. of water with the coated finished surface on the bottom. Both tap water and salt water were used. The salt water was prepared by mixing sodium chloride salt, 5 percent by weight, and tap water. The specimens were alternately soaked at room temperature for 24 hours, then removed from the bath and dried at 140°F temperature for 24 hours. They were weighed after each soaking and drying period. After the wet and dry weights stabilized, the average absorption was determined.

TABLE 1. A SUMMARY OF TESTS, OBJECTIVES, AND RESULTS

Test	Objective	Results
Absorption (a)	To determine sealing effect of coatings on concrete	Epoxy & Hot LO had the least absorption; Watco and LO mixture next best. (b)
Freeze-thaw (a)	To find the top ranking coatings for further study	Found to be among best; Hot LO, Tung mix, LO mix, TWS, Epoxy. (b)
Abrasion (a)	To determine effect of wear on sealant	Tung mix, Coal tar, LO mix and Hot LO performed best. (b)
Ultraviolet Light (a)	To find if sun breaks down sealant	No ill effects noted. (b)
1 Freeze-thaw	To find if temperature during 21-day drying affects F-T cycles	With the exception of specimens coated with Hot LO drying at 100°F reduces F-T durability.
2 Freeze-thaw	To find the F-T durability effect of pavement temperature at the time of, and continuing after, coatings.	Scaling was delayed but after it started it continued. Final results show no effect.
3 Freeze-thaw	To find effect of treatment <u>after</u> scaling had begun	F-T durability somewhat greater than with no treatment but less than if treated before scaling.
4 Freeze-thaw	To find effect of temperature and humidity during 21-day drying (only LO mix & no treatment tested)	High humidity during drying enhanced deterioration. Low humidity during drying increased durability.
5 Freeze-thaw	To find if EpoXeal should be applied during rising or lowering temperature	No difference in F-T cycles noted.

TABLE I. (Cont'd.)

Test	Objective	Results
6 Freeze-thaw	To determine if cracked concrete can be sealed with penetrants. Air-entrained and non air-entrained concretes were used	Scaling developed at cracks which indicates that coatings did not seal the concrete at the crack. Entrained air protected those specimens made of air-entrained concrete.
7 Freeze-thaw	Effect of air-entrainment. See also test 6 above.	F-T durability greatly increased.
8 Skid Resistance	To determine the effect of coatings on skid resistance using British Portable Tester	Best: No treatment. Next: LO Next: Tung Next: TWS

(a) These tests were reported in an earlier report 2.
(b) Boiled Linseed Oil and kerosene mixed in equal volumes. Details are given in Reference 2.

Definitions:

LO - Linseed oil

EpoXeal - commercial product

Watco - commercial product

TWS - Thompson's Water Seal, a commercial product

F-T - Freeze-thaw

A second series of tests on the blocks were made after sandblasting the treated finished surface. All test details were the same as those for the blocks described above. The blocks were sandblasted with Ottowa sand. The 1/4 in. diameter nozzle delivered 680 grams of sand at 60 psi air pressure over a 30-second period at a distance of 11 1/4 in. from the face of the block.

In each of the series of tests on the blocks, 11 sets of specimens were used. One, the control, was not coated; 3 were coated with different linseed oil products, and seven were coated with different products which included tung oil, epoxy resin, and five trade products: WATCO, Horntraz, Thompson's Water Seal, Jennite J-16, and Tropicure Silicate Curing Agent.

It was found that some treated specimens absorbed more water than untreated ones. On the average, sandblasting caused a slight increase in absorption in salt water but it caused a slight decrease in tap water absorption; and, on the average, considerably more tap water was absorbed than salt water. Linseed oil and epoxy treatments were the most effective in preventing water absorption.

Later tests were made on 10 in. square x 2 in. thick blocks to compare sealing effectiveness of four coatings in preventing ingress of moisture through the coatings. A mixture of linseed oil and kerosene, tung oil and kerosene, Thompson's Water Seal, and Epoxeal were used as surface coatings. Tap water and salt water, (5 percent sodium chloride and tap water) were ponded to 1/2 in. depth on the treated finished surfaces. Monfore moisture gages were set at elevations 1/8 in., 1/4 in., and 3/8 in. below the treated surfaces to detect moisture changes. These tests indicated that water penetrated to the gages quicker through the uncoated surfaces than through coated surfaces. The time required for the salt water to penetrate the uncoated concrete and be detected by the top gage was less than that for tap water (20 min. vs. 60 min, respectively). Detection time for salt water, however, was longer than for tap water in penetrating through the coatings. The tung oil-kerosene mixture was the most effective in delaying penetration to 3/8 in. depth for both tap water and salt water. Information on penetration times is tabulated in Table 2.

2. Skid tests:

Tests were made to compare skid resistance of uncoated specimens and coated specimens. Only the materials which appeared at the time to be among the most effective among penetrants in freeze-thaw durability were included in these tests.

The British Portable Tester, BPT, was used to test the finished surface of 10 in. square blocks 2 in. thick. Surfaces were given a rough finish with a wood screed, and they were moist cured 7 days and air dried 21 days. Skid tests were then made on wet uncoated finished surfaces. Coatings of mixed linseed oil and kerosene, tung oil mixed with kerosene, hot linseed oil, and Thompson's Water Seal were applied to the surfaces that had been tested. After the coatings dried 24 hours, the wet coated surfaces of the specimens were again tested.

Tabulated test results are shown in Table 3. The average British Portable Number, BPN, from samples of each of 6 sets of specimens was 61.6 before coating and 48.0 after coating. The average difference in the BPN values was 13.6. Values ranged from a low of 58 to a high

TABLE 2. MOISTURE PENETRATION THROUGH SURFACE COATINGS

Coating	Water Type	Penetration time, hours, for R.H. to reach 100%		
		1/8 in depth	1/4 in depth	3/8 in depth
None	Tap	1.0	1.8	2.9
None	Salt	0.3	1.0	2.5
LO+K*	Tap	0.5	7.0	24.
LO+K	Salt	1.5	30	192
TWS**	Tap	21	27	66
TWS	Salt	24	27	60
TO+K***	Tap	2	24	120
TO+K	Salt	192	312	312

* Linseed oil mixed with kerosene on a 50%-50% volume basis.
** Thompson's water seal.
*** Tung oil mixed with kerosene on a 50%-50% volume basis.

TABLE 3. SKID RESISTANCE PERFORMANCE -- WET SURFACES

Code (Table 4)	Treatment	Specimen 1		Specimen 2		Specimen 3		Avg. of 3		
		BPN Before	BPN After	BPN Before	BPN After	BPN Before	BPN After	Before	After	Diff.
1	1 coat (50-50) Linseed Oil	55	37	63	58	65	63	61	53	8
2-a	2 coats (50-50) Linseed Oil	59	52	56	44	59	52	58	49	9
3	3 coats (50-50) Linseed Oil	63	53	68	57	62	55	64	55	9
4	1 coat hot Linseed Oil	63	47	60	45	62	47	62	46	16
7	Thompson's Water Seal	67	41	59	36	55	34	60	37	23
8-a	2 coats (50-50) Tung Oil	60	45	65	54	68	44	64	48	16

Avg. of all before: 61.6

Avg. of all after: 48.0

Difference 13.6

of 64 before coating and from 37 to 55 after coating.

The linseed oil-kerosene mixes caused a drop of 8 to 9 in BPN, the smallest of the series. The hot linseed oil and the tung oil mixtures produced drops of 16 in BPN, and the Thompson's Water Seal caused a drop of 23.

3. Freeze-thaw scaling tests:

These tests were made to determine the influence of several variables on surface scaling due to freeze-thaw action. The primary variables were the coating overlay applied to the top surface of concrete. Quantities of coating material, methods of application, curing conditions, and temperature conditions at the time of and immediately subsequent to application of the coatings were also studied. Both air entrained and non-air entrained concretes were used in test specimens.

The specimens were 10 in. square by 2 in. thick concrete blocks finished off on the top surface with a wood screed. They were made of Type III portland cement and natural sand and gravel aggregates. After 7 days of moist cure, specimens were dried in the laboratory at different combinations of temperature and relative humidity. Table 4 gives information on coatings, and Table 5 gives information on curing conditions. An asphaltic concrete overlay is also described in Table 4.

Coatings were applied to the dry finished surface. Metal rings were attached and sealed to the coated surface after coatings were dry. The well formed by the ring was then filled with the salt water solution. The specimens were then moved to the 0°F room for freezing and then to the 40°F room where the water thawed. A 6-hour freeze and a

TABLE 4. DESCRIPTION OF COATINGS AND OVERLAY
(See note below table)

Coating Number	Description
0	No Coating.
1	One coat of 50-50 linseed oil and kerosene.
2-a	Two coats of 50-50 linseed oil and kerosene.
2-b	Two coats of 50-50 linseed oil and kerosene on specimens after scaling had begun.
3	Three coats of 50-50 linseed oil and kerosene.
4	One coat hot, 180°F, linseed oil.
5	Jennite (J-20 primer, J-16 sand slurry).
6	Asphalt (MC-0 primer, Ampet AC-5 with a one grain thickness of sand rolled into the AC-5).
7	Two coats Thompson's Water Seal, (TWS).
8-a	Two coats of 50-50 tung oil and kerosene.
8-b	Two coats of 50-50 tung oil and kerosene on specimens after scaling had begun.
9	One coat of EpoXeal with a touch-up coat.
Overlay	A 1 1/2 inch thick asphaltic concrete overlay made up as follows: A seal coat, 120-150 penetration asphaltic cement and intermediate grade lightweight aggregate (Ranger), applied to the concrete surface. Three days later, a tack coat of EA-HVMS emulsified asphalt, high viscosity, medium setting, with 2 percent latex rubber solids was applied to the seal coat. After the water had evaporated from the emulsion, a hot mix overlay was compacted over the emulsion.

Note: Coatings were reported in Reference 3.
The overlay was reported in Reference 6.

TABLE 5. SCHEDULE OF POST-CURE DRYING AND COATING MATERIALS (DRYING FOLLOWING IMMEDIATELY AFTER MOIST CURING IS DESIGNATED POST-CURE DRYING)

Number of Blocks per Coating	Air Content (%)	21-day Post-Cure Drying Condition Temp. (°F)	R.H. (%)	Coating Used (Code Number, Table 4)
3	0	100	50	1,2-a,3,4
3	0	73	50	0,1,2-a,3,4,5,6,7, 8-a,9
3	0	73	50	0,2-b,7
3	0	73	50	0,2-a,2-b,8-a,8-b
3	0	73	25	0,1
3	0	73	50	0,1
3	0	100	50	0,1
3	0	100	75	0,1
3	0	140	25	0,1
3	0	73	50	9 (3 sets)
3	0	73	50	0,2-a,5,7,8-a,9
3	5	73	50	0,2-a,5,7,8-a,9
3	5	73	50	1,2-a,3,5,6,7
3	0	73	50	1,2-a,3,4,7-a,8-a

6-hour thaw cycle was followed throughout the test. Cycling was interrupted at one to two week intervals, depending on the progress of scaling. The surfaces were then flushed, a visual inspection was made, the loose scale was brushed away, and the wells were recharged with salt water. The inspected surfaces were rated by number according to the severity of scaling - number 1 being for no scaling and number 5, the highest, for severe scaling.

Coatings and overlays of asphalt and tar had to be scraped away for inspection. Specimens with such coatings were prepared in such numbers that some could be withdrawn at certain intervals for inspection, and after inspection they were discarded.

For specimens dried in the laboratory (73°F, 50% RH) after curing, the tung oil and kerosene mixture performed best. The next high performers were Epoxeal and linseed oil and kerosene mixtures, in that order. Table 6 summarizes the results of the test.

Tests were made on two coatings to determine if the length of drying time at 140°F after curing had an influence on the severity of scaling. Table 7, giving the results of the test, shows that the longer drying time at 140°F prior to coating resulted in more scaling.

The effects of different drying conditions after curing and prior to coating for one material are shown in Table 8. A long drying period at high temperature and low relative humidity produced the most resistance to scaling for surfaces treated with linseed oil mixed with kerosene. The non-treated surfaces, on the other hand, performed best when cured at 73°F and 25 percent relative humidity.

TABLE 6. FREEZE-THAW SCALING IN SPECIMENS DRIED IN 73° F, 50% RH

Coating Code (Table 4)	Number of F-T Cycles Producing Indicated Scaling	
	Moderate Scaling	Severe Scaling
0	5	28
1	48	84
2-a	37	71
3	55	84
4	18	24
5	28 (a)	
6	28 (a)	
7	15	43
8-a	118	170
9	48	134

(a) Coatings were removed for inspection of the concrete surface. Specimens were then discarded.

Overlay -- No deterioration was found under the rubberized asphalt overlay after 25, 38, and 59 freeze-thaw cycles.

TABLE 7. EFFECT OF 140°F CONCRETE ON PERFORMANCE OF LINSEED OIL-KEROSENE AND TWS COATINGS

Coating Code (Table 4)	Coating	Number of Hours That Specimen Was Held at 140°F Prior to Coating (a)	Freeze-Thaw Cycles Producing: Moderate Scaling	Severe Scaling
0	None	0	12	18
0	None	6	7	18
0	None	24	6	18
2-b	2-LO+Kerosene	0	32	36
2-b	2-LO+Kerosene	6	27	36
2-b	2-LO+Kerosene	24	22	27
7	TWS	0	13	28
7	TWS	6	15	25
7	TWS	24	12	25

Non air-entrained concrete; 7-day moist cure; 21 days at 73°F, 50% RH drying.

(a) All specimens were dried 24 hours in 140°F, 25% RH chamber after the coating operation. They were then placed into F-T cycling.

TABLE 8. TEMPERATURE AND RELATIVE HUMIDITY EFFECT ON FREEZE-THAW SCALING -- BOILED LINSEED OIL AND KEROSENE COATING

Coating Code (Table 4)	21 Day Drying Condition		Cycles to Produce Moderate Scaling	Cycles to Produce Severe Scaling
	Temperature (°F)	Relative Humidity (%)		
0	73	25	5	61
1	73	25	59	64
0	73	50	5	28
1	73	50	58	73
0	100	50	5	35
1	100	50	57	74
0	100	75	5	44
1	100	75	32	48
0	140	25	5	51
1	140	25	67	85

Tests were made to determine if the direction of temperature change (increasing or decreasing) at the time of coating had an influence on freeze-thaw resistance. The results, given in Table 9, indicate that there is no significant difference in severe scaling with the direction of temperature change at time of treatment.

Tests on air entrained and non-air entrained specimens which were cracked prior to coating revealed that the coatings used in these tests did not significantly reduce the rate of scaling at the crack. The air entrained concrete was, however, much more resistant to scaling than the non-air entrained material. Additional tests on non-cracked specimens of air entrained and non-air entrained concrete further emphasized the superiority of the air-entrained material in freeze-thaw action.

Quantities of materials used in the tests are given in Table 10, and costs of a few of the materials are also shown.

4. Conclusions relative to surface treatment:

In summary, it can be concluded from the freeze-thaw tests that:

(a) Air entrainment provides better freeze-thaw protection than any of the coatings studied;

(b) Mixtures of linseed oil and kerosene and of tung oil and kerosene provided considerable protection against freeze-thaw scaling damage if applied without delay after curing and drying; little benefits result from applications after the concrete has begun to scale;

(c) Neither Jennite J-20 and J-16, nor asphalt MC-0 primer with sand filled Ampet AC-5 proved effective in preventing freeze-thaw scaling;

TABLE 9. EFFECT OF TEMPERATURE CONDITION AT TIME OF TREATMENT WITH EPOXEAL

Surface Treatment (Table 4)	Temperature of Specimen at Time of Treatment	Cycles at Moderate Scaling	Cycles at Severe Scaling
9-A	Rising from 73°F to 100°F	40	66
9-B	Constant at 100°F	23	61
9-C	Decreasing from 100°F to 73°F	12	61

TABLE 10. QUANTITIES AND COSTS

Coating Code Table 4	Coating Material	Coverage (sq yd/gal)	Cost in Dollars per sq yd		
			Application	Material	Total
1	Linseed oil + kerosene (1 coat)	28	0.0156(a)	0.0121	0.0277
2-a	Linseed oil + kerosene (2 coats)	1st coat 37 2nd coat 56	0.0156 0.0156	0.0090 0.0061	
			.0312	.0151	0.0463
3	Linseed oil + kerosene (3 coats)	1st coat 46 2nd coat 56 3rd coat 73	0.0156 0.0156 0.0156	0.0070 0.0061 0.0046	
			.0468	.0181	0.0649
4	Linseed oil, hot	28	(no record)	.0121	------
5	Jennite + Primer	1/8 in. thick	------	------	------
6	Asphalt (AC-5 with primer MC-0)	1/16 in. thick	See below		(b)
7	Thompson's Water Seal (2 coats)	1st coat 29 2nd coat 18	0.0156(c) 0.0156(c)	unknown	
8-a	Tung oil + kerosene (2 coats)	1st coat 37 2nd coat 56	0.0156(c) 0.0156(c)	unknown	
9	EpoXeal (2 coats)	1st coat 13 2nd coat 75			0.50(d)

(a) THD District 18 records for applying Linseed Oil Anti-Spall Compound (a mixture of Boiled Linseed Oil and Mineral Spirits) with THD maintenance forces.

0.022 gals/sq yd
$0.0154/sq yd for material
$0.0156/sq yd for labor and equipment

Since the Anti-Spall Compound and the mixture of linseed oil and kerosene appear to be about the same consistency, it is assumed that application costs for the two materials will be essentially the same.

TABLE 10. (Continued)

(b) No costs available but it is estimated that maintenance forces can apply the primer, coating, and stone finish for a total cost of about \$0.20 to \$0.25 per sq yd.

(c) This number represents the cost to THD maintenance forces for labor and equipment in applying Linseed Anti-Spall Compound. It is assumed here that the cost of applying coatings 7 and 8 will be essentially the same as for that compound.

(d) This figure was reported to one of the authors by letter, dated June 19, 1968, from Mr. R. Lyle Brace, Protective Products Corporation.

(d) Coatings used in the tests had little influence on freeze-thaw deterioration in cracks;

(e) Concrete overlaid with 1 1/2 in. thick asphaltic concrete made up of a seal coat, rubberized asphalt tack coat, and hot mix surfacing showed no deterioration after 59 freeze-thaw cycles;

(f) Coatings should be applied to dry surfaces during hot weather;

(g) Skid resistance is decreased by use of the four coatings tested;

(h) Costs of the linseed oil and kerosene mixture were the lowest of those determined in the study.

Implementation: Treatment

All reinforced concrete bridge decks should be made of high quality, air-entrained concrete. Air entrainment is the best way known today to combat freeze-thaw scaling of bridge decks.

Once the deck is in place freeze-thaw durability can be enhanced by 2 applications of approximately 1 gallon of a mixture of 50 percent kerosene and 50 percent boiled linseed oil per 40 square ft of bridge deck. These applications should be made to clean surfaces before the bridge is opened to traffic. One application should be made annually for at least two years after the initial application.

Old decks are influenced very little in freeze-thaw durability by the kerosene-linseed oil mixture, especially after scaling has begun. The mixture is not effective in preventing scaling at edges of cracks.

SECTION 3. PATCHING AND OVERLAYING
by Howard L. Furr

Introduction

Deterioration of bridge decks sometimes becomes so severe that major repairs are necessary. One method of repairing such surfaces is to patch holes and resurface with portland cement concrete or resinous overlays. The deck surface must be clean and sound for good bond between the deck and the overlay. All loose material such as surface scales and spalls must be removed. Chipping tools are generally used to remove the deeper unsound material, and snadblasting or surface scarifiers are used to remove deteriorated and oil soaked surface material.

This part of the report deals with tests made to determine effective ways of repairing the decks. It considers surface preparation of the old decks, bonding between the new and old concrete, and field installations.

The following tests were conducted in this phase of the research:

1. Bond tests between the new and old concretes,
2. Tests on the stiffening effects of overlays, and
3. Field installation tests.

Details of all materials, preparations, test procedures, and results of tests are given in Research Reports 130-1, 130-5, 130-6, and 130-8. (1, 5, 6, 8). This section summarizes the information previously reported in those research reports.

Base Specimens

Base specimens were made to represent existing concrete bridge decks which were to receive overlays. These specimens were 7 in. plain concrete cubes, 2 in. thick plain concrete slabs, 10 in. x 16 in. and 10 in. x 10 in. respectively and reinforced concrete beams 7 in. wide x 5 in. thick x 96 in. long. The 2 in. slabs were sawed into 3 in. wide strips after the overlays were applied.

The concrete used in the 2 in. thick slabs contained 5 1/2 percent entrained air. Other base specimens were without entrained air. The concrete mix used 5.50 sacks of type III portland cement per cubic yard of concrete with 6 gallons of water per sack of cement. The nominal compressive strength of the concrete was 4000 psi at 28 day age.

Surfaces were finished with a wood screed, and the specimens were given a 7 day moist cure followed by 3 to 4 weeks of storage in the laboratory. No curing compound was used on any surface.

Overlay and Bonding Materials

Materials used for the overlays consisted of:

1. Polyester concrete (CDC-100),
2. Epoxy mortar,
3. Latex modified portland cement concrete,
4. Portland cement concrete,
5. Chem Comp (shrinkage compensated) cement concrete,
6. Portland cement mortar reinforced with steel wires 3/4 in. long, (fibrous reinforced concrete), and

7. Portland cement concrete reinforced with 3 in. x 3 in. mesh 4-gage welded wire.

Overlays used in direct shear tests on the 7 in. cubes were 2 in. thick; beam overlays were 1 1/2 in. or 2 in., and slab overlays were 1 in. thick.

Bonding agents were used prior to placement of the overlay in some cases, and in others none was used. The CDC-100 material, a proprietary product, was bonded with a polyester. The latex modified concrete provided its own bond when the coarse material was broomed out of a thin layer of mix placed ahead of the main overlay. The epoxy mortar overlay provided its own bond with no special bonding course. The portland cement concrete overlays were bonded with either cement-sand grout or epoxy resin. Details of the mix and placement are given in references 1 and 5.

Surface Preparation

Surfaces of the base specimens were prepared by either dry rubbing with a stiff wire brush, sandblasting, or chipping with a light chisel. The wire brush removed only loose surface powder. The sandblasting and chipping removed mortar to the level of exposing coarse aggregate.

Some of the surfaces were wet when the overlay was applied, and some of them were dry. The surfaces of one set of specimens were soaked with motor oil prior to preparing the surface for overlay.

Test Summaries and Results

1. Bond between the overlay and the base concrete:

Direct shear tests were made by shearing the overlay from the 7 in. cubes. Table 11 gives results of such tests made to determine the best grout mix. It shows that good bond can be obtained by a neat cement paste or by cement-sand grout using 1 part of cement to 3/4 to 1 1/2 parts of sand by weight. Grout used for bonding in other specimens was made of 1 part of cement to 3/4 parts of sand.

Table 12 shows that the bond on the oil soaked specimens was reduced even after chipping and sandblasting. The surface which was dry when the overlay was applied gave considerably better bond than the overlay placed on a wet surface, especially when epoxy was used for bonding. The sandblasted dry surface, SS3 in Table 11, produced better bond than did the chipped dry surface, SS2 in Table 12. The latter table shows that good bond was obtained with the steel fiber and the wire mesh concrete S4.

The latest tests gave further information on bond to sandblasted surfaces, and the results are shown in Table 13. The grouted surfaces provided better overall bond strength than any of the other bonding methods of that test series.

The overall evaluation of the shear bond tests shows that a clean surface is required for bonding and that grout provided an excellent bonding material.

TABLE 11. INFLUENCE OF SURFACE PREPARATION AND GROUT MIX ON BOND STRESS

Series (Description)	Cubes Designation	Surface Preparation	Grout Mix Cement: Sand	Overlay Concrete	Bonding* Stress (psi shear)
S3	A	Brushed	1:1 1/2	7(sk)	427
(Dry surface before placing overlay)	B	Brushed	1:1	7(sk)	424
	C	Brushed	1:3/4	7(sk)	537
	D	Brushed	1:0	7(sk)	506
SS3	A	Sand Blast	1:1 1/2	7(sk)	408
(Dry surface before placing overlay)	B	Sand Blast	1:1	7(sk)	428
	C	Sand Blast	1:3/4	7(sk)	535
	D	Sand Blast	1:0	7(sk)	367

*Average of two cubes.

TABLE 12. BOND STRESS FROM DIRECT SHEAR TEST ON 7-INCH CUBES

Series (Description)	Cubes Designation	Surface Preparation	Bonding Agent	Overlay Concrete	Bond* Stress (psi shear)
S1 (Dry surface before placing overlay)	A	Chipped	Epoxy	7sk	531
	B	Brushed	Epoxy	7sk	612
	C	Chipped	Grout**	7sk	458
	D	Brushed	Grout	7sk	458
S2 (Surface soaked with water for 24 hrs. Overlay was placed on wet surface)	A	Chipped	Epoxy	7sk	94
	B	Brushed	Epoxy	7sk	77
	C	Chipped	Grout	7sk	405
	D	Brushed	Grout	7sk	410
S3 (See Table 11)					
S4 (Dry surface before placing overlay)	A	Chipped	Grout	7 sk, 1% Stl. Fiber	325
	B	Brushed	Grout	7 sk, 1% Stl. Fiber	455
	C	Brushed	Grout	7 sk, 3" sq. wire mesh, 10 gage wire	544
SS1 (Surface soaked with motor oil for 3 days)	A	Chipped	Epoxy	7sk	452
	B	Chipped	Grout	7sk	343
	C	Brushed	Grout	7sk	55
	D	Sand Blast	Grout	7sk	327
SS2 (Dry surface before placing overlay)	A	Chipped	None	7sk	129
	B	Brushed	None	7sk	187
SS3 (See Table 11)					

**All grout used 1 part cement to 3/4 parts sand.

TABLE 13. SHEAR STRENGTH OF OVERLAY BONDS

Type of Overlay	Average Bond Shearing Stress (psi) Base Surface Preparation	No Bonding Agent	Grout Bonding Agent	Epoxy Bonding Agent	Other	Remarks
Plain Concrete	Sandblast		420	462*		Air entrained overlay
Wire Mesh Concrete	"		555	362		Air entrained overlay
Steel Fiber Concrete	"		668	565		Air entrained overlay
Chem Comp Concrete	"		521	402		Air entrained overlay
Latex Modified Concrete	"				382	Latex modified grout and overlay
	(Average		541	448	382)	
Plain Concrete	Sandblast		393			Not air entrained overlay
Plain Concrete	"		388			Air entrained overlay
Latex Modified Concrete	"				458	Cement "B", latex modified grout and overlay
Latex Modified Concrete	"				382	Cement "B", latex modified grout and overlay
Epoxy	"	214				Guardkote 250 overlay
Polyester	'	344				CDC-100 (polyester) overlay

NOTE: The values shown represent the averages of three specimens for each overlay, A through D, and six specimens for each of E through I; and four specimens for J.

Freeze-thaw tests, reported in reference 5, showed that grout and epoxy bonded overlays remained fast after 300 laboratory freeze-thaw cycles. The latex material overlay came unbonded during the test.

2. Stiffening effect of overlays:

The 96 in. long beams were set on simple end supports and were loaded statically at midspan both before and after they were overlaid. The midspan deflection was taken each time. They were then subjected to 2 million load repetitions after which the load-deflection test was repeated. The beams were deliberately cracked during the initial static loading before the overlay was placed. This was to simulate a deck that had cracked in service before an overlay was applied. During the cyclic loading, which applied upward load as well as downward load, the overlay was cracked in each case. All of the cracks occurred at or near midspan.

The overlays added considerable stiffness to the beams, depending on the thickness of the overlay. The stiffness was maintained at essentially the same value through the 2 million load cycles, and no unbonding of overlay was detected by inspection after the repeated load test.

3. Field installations:

During the course of the study, two field overlay installations were made. Details of those installations have been reported (8), and only a summary is given here.

The two experimental portland cement concrete overlays were placed by the Texas Highway Department (THD) for bridge deck repair and resurfacing. One installation was in THD District 7 on Route 208 in Coke County, across the Colorado River at Robert Lee, Texas. The other was in THD District 2 on Spur 179 in Erath County across the Bosque River at Stephenville, Texas. The former was 1 5/8 in. thick overlay; the latter was 2 in. thick.

Both bridges were damaged lightly by scaling and severely by spalling and cracking. Each had developed numerous transverse cracks during some 22 years of service and there were some areas of serious checkerboard cracking between beams. The Colorado River bridge carried occasional heavy trucks with oil field equipment and with grain. The Bosque River bridge traffic was largely light traffic, with farm supplies and produce providing the heavier loads.

Delamination was removed from both bridges with an air hammer. The Colorado River bridge deck was then cleaned by sandblasting whereas the Bosque River bridge was scarified with the McDonald Scabbler. Grout was brushed into the clean deck, and overlay was vibrated into place over the fresh grout. A vibrating screed was used for leveling and compacting the material on the Colorado River bridge, and internal vibrators, followed by a pavement finishing machine, were used on the Bosque River bridge. The Colorado River bridge overlay was placed in October 1969 by THD District 7 maintenance forces. The Bosque River bridge overlay was placed in November 1971 by THD District 2 maintenance forces supplemented by contract forces for placement and finish of the overlay.

Both overlays have developed reflective cracks, and inspections have revealed several small areas of unbonding. The unbonded areas are generally, but not in all cases, over or very near cracked areas. Other areas of unbonding are attributed to overly thin grout from excessive mixing water or from water spray blown on the old deck during placement of the overlay. A microscopic examination of slices across the interface of overlay and base concrete of cores from the bridges reveal relatively poor grouting on the Colorado River bridge, but good to excellent grouting on the Bosque River bridge. The latter showed cracked portions of aggregate and of mortar, caused by impacting blows of the scarifier, that had not been dislodged in the cleaning process. Those, no doubt, cause weaknesses in bond.

Both overlays have been in continuous service since installation, and they are in good condition. The cost of the 13,440 sq ft overlay and patching on the Colorado River bridge was $0.92 per sq ft, and that of the 16,000 sq ft Bosque River bridge was $0.893 per sq ft for patching and overlay. An additional $5,800 ($0.363 per sq ft of overlay surface) was required to provide full depth deck replacement repairs at two locations in the latter bridge.

Discussion

All of the laboratory tests show that excellent bond strength can be developed between base concrete and 1 1/2 in. to 2 in. overlay concrete. Direct shear strength of laboratory specimens listed in Table 13 are all in excess of six times the theoretical static service requirements estimated in Research Report 130-5 (5). The repeated load tests and freeze-thaw tests gave further support to the good bonding properties of cement grout and epoxy resin.

The requirements for good strength and for durability are:

1. A concrete base of high quality concrete.
2. A clean concrete surface prepared by sandblasting, scarifying or planing, and sweeping and washing.
3. Cement grout or epoxy resin thoroughly scrubbed into the surface.
4. A high grade structural concrete overlay with entrained air, low slump, and thorough compaction.
5. Prevention of rapid surface drying prior to curing.
6. Curing to provide the desired strength and durability prior to opening the installations to traffic.

Attention must be paid to the influence of curing on the durability and the strength of the overlay. Adequate strength for an overlay can be attained, under good conditions, with a relatively short curing period--possibly from 3 to 4 days. Durability is enhanced by longer curing periods, but the measure for that increase is not as well established as it is for strength. It is essential that curing be adequate to insure a concrete of the quality that is required of the installation. The overlay replaces structural concrete of the deck only where the old material is removed, such as in spalled areas. The field overlays reported here filled many spalled areas, but for the most part the concrete was added to the full depth slab. The overlay in such installations serves both as a sturctural material and as a material to smooth the rough surface. It is most likely that the stresses in the overlay, due to traffic, are less than those in the original deck under the same loads. The less severe weather conditions encountered in some areas are less demanding of durable materials than the conditions of heavy freeze-thaw activity found in other areas.

The established construction specifications (21) insure that strong and durable materials are provided in a structure. The requirements of those specifications should be followed. Each situation that requires that the installation be opened to traffic prior to the termination of normal curing must be taken into account in the design of the overlay. In such cases mix design and curing procedures should be approved by the Highway Department before the installation begins.

Conclusions Relating to Patching and Overlaying

From this part of the study the following conclusions have been reached:

1. Patches and overlays of portland cement concrete, resinous concretes, and latex modified concretes, and wire fiber reinforced concretes were shown to be feasible in the laboratory.
2. Field installations of portland cement concrete patches and overlays only were made. Those installations showed that the overlays can be applied by highway maintenance personnel and by contract personnel.

3. The existing concrete to which the overlay or patch is to be bonded should be sound concrete of good quality.
4. Overlays of portland cement concrete, steel fiber reinforced portland cement concrete, epoxy resin concrete, and latex modified cement concrete can be bonded to clean concrete.
5. Bond strengths of approximately 100 psi to approximately 600 psi in shear can be developed between the overlay and new, clean concrete. Values averaged generally between 300 and 500 psi for portland cement concrete bonded with grout, and only slightly lower for overlays bonded with epoxy resin.
6. The old concrete surface must be thoroughly cleaned prior to application of bonding agents or overlays. This should be done by sandblasting, scarifying or both followed by brushing and washing. All dust must be removed.
7. Sand-cement grout and epoxy resin make excellent bonding agents for overlays and patches.
8. Overlay concrete must be air-entrained to make it durable against freeze-thaw activity.
9. No delamination of 1 1/2 to 2 in. overlays was experienced from freeze-thaw tests nor from 2 million applications of load to overlaid beams.
10. Two field patching and overlay installations, one in Coke County, the other in Erath County cost $0.92 and $1.256 respectively per square foot of bridge deck surface. The former was a 1 5/8 in. thick overlay with numerous deck

patches, covering 13,440 square feet of deck. The latter was a 2 in. thick overlay with numerous deck patches, covering 16,000 square feet of deck.

Implementation: Patching and Overlaying

An evaluation of a deteriorated deck must take into account the costs in dollars and down time as well as the extent of damage and quality of the material in the deck. An extensive program of deck repair can be made at far less initial cost than removal and replacement, but for a sound repair job the deck to be repaired should have good material left after the deterioration is removed. A patch or overlay should be applied only to clean, sound concrete. All personnel involved in the installation should be thoroughly familiar with the requirements and procedures relating to his responsibility before the job is begun. Improper preparation, installation, or curing can cause trouble later in the service life of the deck.

The recommended minimum requirements for these concrete overlays and patches given below will produce good repairs if the material in the deck is good after all deterioration is removed.

RECOMMENDED MINIMUM REQUIREMENTS FOR THIN CONCRETE OVERLAYS AND PATCHES

NOTE: The recommendations which follow are considered to be minimum requirements for the usual problems encountered in installing bonded thin, about 2 in. thick, concrete overlays on concrete bridge decks. Additional requirements to fit the particular situation not covered below should be developed as needed.

LABOR: Personnel are required to remove unsound concrete, to uncover rusty steel, to remove rust from steel exposed before and after preparation, scarify the deck, clean the deck, apply bonding material, mix, place and cure patch and overlay concrete, and to take care of special provisions in specifications.

MATERIALS:

Grout: A mixture of one part by weight of portland cement with 3/4 parts of saturated surface dry sand passing No. 8 sieve and 1/2 (approx.) parts water mixed to a thick creamy consistency.

Epoxy: Only that which has a proven record of good service should be used for bonding agent. The recommendations of the manufacturer should be followed in applying it.

Overlay Concrete: (Weights per cubic yard of Concrete)

The mix will depend on the type of aggregates. Sound materials are required, and the water to cement ratio should be kept low. The mix given below proved to work well in the installation described in this report.

Gravel: 1850 lb saturated surface dry.

Sand: 1140 lb saturated surface dry.

Cement: 659 lb, Type III

Water: 285 lb (Approx.) to produce 2 in. slump.

Entrained Air: 6 per cent.

Aggregate Gradation:

Sand: Meet THD gradation for concrete sand.

Gravel:

Size	Percent retained
3/4	0
1/2	15
3/8	25
No. 4	58
No. 8	2

LOCATING UNSOUND CONCRETE: All unsound concrete should be located and marked for removal. Trained personnel with the TTI delamination detector, sounding hammer, or chain drag should be used to locate the material to be removed.

REMOVAL OF UNSOUND CONCRETE: All unsound concrete must be removed. Removal should extend into sound concrete to insure that no poor concrete remains in place. Air hammers, chipping hammers, and saws may be used for removal.

CLEANING STEEL: All rusty steel should be uncovered and the rust should be removed by sand blasting. If the steel is loose (not bonded), it should be exposed all the way around so that it will be encased in new concrete when the patch or overlay is placed.

CLEANING THE DECK SURFACE: The deck must be thoroughly cleaned to remove oil, grease, and other contaminants. Loose and unsound surface mortar and aggregates, too, must be removed. Sandblasting is the preferred method of doing this. Scarifying machines such as the Tennant machine and the McDonald Scabbler should be used only when it is necessary to remove more than about 1/16 inch of surface material.

The deck must be thoroughly cleaned of dust and debris following sandblasting and scarifying. Particular attention must be given to removal of dust to be sure that no film of either wet or dry loose fine material remains.

APPLYING BONDING AGENT: Either epoxy resin or cement grout may be used to bond the new concrete to the old concrete. The surface must be clean and free of dust and debris for either of the bonds. Epoxy bond requires a dry concrete base unless special resins are used. The old surface should be saturated surface dry when grout is used. Traffic must be kept off the clean surface. Grouted surfaces should not be tracked.

Epoxy: Apply epoxy ahead of the fresh concrete according to recommendations of the manufacturer of the material.

Grout: Thoroughly brush the grout into surfaces to be patched or overlaid. A stiff broom should be used, and the average thickness of the grout should be approximately 1/8 in. when it is worked in. Care must be taken to see that it is worked in thoroughly; that it is not too thick (corners and depressions should be carefully treated); and that the grout is well mixed. The concrete should be placed on the grout after it has dried to a damp condition, not a dry condition.

CONCRETE PLACEMENT: Place concrete for patches and overlay on only properly prepared bases over untracked epoxy or grout bonding agent. The concrete must be worked into corners, depressions and around exposed steel. Internal vibrators should be used, and they may be supplemented by surface vibrators. If necessary for stiffer mixes, compaction with a ram in addition to internal vibration might be necessary.

Overlay concrete may be finished with a pavement finisher or by vibrating screed. Texture should be provided as called for by THD.

Patch concrete must be finished rough in order to receive the overlay and to bond well. No curing compound is to be used on patch concrete which will be followed by an overlay.

CURING: Wet mat should be used for curing. Patches should not be overlaid before the patch concrete has set up. If epoxy is to be used for a bonding agent, the patch concrete must be thoroughly cured and dried unless a non-water sensitive epoxy is used. Grout may be applied as soon as the surface is damp-dry.

The overlay should be cured as specified in THD standard specifications.

SECTION 4. CRACK AND FRACTURE REPAIRS WITH EPOXY RESIN

by Leonard L. Ingram

Introduction

The excellent adhesive properties of epoxy resins have made them very attractive in the family of highway construction materials. Their potential value is further enhanced by favorable durability characteristics. They have developed good performance records in the 20 or so years in which they have been used in highway applications. Because of their attractive properties and good records of service, the volumes used in highway applications have grown tremendously during the relatively few years that have elapsed since their introduction for those applications.

There have been improvements in the epoxy resin products through formulation designed to meet the particular need at hand. Application techniques, too, have improved through knowledge gained from performance records and research. The material is used with confidence, but it is relatively expensive.

Most of the first applications of epoxy resins to concrete structures and pavements were for maintenance of deteriorated concrete (22-26). Gaul and Smith (27) reported one of the first uses of epoxy in the structural repair of a concrete seat beam, a cracked craneway and cracked floor slabs. Since that time, many reports (28-30) have been published concerning methods and techniques for structural repair of damaged concrete. A manual developed by Kreigh (31) for the U. S. Army Corps of Engineers contains a comprehensive treatment of methods and techniques for epoxy injection repairs.

The purpose of the work reported here was to study methods and techniques of epoxy injection repair and to select or develop methods of epoxy injection repair that can be used in the field by highway maintenance personnel. The description and results of laboratory studies and field repairs are given in the sections that follow.

Materials

Portland cement concrete test specimens were made using Type III cement and natural sand and gravel from pits near Hearne, Texas. The maximum size aggregate was 3/4 in., and the mix design was 1990 lb of gravel, 1170 lb of sand, 470 lb of cement and 243 lb of water to produce 2 to 3 in. of slump. The specified air content was 5 to 6 percent. The nominal ultimate strength at 7 days was 4000 psi.

Components for epoxy bonding systems were obtained from 2 suppliers for use on this project. The bonding systems were 100 percent solids systems and contained no volatiles. No effort was made to test many different brands of epoxy bonding systems. Instead, emphasis was placed on developing methods and techniques of epoxy injection repairs.

Test Specimens and Tests

The specimens tested and the procedure for their testing is given in each of the test series that follow.

Series A:

The objective of this test series was to determine the effectiveness of an epoxy resin to restore tensile strength to cylindrical concrete specimens.

For this series, 24 standard 6 x 12 in. cylinders were made and moist-cured 7 days. Compressive strength and modulus of elasticity tests were made at 7 days and at approximately 1 month. Three cylinders were moist-cured 7 days; 3 were moist-cured 7 days followed by air-drying at 73°F, 50% relative humidity for 25 days, and 3 were moist-cured for approximately 1 month. Eight of the cylinders remained in the moist room until tested while 13 were transferred to a room of 73°F with 50% relative humidity after 7 days of being moist-cured for 25 days of air-drying.

The splitting tensile test, ASTM C-496, was selected as the test method to compare 2 methods of applying an epoxy resin. One method required that adjoining faces be coated with the epoxy and the pieces joined together while the other method utilized pressure to force the epoxy into a crack in the concrete. The air-dried cylinders were tested by both methods, 5 cylinders for each method. The 5 moist-cured cylinders were tested using pressure injection only.

Series B:

The objective of this test series was to determine the effectiveness of an epoxy resin in the repair of cracks subjected to repeated flexural load cycles. For this series 5 doubly reinforced laboratory beams, 7 in. wide, 5 in. deep and 8 ft, 6 in. long were made.

They were moist-cured 7 days and dry-cured at 75°F for a minimum period of 1 month before testing began.

Tests in this series consisted of the following: (See Fig. 6)

1. Mid-span load versus mid-span deflection using static load. Upon completion of this loading, the beam was loaded to produce several flexural cracks in the vicinity of mid-span.

2. The load versus deflection test of step 1 was repeated using the cracked beam. The cracks were then filled with epoxy using pressure for injection.

3. The load-deflection test of step 1 was performed on the repaired beam.

4. Two million load cycles were applied to the repaired beam. The loader used rotating eccentric weights. The load-deflection curve developed under static loading was used as the calibration for relating load and deflection. Deflection, and hence load, was controlled by regulating the cyclic rate of rotation of the eccentric weights. At the beginning of the dynamic load cycling, the control deflection was taken from the load-deflection plot after the epoxy injection. After each 1/2 million cycles, another static load-deflection test was made and the control deflection taken from that plot for the next 1/2 million load cycles.

Series C:

The objective of this test series was to determine the effectiveness of an epoxy resin in restoring flexural strength to broken or cracked laboratory beams. The same 2 methods of epoxy application as used in Series A were used here. For this test series 15 concrete test beams, 6 x 6 x 36 in. were fabricated. All the beams were moist-cured 7 days. Then 10 of the beams were moved to a room of 73°F to dry-cure for approximately 2 months, and the other 5 beams remained in the

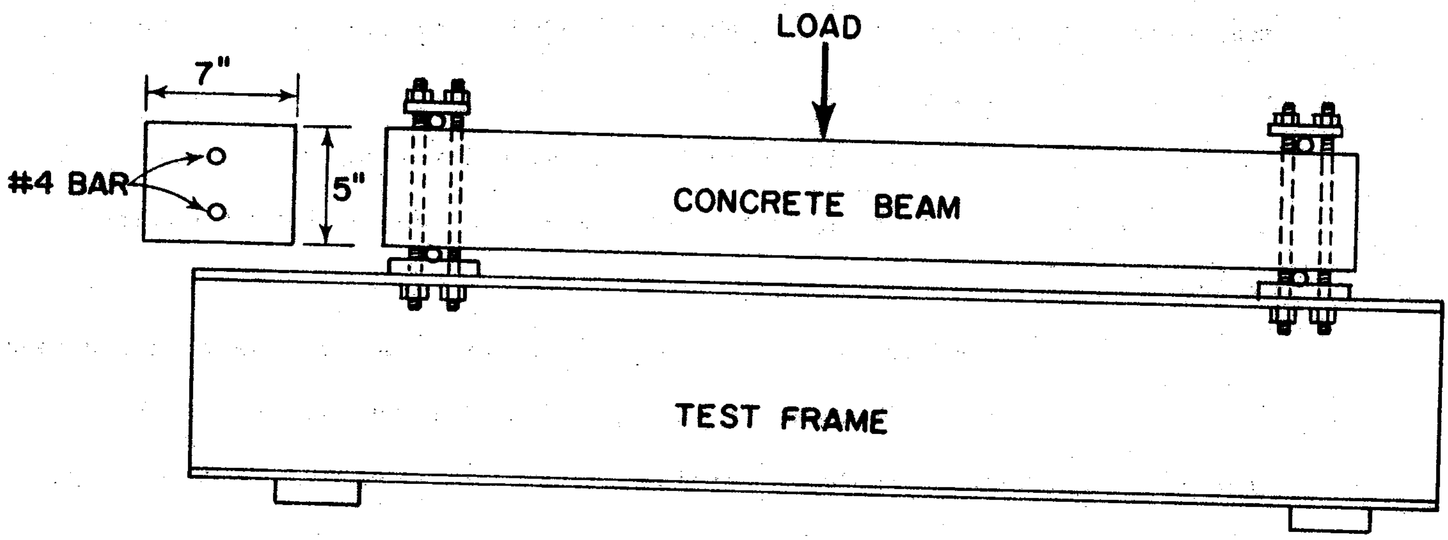


Figure 6. Test Frame.

room for moist-curing. Five of the air-dried beams were broken by third point loading, ASTM C-78. Each broken face was then coated with an epoxy, and the pieces were fitted back together and held in place until the epoxy had set. The other 5 air-dried beams were cracked by applying a concentrated load at mid-span. The outer perimeter of the crack was sealed leaving an entry port at midheight and an air vent in the top of the crack. An epoxy was pressure-injected into the crack until it flowed freely at the air vent. The 5 beams were taken from the moist-cure room. They were cracked and were pressure-injected by the same procedure as just described for the 5 air-dried beams. Free water in the crack was displaced by the epoxy in these wet beams. Two days were allowed for epoxy cure. Then the beams were broken or cracked as done initially.

Series D:

The objective of this test series was to determine the effect of freeze-thaw cycling on an epoxy bond of specimens broken and bonded back together. Twenty prisms, 3 x 3 x 16 in., were cast for this test series. They were moist-cured 7 days and air-cured for approximately 2 months at 73°F. Ten of these prisms were broken by applying a concentrated load at midspan of the simple beam set up with a 14 in. span length. Each broken face was coated with a epoxy, and the pieces were fitted together and held in place until the epoxy had set. The 20 prisms, 10 broken and repaired prisms and 10 unbroken prisms, were subjected to 50 freeze-thaw cycles. The prisms were moved on a hand cart from the freezing room, 0°F, to the thawing room, 40°F, twice each day. One cycle was completed each 12 hours, 6 hours for freezing and 6 hours for thawing. After the freeze-thaw cycling, the 20 prisms were broken as described above.

Series E:

The tests described in this series were initiated to study the flow characteristics of pressure-injected epoxy. Cracks for injection were simulated by separating 2 pieces of glass, 2 ft x 2 ft, with different thicknesses of shim brass. Strips of shim brass were placed along the perimeter of the glass on 3 sides with an opening at the center of one side through which the epoxy was introduced. The 4th side was left open for the escape of the air displaced by the epoxy. The formed crack was sealed along the 3 sides containing the shim brass with the exception of the opening through which the epoxy was forced. Different simulated crack widths and different injection pressures were studied.

Series F:

In this test series 3 concrete blocks, 20 in. square and 5 in. thick were prepared to test the ability of an epoxy to bond a cracked section in an environment of salt water covered with an oil slick. This was an effort to simulate the repair of a crack in sea water. The blocks were cracked through and placed in a vat of salt water solution with an oil film on the surface of the water. The blocks, half under water and half out of water, were left in the vat 2 to 3 weeks. Once each day they were swayed back and forth to generate small waves to create a splash zone on the blocks. After approximately 2 weeks, the application of epoxy to 1 block began. Entry ports were located at 5 in. spacings along the vertical crack from the top to the bottom of the crack. The ports, 1/4 in. ID tubing 1 in. long, were fitted into holes drilled into the crack and then an epoxy, formulated to cure under water, was applied to the exterior of the crack to seal it. When the sealing epoxy had set, a low viscosity epoxy that sets in the presence of moisture was injected under pressure into the crack beginning at the bottom port. As the epoxy entered and filled the crack, water was displaced. When the epoxy flowed freely at the port just above the entry port, injection was stopped; the entry port was plugged, and the injection line was moved to the next port above the newly plugged port. The injection was begun again, and sequence was repeated progressively up the crack until all the crack was filled.

A second block was prepared in the same manner as the first, i.e., entry ports spaced along the crack and the crack sealed. In an effort to clean the interior of the crack, toluene was flushed through the crack 3 times. The top and bottom ports were open, and the interior ports were plugged temporarily. The solvent was forced through the bottom port, and it flowed out the top port. Care was taken not to allow the oily film to enter any port after the flushing. Water was allowed to enter again through the bottom port after flushing. The ports above the water line were opened to allow the remains of the solvent to escape. After one day, the crack was pressure-injected in the same manner as the first block described earlier.

Motor oil was poured into the crack in the 3rd specimen and allowed to penetrate by gravity. The ports were then put in place; the crack was sealed and later flushed with toluene as described for the 2nd block above. After 1 day epoxy was injected under pressure into the crack as discribed for the previous blocks.

Test Results

Series A:

The standard 6 x 12 in. cylinders of this test were either split apart or cracked as outlined previously. In the initial testing 4 of the cylinders shattered to pieces before the load could be relieved after the initial crack formed. Consequently not all the cylinders were repaired and tested again. Out of the total of 15 cylinders orginally cast, 11 were successfully tested. The results of the tests are in Table 14. Dry cylinders that were pressure-injected had a minimum of 70% strength restoration, the highest of the series. The epoxy used in this test did not bond well to damp surfaces and the cylinders that were kept wet before and after injection yielded poor strength restoration. Some of the cylinders that were split apart provided poor surface for bonding and therefore yielded poor results. One of the better cylinders had about 70% strength restored, and one had a higher strength after injection than before. It is reasoned that some strength capabitilies will be lost due to small cracks that cannot be filled with epoxy during the pressure injection.

TABLE 14. TENSILE STRENGTH OF 6 x 12 CYLINDERS BEFORE AND AFTER REPAIR

Cylinder No.	Remarks	Column 1 Tensile Stress Before Repair, psi	Column 2 Tensile Stress After Repair, psi	(Column 2) ÷ (Column 1)
1	Dry cylinders	540	186	0.34
2	split apart	526	106	.20
3	and bonded back	530	257	.48
4	together	535	371	.69
5	Dry cylinders,	540	509	.94
6	cracked and	544	610	1.12
7	pressure in-	588	406	.69
8	jected	561	416	.74
9	Wet cylinders,	548	175	.32
10	cracked and	530	141	.26
11	pressure in-	557	310	.55
	jected			

Specimens 1, 2, and 3 were broken into several pieces in initial tests and were difficult to fit together. Some pieces were lost during those initial tests.

Series B:

Approximately 8 to 10 cracks were formed by bending the test beam with a mid-span load. Of these cracks there were usually only 3, generally 1 at mid-span and 1 on either side of mid-span, that could be filled. The remainder of the cracks closed up when the load was removed. Beam stiffness ratios, Table 15, give some indication of the effect of the cracking and crack repair on the beam stiffness. Typical load versus deflection plots are shown in Figure 7. Cracking reduced the stiffness of the beams about 80 percent. The injection of the epoxy increased the stiffness of the cracked beam 150 to 200 percent. The repaired cracks did not open up during the test but a number of the cracks that were too narrow for injection did open up. Those cracks which were not repaired were responsible for the lower stiffness of the repaired beams as compared with the uncracked beams.

Series C:

Six in. square beams 36 in. long were used in this series. Faces of beams broken in the beam breaker were coated and fitted back together and held in place until the epoxy cured. Modulus of rupture values, ASTM (C-78), are given in Table 16 for initial breaking and for breaking after the pieces were bonded back together. The average modulus of rupture for the 10 breaks after epoxy bonding was about 80 percent of the average of the initial 10 breaks. In each case, the break after epoxy bonding was through the concrete adjacent to the bond line. This indicates that the epoxy was stronger at the bond line than the adjacent

TABLE 15. RATIO OF MIDSPAN LOAD TO MIDSPAN DEFLECTION, P/Δ

Beam No.	(1)* Before Cracking	(2) After Cracking	(3) After Injection	(4) After 2 million Load cycles	(2) ÷ (1)	(3) ÷ (2)	(4) ÷ (3)
1	12,000	2,300	3,950	3,240	0.19	1.7	0.82
2	11,000	2,900	3,950	3,740	0.26	1.4	1.94
3	16,300	2,390	4,390	3,660	0.15	1.8	0.83
4	11,500	2,290	4,240	3,740	0.20	1.8	0.88
5	12,000	2,060	4,800	3,320	0.17	2.3	0.69

*The sequence of steps in the test are numerically the same as the column headings

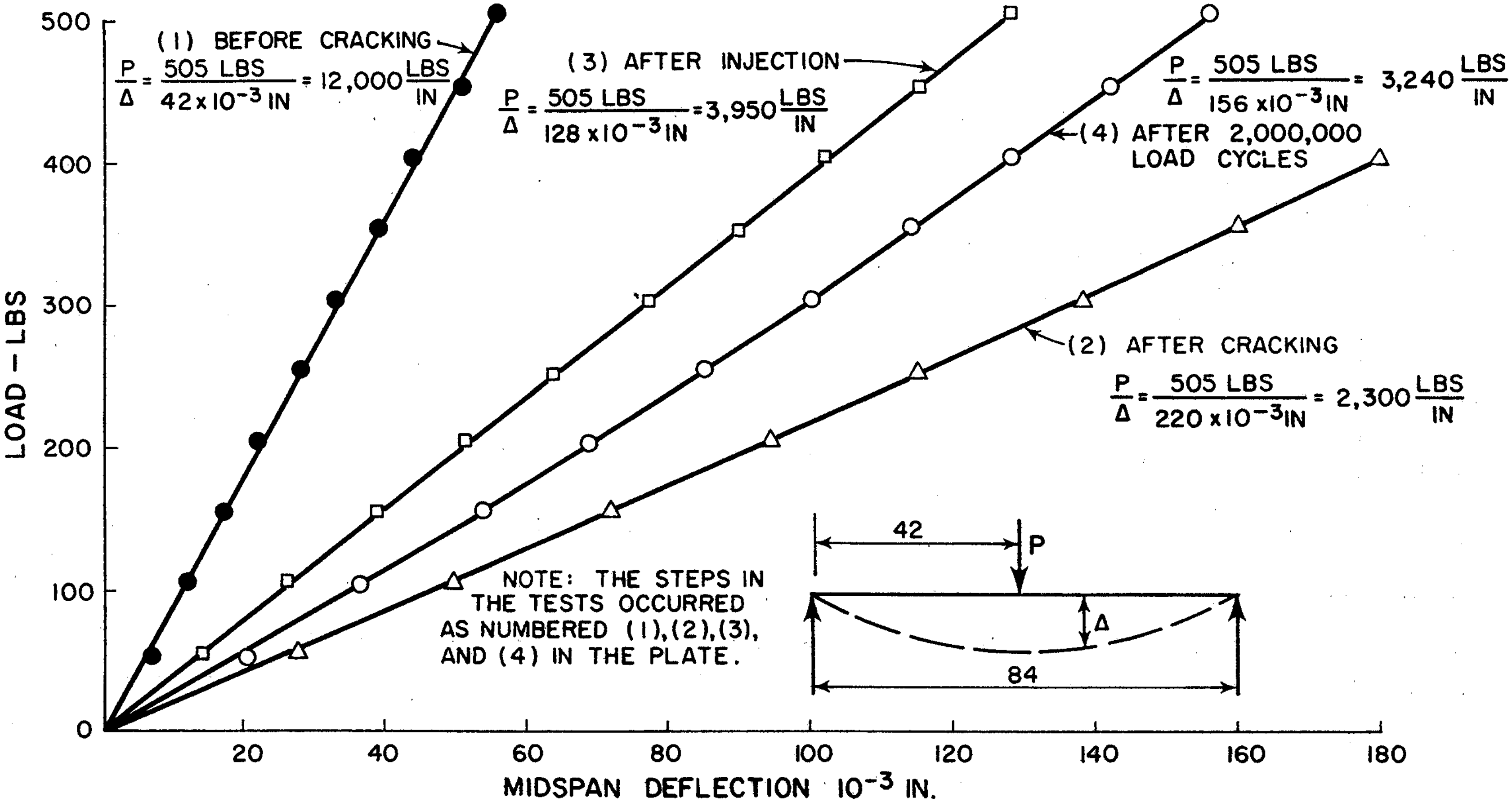


Figure 7. Typical Load Versus Deflection Curves.

TABLE 16. FLEXURAL STRENGTH OF LABORATORY BEAMS BEFORE AND AFTER REPAIR

Beam No.	(1) R*, Before Repair**	(2) R, After Repair	(2) ÷ (1)
1	645	459	0.71
2	612	554	0.90
3	600	429	0.72
4	633	592	0.93
5	558	460	0.82

*R defined by ASTM C78 as modulus of rupture.

**Average of two breaks per beam.

concrete at the line of failure. That concrete was probably weakened with the development of small internal cracks at the time of initial breaking.

Five beams were cracked by center point load, and efforts were made to prevent their breaking apart. These beams were pressure-injected in much the same manner as the beams of Series B. Crack widths ranged from 0.011 in. to 0.110 in., and the injection pressure was 20 to 30 lb per square in. After the bonding epoxy had cured, the beams were broken under center point load to test the epoxy bond. One beam broke partially in the epoxy bond and partially in the concrete. The other 4 beams broke in the concrete but the break line crossed bond line at about mid-depth.

Five beams that had remained in the moisture room were cracked and injected with epoxy in the same way as just described for the dry beams. Crack widths for the wet cracks ranged from 0.010 in. to 0.052 in., and the injection pressure was 20 to 30 lb per square in. The repaired wet beam break occurred in the bond line at about 1/3 the load required to break the air-dried-injected beams. The epoxy resin used for this repair was the same as that used for repairing the dry beams. It was formulated by the manufacturers for use under dry conditions but was used with these wet epecimens to test its strength under wet conditions. This emphasizes the need that care be exercised in selecting an epoxy system for crack repair. Formulations are available for applications to wet surfaces.

In further laboratory tests, the half length beams from earlier tests in this series were cracked and pressure-injected. After curing, they were broken under a center point load following the same test procedure as for the full length beams. The failure in these tests occurred in the concrete adjacent to the bond line, similar to those of previous beam breaks. The results are shown in Table 17.

Series D:

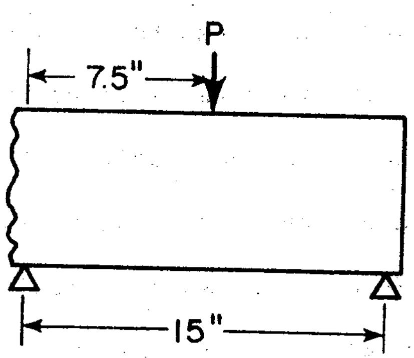
The data from the tests are given in Table 18. The average strengths of the epoxy bonded specimens were approximately the same as those broken after freeze-thaw cycling. This would indicate that the freezing and thawing affected both the epoxy bonded and unbroken specimens in about the same way. About 1/2 of the breaks in the bonded prisms occurred primarily in the bond line. These are more bond line failures than found in specimens which had not been subjected to freeze-thaw cycles. There are not enough freeze-thaw tests to state conclusively that freezing and thawing had a detrimental effect on the bond, but the results favor that conclusion.

Series E:

Sections of glass 2 ft square separated by shim brass were used to simulate cracks for epoxy injection. Although the polished glass surface is far different than the rough fractured surface of concrete, the glass crack test gives information of a qualitative nature that might be useful in concrete crack injection. Crack widths varied from 0.004 in. to 0.013 in., and injection pressures varied for 10 psi to 60 psi. A paint pressure pot adapted for epoxy injection was used as the pressure pot in the tests to pressure-inject the epoxy between the pieces of glass. The entry port was located midway on

TABLE 17. MODULUS OF RUPTURE OF LABORATORY TEST BEAMS

Beam No.	Initial Cracking Modulus, R (Psi)	Cracking Modulus after Injection, R (Psi)
1	885	855
2	750	935
3	810	1030
4	865	1000
5	990	720
6	810	865
7	780	730
8	905	790
9	1030	1140
10	980	960

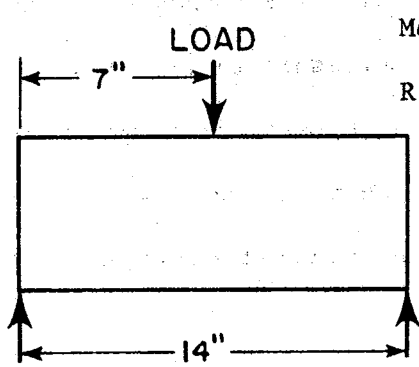


6" x 6" "half beam" broken by center span loading in previous tests. Modulus of rupture = R

$$R = \frac{5}{48} P \text{ (Psi)}$$

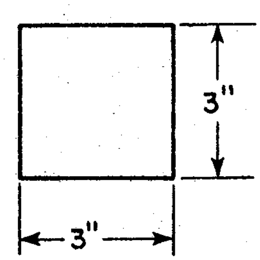
TABLE 18. CRACKING LOADS FOR FREEZE-THAW PRISMS

Specimen No.	Modulus of Rupture Initial (Psi)	Modulus of Rupture After 50 Freeze-Thaw Cycles (Psi)
1	760	580
2	890	780
3	855	795
4	835	740
5	720	740
6	680	-
7	660	545
8	985	660
9	755	620
10	835	875
	avg. 798	avg. 704
11		700
12		720
13		740
14		815
15		780
16		620
17		660
18		780
19		780
20		
		avg. 733



Modulus of rupture = R

$$R = \frac{7}{9}\text{ Load (Psi)}$$



Section

one of the edges of the square glass. A commercially available low viscosity, 100% solids epoxy was used. The distance traveled by the leading edge of the epoxy was measured for each minute of injection time. Figure 8 shows a typical injection sequence of a 0.010 in. crack after 1, 3, 5 and 10 minutes respectively. Data for similar tests are given in Table 19. Figure 9 shows the relationship between crack width, velocity and injection pressure as observed in these limited tests.

For the 0.004 in. crack width, an increase in pressure from 30 psi to 50 psi resulted in little change in the flow of the epoxy. At pressures less than 30 psi, no flow of the epoxy into the crack could be observed. An injection pressure of about 40 psi was found to be the best pressure for the simulated 0.006 in. crack width. The distance traveled by the leading edge of the epoxy at 40 psi exceeded that at 50 and 60 psi. The best injection pressure for the 0.010 in. crack was found to be 50 psi. Flow of the epoxy was evident at pressures as low as 10 psi for the .010 in. crack. The flow observed was about equal for pressures of 30 and 40 psi. About the same distance of flow was observed for the 0.010 in. crack at 50 psi as the 40 psi but in 1/2 the injection time. The data from pressure injection of the 0.013 in. crack indicate that a pressure of about 30 psi would produce about as much penetration as 50 psi. Over the range of crack widths used in this test, an injection pressure of about 40 psi was found to be desirable for the equipment and test set utilized.

TABLE 19. PRESSURE INJECTION OF SIMULATED CRACKS

Crack width in.	Injection Pressure (psi)	Velocity of Leading Edge of Epoxy (in/min)	Velocity ÷ Pressure (in/min/psi)
.004	30	0.33	0.011
.004	40	0.43	0.011
.004	50	0.46	0.009
.006	10	0.18	0.018
.006	20	0.40	0.020
.006	30	0.25	0.008
.006	40	1.41	0.035
.006	50	0.76	0.015
.006	60	1.03	0.017
.010	10	0.33	0.011
.010	20	0.43	0.022
.010	30	1.37	0.045
.010	40	1.31	0.033
.010	50	3.00	0.060
.013	10	0.77	0.077
.013	20	0.81	0.040
.013	30	1.94	0.064
.013	40	1.34	0.033
.013	50	2.00	0.040

1 minute

3 minutes

5 minutes

10 minutes

Figure 8. Simulated Crack, 0.010 inch, Injection Pressure at 50 psi.

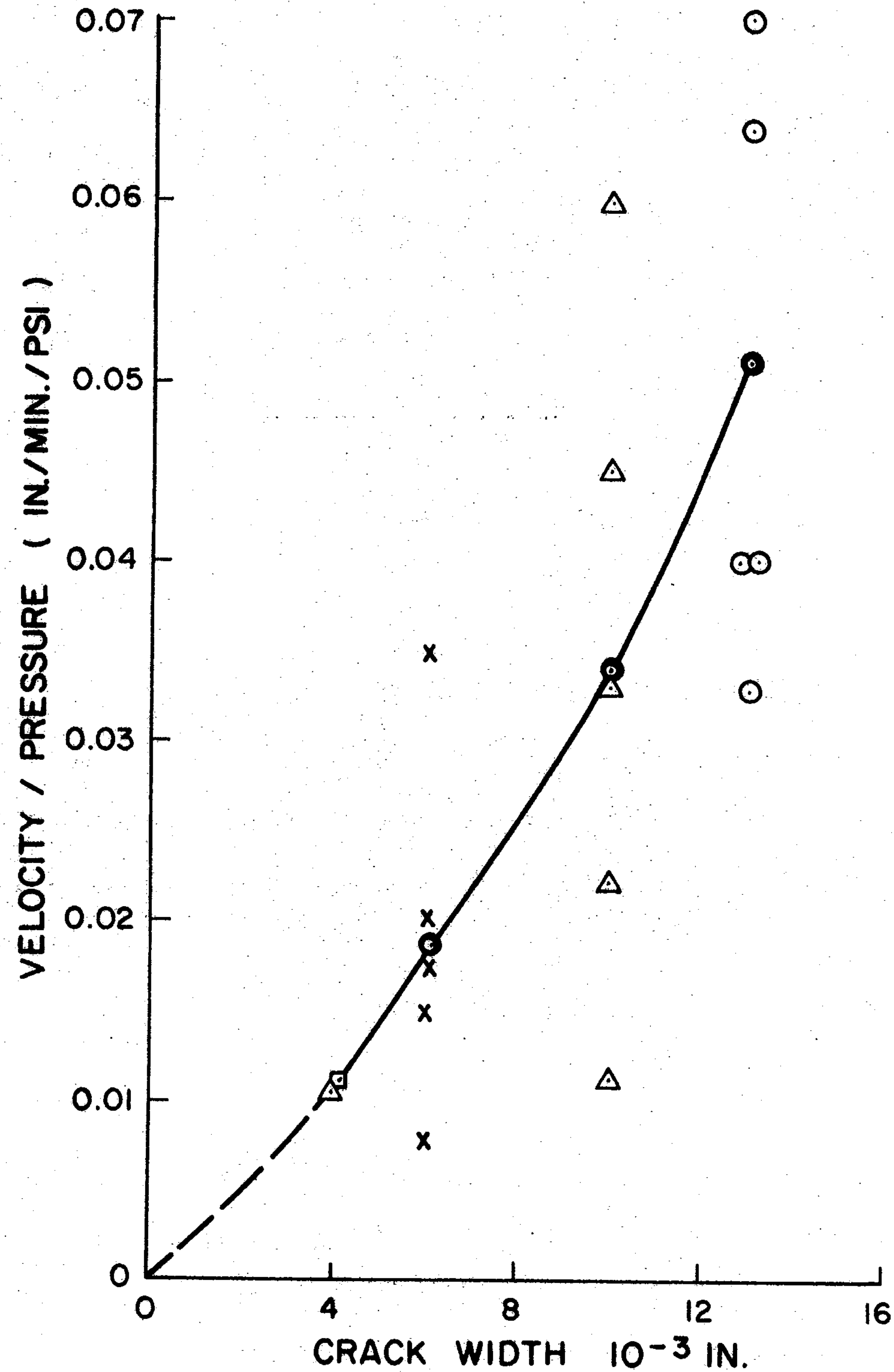


Figure 9. (Velocity ÷ Pressure) Versus
Crack Width For Simulated Cracks.

Series F:

Laboratory tests with the 20 in. square blocks have been described earlier in the report. Poor bonding was evidenced in the area of the splash zone on the first block tested. This area covered about 3 in. to either side of the calm water line. The failure in flexure of the area above the water line was completely in the concrete. In the approximately 6 in. length along the crack just above the water line, the failure occurred in the bond line. In the immersed part of the crack, failure occurred partially in the concrete and partially in the bond line.

The flushing with a solvent in an effort to remove some of the oil film in the crack did not prove to be particularly advantageous. The failure results of this second block were similar to those of the first block tested. The failure of the third block, in which oil was introduced along the full length and width of the crack, was completely in the bond line. Solvent flushing proved to be ineffective in removing the oil contamination in the third block.

Field Experience

The following paragraphs give a brief description of four field experiences with pressure injection of epoxy. These field exercises were set up to demonstrate to THD maintenance personnel the procedures and equipment that can be used in field repair of cracked concrete structures. Demonstration structures were taken when they became available, and they were not always suitable for the stated purpose.

In each case, the epoxy used was a low viscosity epoxy resin of 100% solids.

San Fernando Creek Bridge:

The San Fernando Creek Bridge is located on State Highway 29 between Llano and Mason, Texas, THD District 14. The epoxy injection repair was made on a relatively large crack in a rectangular column. The crack extended approximately half the perimeter of the column. It was caused by settlement of the companion column. The exterior of the crack was sealed with a thick, fast curing epoxy leaving entry ports at about 6 in. intervals along the crack. Injection began at the port nearest one end of the crack and progressed from port to port around the perimeter of the column. Injection was stopped at a port when it was evident that there was no flow or when epoxy flowed out of the adjacent unplugged port.

Maintenance personnel of THD District 14 were present and assisted in the injection.

Palmer Grade Separation Bridge:

This structure was severely damaged when a fuel truck went out of control, skidded underneath the bridge and caught fire. One half of the structure, spanning the northbound traffic lanes of IH 45 near Palmer, Texas, had to be completely replaced. Heat from the fire caused many small cracks to form in one pier cap. It was assumed when the work began that the cracks extended about 4 in. into the pier so entry ports on the surfaces of the cracks were spaced about 4 in. apart. The surfaces between cracks were sealed and the following day injection began. A pressure pot with 3 exit lines, clear tygon plastic tubing, was utilized to work with 3 separate cracks at the same time. It was found that only 2 cracks of the many on each side of the pier took any appreciable amount of epoxy at injection pressures up to 60 psi. It is believed that the majority of the cracks were very shallow and if waterproofed, would not be of concern to the integrity of the structure.

This structure did not lend itself to epoxy injection repair, but it provided a subject for demonstrating the equipment and procedure to THD maintenance personnel.

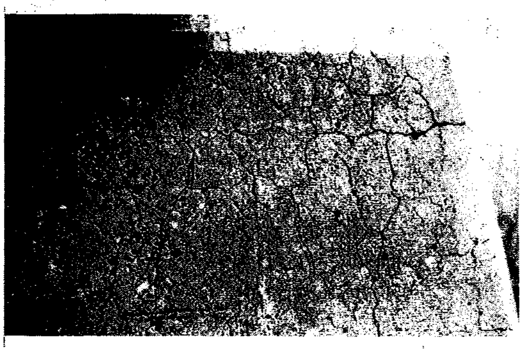
Neches River Bridge:

The bridge is located on Highway 87 between Port Arthur and Orange and crosses the Neches River. TTI worked with THD District 20 in setting up an experimental program to make epoxy injection repairs to two different types of piling caps--one circular in shape and the other trapezoidal in shape. According to the plans for the structure there were 6 in. of concrete cover over the steel in the piling caps. It was assumed then that the cracks extended at least to the depth of the steel.

The pressure pot method was used for pressure injection of the cracks. A low viscosity, 100% solids epoxy formulated for wet or damp conditions was used. The exterior of spacing entry ports equal to the desired penetration depth was followed. It was assumed that the cracks were 6 in. deep; therefore, entry ports were spaced approximately 6 in. apart along each crack. Entry ports for the majority of the cracks consisted of short lengths of tubing, about 1 in. long and 1/4 in. inside diameter. These ports were attached to the surface of the pier with contact cement so that a small segment of the crack was covered by the tubing. Some relatively large cracks were found near the base of the trapezoidal cap of which portions are beneath the water during high tide. There was also some splashing of salt water against the piers when boats passed. Holes were drilled into these larger cracks with an electric pneumatic hammer drill, and plastic one-way valves were installed. Friction between the valve and the concrete was sufficient to hold the valves in place.

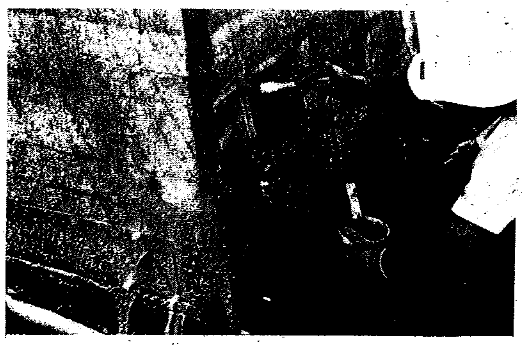
The surfaces of cracks between entry ports not susceptible to wetting were sealed with a general purpose epoxy adhesive of thick consistency. For those areas where the crack surfaces between ports were damp or wet, an epoxy formulation of thick consistency for use under wet conditions was used. Figure 10 shows THD personnel sealing the surfaces of cracks in the trapezoidal piling cap.

The crack sealing epoxy was applied and allowed to cure over night. When the pressure injection began, it was found that very little epoxy could be forced into the cracks with injection pressures up to about 80 psi. Efforts were then begun to determine the cause of the high resistance to flow of the epoxy. The hammer drill was used to drill about a 1 in. deep hole into some of the cracks. This revealed that most of the cracks extended from about 1/2 to 1 in. into the cap. Only the cracks near the bottom of the cap which were much wider at the surface extended to depths greater than about 1 in. After this



a. Cracking in the Circular Shaped Cap

b. Placing Entry Ports and Sealing Between Ports



c. One-way Plastic Valves Installed and Special Epoxy Used in Seal Cracks

Figure 10. Cracking of Caps; Placing Entry Ports for Epoxy Injection.

discovery, efforts to pressure-inject smaller shallow cracks were abandoned.

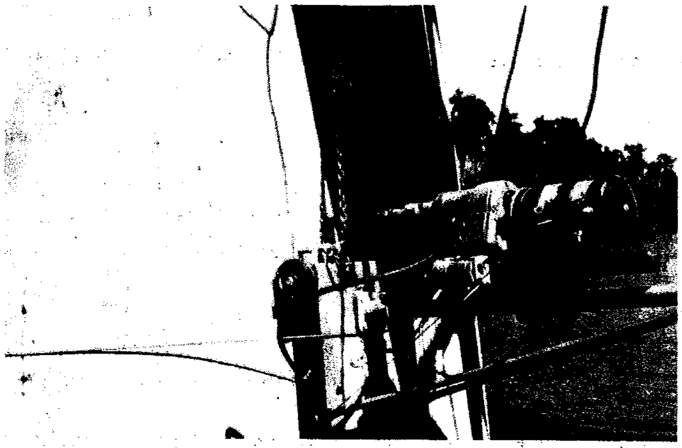
Many of the cracks in the circular shaped cap extended around the perimeter of the cap. A portable coring unit was used to take cores in the cracks to determine the extent of cracking, Figure 11a. Two main cracks in the upper portion of the circular cap were selected on which to concentrate repair efforts. Figure 11b shows the injection in progress and coring of cracks that had been injected the previous day. Cores were cut about 3 in. into the cap along the cracks. They extended further into the cap and were filled to the depth cored.

Maintanance personnel from THD District 20 were present and assisted in all phases of the job.

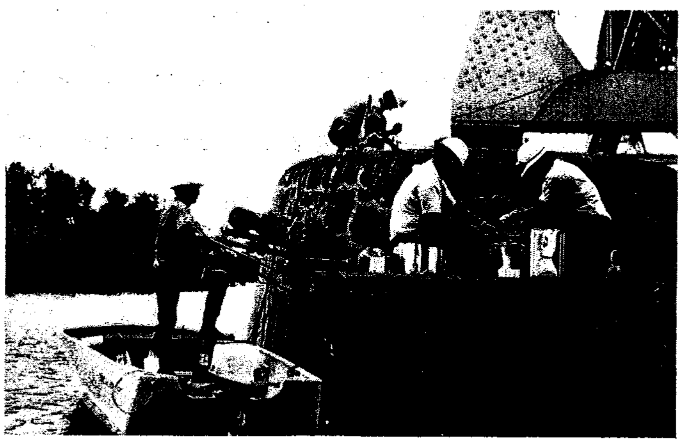
Bell street Bridge Over IH 27 in Amarillo:

This bridge had continous arch shape cast-in-place concrete beams supporting a concrete slab. One of the exterior beams cracked severely due to an undetermined cause. The crack was located at about 1/3 span and extended diagonally the full depth of the beam. A steel plate was positioned beneath the beam and anchored to the slab above. Steel plates welded to the bottom plate extended about 4 in. up the sides of the cracked beam making it impossible to seal the exterior of the crack along the bottom of the beam and partially up the sides.

On the accessible portion of the crack, entry ports were spaced about 6 in. apart. The crack surface between ports was sealed with a thick epoxy gel. Epoxy injection began approximately 5 hours after the gel was in place. Injection began on each side of the beam simultaneously



a. Coring Machine Set Up for Coring



b. Epoxy Injection in Progress and
Coring of Crack Previously Injected

Figure 11. Epoxy Injection on the Circular Piling Cap.

at the lowest entry port. Injection pressures ranged from about 50 psi where the crack was wide to about 80 psi where the crack was narrow. No measurements of crack width were made. It became apparent that the epoxy was flowing to the void area between the bottom of the beam and the steel plate after about 1/2 gallon of epoxy had been injected. It was thought that if the epoxy in the void were cured, it would seal off that area and allow the filling of the remainder of the crack. Injection was stopped about 5:30 p.m. and continued the following morning. Injection began at the next highest port not used the previous day. A total of about another gallon was injected into the crack. The area between the steel and side of the beam had to be plugged to prevent the epoxy from dripping out. Realizing that large quantities might have been needed to fill the void area between concrete and steel, the injection lines were moved up to higher ports to introduce epoxy into the upper portion of the crack.

Maintenance personnel from THD District 4 were present and assisted with the repair work.

Implementation: Epoxy Resin Repair

The excellent adhesive properties of epoxy resins have made them very attractive materials for highway construction and repair.

The variety of epoxy compounds marketed today makes it essential that the labels be read and understood by those people working with epoxy products. No amount of equipment will substitute for worker education. Those involved in using epoxy materials should be thoroughly informed of the characteristics and hazards of the particular materials they must handle. The handling of epoxy materials is not a dangerous occupation as long as reasonable care is taken, and personnel and equipment are kept clean. The possibility of a burn, a damaged eye, or other loss-of-time accident makes knowledge and observance of handling practices absolutely essential.

It should be noted that repair of active cracks, "working" or expansion cracks, may not prevent the crack from propagation at a later date. Active cracks should receive additional engineering considerations other than maintenance and repair. In the sections that follow, detailed methods are presented for crack repair by epoxy injection. The details are presented to aid the user in implementing the methods presented.

Crack Preparation, Dry Cracks:

Past work has indicated that a great deal of time and effort has gone into crack preparation. This expenditure of time may not be justified, and a minimum preparation as described in the following steps should be required for pressure grouting.

1. The outside surface along the crack must be clean. A solvent such as methyl ethyl ketone can be used to remove dried mud, grease, or other foreign material. Concrete coatings such as paint not tightly adhering to the concrete should be removed.
2. If any crack has a lot of loose debris in it at the surface, it should be blown out with an air jet, free of oil and moisture,

from a 75 to 100 psi source. Any large loose particles should be removed by hand.

3. Large holes or voids in the surface should be repaired as outlined for patching.

Crack Preparation, Wet Cracks:

Cracks on surfaces exposed to rainfall should not be grouted until all concrete has had an adequate opportunity to dry. A rule of thumb is to allow twelve hours longer for the crack to dry than for the exposed surrounding concrete surfaces. However, good engineering judgement should be used. Bonding an epoxy resin compound to a damp concrete surface requires special consideration because most epoxy compounds that are available will not set on damp surfaces. Special epoxy compounds are required for this work on damp concrete.

Repair Equipment:

The pressure pot method is recommended. This method of injecting epoxy compounds employs equipment similar to that used in paint spraying operations. A 2-gallon heavy viscosity fluid pressure tank with adequate safety features capable of operating at 100 psi is recommended for use. Such a tank needs to be provided with a pressure source such as nitrogen, capable of maintaining the pressure in the tank at all times. In addition, it must be supplied with an air pressure regulator so that control of the injecting pressure can be maintained.

Placing Procedure for Pressure Injection:

A short steel nipple, about 1/4 in. inside diameter by 3/4 in. long is prepared by reaming one end to provide a snug fit for the flexible injection hose. The other end of the nipple is then ground flat and glued across the crack to the concrete surface.

When placing the piece of tubing, the flat end is coated with "contact" cement. The cement is allowed to dry for approximately 2 minutes or until such time that the glue has become tacky enough to hold the fitting on the surface. The coated end is firmly pressed against the concrete over the crack. Care should be taken not to seal the crack with the contact cement. (Depending on ingenuity of applicator, other methods have been used.)

The entry ports must be properly spaced on all cracks. While guidelines can be given for proper spacing, good judgement must be the final criterion. Guidelines for spacing of ports are given as follows:

1. If the cracks are less than 0.005 in. wide, entry ports should never be spaced greater than 6 in. apart, regardless of the thickness of the cracked member.
2. For cracks in members less than 2 ft in thickness, the spacing of ports should not be greater than the thickness of the cracked member.
3. For cracks in members over 2 ft in thickness, ports should be placed on the crack on all available sides. Ports should be not only spaced at a distance equal to the depth of penetration desired, but also at intermediate points in order to monitor the flow of the epoxy.

The first and last entry ports should be established at or near the bottom and top respectively of any vertical crack or at the ends of any horizontal crack on a vertical wall or on a horizontal member. Subsequent ports should be established according to the guidelines given above.

After the fittings have been in position for 10 to 15 minutes, the crack surface is completely sealed with a thixotropic epoxy compound. In order to withstand pressures exerted by the epoxy as it is being grouted, the thixotropic epoxy seal should be applied with a spatula along the crack to a thickness of 1/16 to 1/8 in. and at the width required to extend the epoxy approximately 1 in. to each side of the crack except at the ports. At the ports the thickness should be approximately 1/2 in. around the fitting. Care must be taken that an excellent seal is obtained around the fittings, as this assures that they will stay in position and that there will be no leakage during the grouting operation. After the cracks and fittings have been sealed, and after the epoxy seal has cured (generally overnight), the cracks are ready to be pressure grouted.

The desired quantity of epoxy grout is mixed in a 1/2 gallon plastic container and placed in the pressure pot. After making sure all exit valves are closed, the pot is pressurized. The injection flow line is inserted into an entry port beginning with the lowest or extreme end of the crack.

After the line has been placed, the exit valve is opened to start flow of the epoxy into the crack. Flow of the epoxy can be monitored by watching the movement of small air bubbles in the line. Generally a few small bubbles can be found. If none are present, bubbles can be introduced by closing the exit valve for the desired line, removing and then replacing the line. When the exit valve is opened, an air bubble will appear. The line is left in place until the epoxy starts to flow out of the adjacent port. When this happens the exit valve is closed. The injection line is moved to the next port and the port just used is plugged. However, if intermediate ports have been utilized, the injection line is not moved until the epoxy grout reaches the port spaced at the desired depth of penetration. As the epoxy appears at each intermediate port the port is plugged. In the case that flow ceases before the grout reaches the desired port the injection line should be moved to the next port which has not been plugged.

Ports are plugged by inserting a 1 in. length of a standard 3/16 in. wooden dowel. The doweling of the type required is available at most paint and art stores, hobby shops, or lumber yards.

The grouting process should be a continuous operation and interruptions must be kept to a minimum. To minimize the time required to replenish the epoxy in the pot, a new batch should be mixed approximately 5 minutes prior to termination of the pot life. As soon as the new batch of epoxy grout has been mixed, the grouting operation is stopped by shutting off the pressure entry valve and the epoxy exit valve. The tank then is depressurized by removing the lid. When the gauge pressure reaches zero, the lid is opened and the epoxy container is removed and discarded. The new container of freshly mixed epoxy is placed in the tank. The lid is replaced and secured, and the tank is pressurized. Before resuming the grouting operation, the old epoxy should be bled from the line. This is done by removing the line from the port, holding the end of the line over a waste container and opening the epoxy exit valve. The line is bled until the old epoxy has been completely removed. This is indicated by the presence of large air bubbles which are automatically entrapped between the old and fresh epoxy during the change over. Once the fresh epoxy has reached the end of the line; the exit valve is closed; the end of the line is replaced in the port, and the exit valve is opened.

Multiple lines may be connected to one pressure pot and used for pressure grouting. However, in order to insure continuity of the injected material within the crack, it is recommended that each continuous crack be grouted utilizing only one injection line.

Should the supply of epoxy grout in the pressure vessel be depleted before the pot life is reached, fresh epoxy should be mixed as rapidly as possible. The same procedures as outlined above is used for recharging the pressure pot.

Any delay in the pressure grouting operation (e.g. lunch break, quitting time, etc.) over 15 minutes in duration demands that the pressure pot be cleaned and all lines thoroughly cleared of epoxy compound. This is accomplished by placing a full container of toluene, methyl ethyl ketone or other solvent in the pot which is then pumped through all lines into a waste container. This process is then repeated with a 1/2 full container of solvent. This process will leave the equipment ready for reuse.

After a crack has been completely injected and the adhesive has cured; the surface seal, fittings, and any spillage may be removed. The injection fittings can easily be knocked off with a hammer. The remaining epoxy can be removed by grinding. While almost any grinder will suffice, a typical high speed hand grinder with a rigid abrasive disk suitable for grinding concrete is recommended. Safety glasses or goggles and respirator should always be worn when performing this operation.

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