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16. Abstract

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A STUDY OF THE EFFECTS OF INTERFACE CONDITION ON THIN BONDED PCC OVERLAYS

by

Kandiah Kailasananthan
B. Frank McCullough
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Research Report Number 357-1

Thin-Bonded Concrete Overlay

Research Project 3-8-83-357

conducted for

Texas State Department of Highways
and Public Transportation

in cooperation with the
U.S. Department of Transportation
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by the

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The University of Texas at Austin

December 1984

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PREFACE

This is the first report produced under Research Study 357, "Thin Bonded Concrete Overlay." The long range goal of this project is to assist the Texas State Department of Highways and Public Transportation in establishing criteria for adopting thin bonded concrete overlay for rehabilitating distressed but structurally satisfactory concrete pavements and in developing construction specifications and design procedures for thin bonded concrete overlays with various alternative materials.

This report presents the results of a series of laboratory tests and tests on field cores from a recently completed thin bonded concrete overlay project in Houston.

The research approach was to construct test slabs in the laboratory, to simulate field slabs and field conditions and test the sample cores for interface bond strength, and bond between reinforcing steel bars and concrete. Interface bond strength was also tested on sample cores from the fied slab.

Many persons have contributed significantly to this work, and the authors are grateful to them all. Special mention needs to be made of Mr. W. V. Ward and the Houston Urban Office staff, who provided assistance through all stages of this study. Thanks are also due to Mr. Jim Long, Mr. Jeff Kessel and Mr. Moussa Bagate for their assistance in carrying out the laboratory experiements, and Lyn Gabbert for her help in typing the manuscript.

LIST OF REPORTS

Report 357-1, "A Study of the Effects of Interface Condition on Thin Bonded PCC Overlay," by Kandiah Kailasananthan, B. F. McCullough, and D. W. Fowler, presents the findings of the laboratory experiments on thin bonded overlay which were conducted as a prelude to completing the Houston 610 Loop experimental section and the results of the experiments on the field cores obtained from the test section. October 1984.

ABSTRACT

The purpose of this study was to verify the feasibility of using thin bonded PCC overlays and to evaluate their performance when exposed to traffic and environmental conditions in Houston.

Laboratory experiments were performed and a test section was constructed; cores were taken and analyzed to determine the correlation between laboratory findings and findings from the field, so as to arrive at useful conclusions that would enable the Texas State Department of Highways and Public Transportation to design overlays for future rehabilitation programs on CRCP.

KEYWORDS: Rigid pavement, PCC overlay, rehabilitation, resurfacing, factorial experiments.

SUMMARY

A series of laboratory experiments was conducted to study the effects of thin bonded concrete overlays at the interface. The study was designed to find a suitable bonding medium and to find the optimum treatment for the original surface. The study was extended to find whether or not the position of the steel affects the bonding capabilities of the thin bonded overlay.

A 1000-foot test section was constructed; cores were obtained from these sections after one month exposure to traffic conditions in Houston, and they were further tested in the laboratory.

The study showed agreement between field and laboratory findings.

IMPLEMENTATION STATEMENT

This study provided several results which can be implemented by the Texas State Department of Highways and Public Transportation. One concerns the optimum treatment for the original surface. It should be, at least, sand blasted, and it is better to mill the surface and then sandblast and airblast to get rid of all loose material and to present a good, clean, dry surface. The overlay should be well compacted. Less well compacted overlays may allow small voids to form underneath, thus weakening the bond.

When an overlay is placed in dry weather, the interface can be dry and no grout is needed. When an overlay is placed in wet weather, it is better to use grout and Daraweld as a bonding material.

In addition, it was found that the position of the steel does not affect the bonding capabilities of the reinforcing steel. It can be placed directly on the surface of the existing pavement, rather than at the middepth of the overlay, thus saving construction time and cost.

TABLE OF CONTENTS

CHAPTER 3. EXPERIMENTAL PROGRAM

CHAPTER 4. CONSTRUCTION OF EXPERIMENTAL SECTION

CHAPTER 5. RESULTS OF TESTING PROGRAM

CHAPTER 1. INTRODUCTION

This chapter presents the background, objective, and scope of the study. The concept of rehabilitation and the history of resurfacing are also discussed.

BACKGROUND

Now that most of the nation's Interstate Highway System is completed, the general interest of the highway administrators is now shifting from construction to maintenance, rehabilitation, and resurfacing. Although the rehabilitation of portland cement concrete (PCC) pavements has generally been accomplished by using bituminous materials for resurfacing, PCC has been used to resurface existing pavements for more than 60 years. Performance data indicate that a relatively low-maintenance service life of 20 years can be expected from PCC resurfacing, and many such resurfacings have provided 30 to 40 years of service. Generally, PCC resurfacings have not been as widespread as asphaltic concrete resurfacings because of their higher initial cost and construction complexity, but several developments within the last 10 to 15 years have caused the states to reevaluate the use of PCC resurfacings. These are

(1) improved construction equipment and procedures;

(2) improved reinforcing techniques, such as continuous reinforcement, fibrous reinforcement, and prestressing;

(3) the uncertain future supply of asphalt and its rapidly increasing cost;

(4) the trend toward selection of resurfacing type based on life-cycle costs rather than initial costs, and

(5) the large traffic volumes on some highways, which present traffic handling conditions that are expensive and dangerous and encourage keeping work on a facility at a minimum.

PCC offers a wide range of resurfacing alternatives, including plain concrete, conventionally reinforced concrete, continuously reinforced concrete, fibrous concrete, and prestressed concrete, which can be used with three interfaces: bonded, partially bonded, and unbonded (Refs 6 and 7). Unbonded plain, conventionally reinforced, and continuously reinforced concrete resurfacings have been widely used for highways whereas partially bonded plain and conventionally reinforced concrete resurfacings have been used extensively for airport pavements. Recent developments in surface cleaning techniques have resulted in new emphasis on the use of thin, bonded overlay, especially when the primary need for resurfacing is to improve the rideability or the surfacing texture of the existing pavement in terms of skid resistance.

A major problem with PCC resurfacing is reflection cracking (Refs 1 and 2). Thermal movements and load-induced deflections at joints or cracks in the existing pavements can cause cracking to occur through the resurfacing, which in turn creates potential maintenance problems. The various resurfacing alternatives permit the selection and design of a resurfacing type and interface that will minimize these detrimental effects.

Due to changing economic considerations and uncertainty in future availability of asphalt concrete, increasing interest is being expressed in the use of PCC overlays over existing pavements. Unbonded concrete overlays and bonded concrete overlays, both reinforced and unreinforced and of various thicknesses, have been constructed by a number of agencies.

OBJECTIVE OF STUDY

The purpose of this study is to evaluate the performance of various types of thin overlays placed over existing concrete pavements to determine the circumstances under which such overlays may be effective. The types of overlay considered were

(1) plain concrete, with one level of thickness, 2 inches;

(2) reinforced concrete, with two levels of thickness, 2 and 3 inches; and

(3) fibrous concrete, with two levels of thickness, 2 and 3 inches.

SCOPE OF THE STUDY

For this experimental study, a 1,000-foot section on I-610 Eastbound between Cullen Blvd. and Calais Street in Houston was selected for a field study. The test section was used to evaluate thin bonded PCC resurfacing and to determine its performance when it is exposed to extra heavy traffic loads and to Houston's conditions. Laboratory experiments were performed to predict the behavior of materials used on the test section; this was essential selecting the critical variables.

REHABILITATION

Rehabilitation is defined as "The restoration, resurfacing, or repair of highways and highway facilities to restore the serviceability, as nearly as possible, to the original constructed level of service or as subsequently improved." (Ref 4)

This definition of rehabilitation implies that the repair or restoration of the project provides a level of serviceability equal to its level when it was placed in service. Since the present and future projected traffic levels are, in most instances, greater than the initial estimates, it may be necessary to increase the structural capacity of the pavement for the rehabilitation to achieve its intent of providing satisfactory serviceability-performance for some given time period into the future.

The concrete resurfacings have been used for one or more of the following purposes:

(1) to restore the rideability of the existing pavement,

(2) to provide an appropriate surface texture to the existing pavement, or

(3) to restore or increase the load-carrying capacity and/or fatigue life of the existing pavement.

HISTORY OF RESURFACING

The network of vehicular pavements in this country, from city streets and farm to market roads to the primary highway system and the Interstate Highway system, has been developed through a continual process of construction. These pavements, adequate when constructed, soon experienced ever increasing loadings, in terms of both number and weight. These loadings, combined with the adverse effects of the environment on the performance of construction materials, have resulted in various states of distress. The fact that the nations' economy depends on the pavement network has led the engineer in a continual search for ways to economically maintain the pavement system with minimal disruption to the traffic flows, thus minimizing the user's cost.

Out of necessity, pavement rehabilitation during World War II was minimal and, with the increase in the volume and weight of truck traffic during this period, a tremendous backlog of vehicular pavement rehabilitation work developed. It was obvious that priority had to be given to the upgrading of the nation's pavement network. Many of the original pavements had been constructed with lanes 8 to 10 feet wide and these had to be widened as well as resurfaced. Concrete played an important role in the rehabilitation; however, it was during this period that bituminous concrete resurfacing received the most attention because it produced a dramatic improvement in the rideability of the existing surface and could be constructed with less disruption to the traffic flow. Experience had shown that a concrete thickness of 4 to 5 inches minimum was necessary to prevent excessive cracking due to curling or warping, unless the surfacing could be successfully bonded to the existing pavement. During and immediately following World War II, concrete resurfacing played an important role in the continual upgrading of military air field pavements. Many of the original air field pavements were constructed of 8 to 10 inches of plain or reinforced concrete, generally using standard highway practices. With the rapid increase in aircraft weight and traffic, these original pavements soon had to be resurfaced to increase their load carrying ability. During the Korean War period, an extensive research program was carried on by the Corps of Engineers to develop a system methodology for the design of concrete resurfacing.

The resurgence was instigated both by new designs and decreased demand for concrete materials. The designs have involved continuous or fiber reinforcement, the former being intended for new construction. However, with the Interstate Highway system nearing completion, new construction is decreasing significantly, and emphasis is shifting to rehabilitation. Concrete advocates have turned their attention to overlays, aided by two additional developments -- the uncertain future supply of asphalt and the questionable service life of asphalt overlays on concrete. While both are subject to interpretation and evaluation by individual agencies, the supply problem is more a matter of time. Performance, on the other hand, is subjective. Each agency has its own criteria for adequate service, i.e., what is expected of an overlay on a given pavement in terms of performance and life. Thus, each agency may evaluate the economic advantages and disadvantages of concrete overlays differently.

Many highway engineers have begun to base their recommendations regarding resurfacing on a total-cost economic analysis, which includes initial cost, maintenance and repair costs, and present worth of future resurfacings during the life of the resurfacing. These are often presented in terms of "annualized costs."

The most important consideration is cost. While the price of asphalt has increased more than concrete over the past few years, there is still a considerable gap between them. However, this gap may continue to narrow as world supplies of petroleum decrease. In addition, initial cost should not be the primary criterion: long-term performance must be considered.

The resurgence of consideration of concrete resurfacing is evidenced by the number of concrete resurfacing projects that have been constructed in the last 10 years. Although some of this increased activity can be attributed to emphasis being placed on the Federal Highway 4-R Program of rehabilitation, restoration, resurfacing, and reconstruction, other factors, such as the use of continuously reinforced concrete, fibrous concrete, and prestressed concrete as resurfacing materials and the development of improved bonding techniques, have broadened the application of concrete resurfacings and attracted the attention of engineers. Recent resurfacings utilizing many of these materials and construction techniques have been constructed as test or trial sections to collect data for the extensive pavement resurfacing program facing the highway engineer in the future.

Recent innovations in construction equipment, especially in surface-milling machines, have resulted in renewed interest in the use of thin bonded-concrete resurfacings to upgrade existing pavements. Because of these recent developments, thin bonded-resurfacing has received more attention than the other types of resurfacing, which are essentially unchanged over the last several years insofar as construction techniques are concerned.

CHAPTER 2. OVERLAY TYPES

This chapter develops the basic concept of resurfacing types in terms of interface types and resurfacing types.

INTERFACE

Experience has shown that the performance of a resurfacing can be influenced by the condition of the pavement at the time of resurfacing, which can vary from structurally sound to badly distressed. Thus the existing pavement conditions will influence the selection of the interface treatment. The interface types are characterized by the degree of bond between existing pavement and the overlay and, as used herein, are termed bonded, partially bonded, and unbonded. In the following paragraphs, each of the interface types are discussed further.

Bonded Interface

For a bonded interface, steps are taken during preparation of the existing pavement and resurfacing construction to insure a complete bond so that a monolithic structure results. The steps include meticulous cleaning of the existing pavement, application of a bonding medium, careful placement and consolidation of the resurfacing concrete, and protection throughout the cure period. Joints must be provided in the bonded resurfacing coinciding with those in the existing pavement to minimize uncontrolled cracking. Intermediate cracks in the existing pavement can be expected to reflect through the resurfacing.

Partially Bonded Interface

In this case, no special attempt to achieve or prevent bond between the resurfacing and existing pavement is required. Minimal surface preparation is necessary and normal concrete mixtures, construction practices, and curing procedures are used. Joints in the resurfacing that coincide with or are located within 12 inches of joints in the existing pavement are required, to minimize uncontrolled cracking. If the existing pavement is of long-panel design (over 20 feet), intermediate joints or reinforcement are desirable in the resurfacing to minimize the effects of reflection cracking. Intermediate cracks in the existing pavement can be expected to reflect through the resurfacing.

Unbonded Interface

For an unbonded interface, a positive separation course (unbonding medium) is used between the existing pavement and the resurfacing. Normal paving concrete mixtures, construction methods, jointing layouts, and curing procedures are used for the resurfacing.

Other Interfaces

Concrete may be used to resurface existing flexible pavements (termed "white-topping"). Generally concrete resurfacing is cast directly on the existing flexible pavement; however, portions of existing flexible pavement may be removed and replaced with concrete (inlay resurfacing). Levelling courses may be used when the existing surface is badly distorted. Concrete may also be used to resurface existing concrete pavements that have previously received two or more asphaltic concrete resurfacings resulting in a thick (4-inch) interlayer. Although these resemble unbonded resurfacings, thick interlayers create design problems. Finally, the existing concrete may be broken up and an unbonding medium applied before a concrete resurfacing.

TYPES OF CONCRETE RESURFACINGS

Concrete resurfacings include plain concrete and all types of reinforced concrete. Although the predominant type of resurfacing has been plain and conventionally reinforced concrete, a review of past practices reveals that there have been few, if any, standards established regarding the selection of resurfacing types. Instead, it appears that the final selections of resurfacing type is based on local experience, evaluation of the condition of existing pavement, the causes of the distress mechanism leading to the need for the resurfacing, and an economic analysis. Nevertheless, past practices have led to the identification of certain factors helpful in the selection of resurfacing type. These factors are described for each of the resurfacing types that can be used.

Plain Concrete

Plain concrete may be combined with each of the three interfaces to resurface existing concrete and flexible pavements. Joints must be provided in bonded plain concrete resurfacings must coincide with joints in the existing pavement, to prevent reflection cracking in the resurfacings. For partially bonded plain concrete resurfacings, joints that match or fall within 12 inches of joints in the existing pavement must be provided. Both bonded and partially bonded plain concrete resurfacings are generally restricted to structurally sound existing pavements. When unbonded plain concrete resurfacing is used, there is no requirement to match joints in the existing pavement because the unbonding medium effectively minimizes reflection cracking. For this reason, unbonded plain concrete resurfacings are generally used when the existing pavement is distressed or when it is not economically feasible to match joints.

It is generally considered that thin plain concrete resurfacings must be bonded to existing pavement to minimize distress in the resurfacing caused by warping. Thin, bonded plain concrete resurfacings are normally used to restore the rideability or surface texture of existing structurally sound concrete pavements. Although a thicker bonded plain concrete resurfacing can be used to strengthen an existing pavement, the required thickness will be such that a partially bonded or unbonded resurfacing is practical and probably more economical because of the lesser cost for surface preparation and construction.

Conventionally Reinforced Concrete

Reinforced concrete, which contains distributed steel in the panels, may be combined with any of the three interface types to resurface existing pavements. For bonded resurfacing, joints must be provided in the resurfacing that coincide with tnose in the existing pavement. The matching of joints in a partially bonded reinforced concrete resurfacing with those in an existing pavement is preferred but not essential, because the reinforcement will control reflection cracking resulting from intermediate cracking in the existing pavement, making it possible to resurface distressed pavements with both bonded and partially bonded reinforced concrete. Unbonded reinforced concrete resurfacing is used when the existing pavement is badly distressed or distorted and a levelling course is needed.

Continuously Reinforced Concrete

Continuously reinforced concrete (CRC) contains continuous longitudinal steel reinforcement with no intermediate transverse joints. Transverse reinforcement may or may not be used. From both design and construction standpoints, bonded CRC resurfacings were not considered practical by many engineers and so only a few have been constructed. Minimum thickness, steel requirements, and jointing requirements for CRC resurfacings are essentially the same as for CRC pavements. Partially bonded and unbonded CRC resurfacings are used to restore the rideability and to increase the load-carrying capacity of existing pavements. CRC resurfacings are particularly applicable for existing pavements exhibiting structural distress and when it is not practical to match the joint patterns.

Fibrous Concrete

Fibrous concrete utilizes short, small-diameter fibers randomly dispersed into the concrete during mixing to provide omni-directional reinforcements. Several types of fibers for reinforcement have been researched, but steel fibers are most commonly used in pavement applications. Fibrous concrete nas been used with each of the three interface types to resurface existing pavements. When a bonded fibrous concrete resurfacing is used, joints in the resurfacing must coincide with those in the base pavement. When a partially bonded resurfacing is used, the matching of joints in the existing pavement is preferred but not essential, because the fiber reinforcement effectively controls reflective cracking.

As with plain concrete, thin fibrous concrete resurfacings should be bonded to the existing concrete to minimize effects of warping stresses. The ability of the fiber reinforcement to control reflection cracking permits the use of fibrous concrete resurfacings on existing pavements that exhibit some degree of structural cracking. Because there has been little use of fibrous concrete resurfacing, it must still be considered to be experimental or in the development stage.

Prestressed Concrete

The strength of concrete and its load-carrying ability can be dramatically increased through prestressing; that is, application to the concrete during construction of a significantly high compressive force which offset tensile stresses caused by applied loadings. Prestressed concrete is used widely for structural members and it has been used extensively for pavement applications outside of the U.S., but there has been little use for pavements in the U.S. as a resurfacing material; its use in the U.S. has been limited to a few airfield applications. On one highway, construction with prestressed concrete simulated a resurfacing and for this synthesis has been considered to be a resurfacing of a flexible pavement. Prestressed concrete surfacings in the U.S. have been post tensioned, which requires the use of a friction reducing material (unbonding medium) at the interface. The inherent high strength of prestressed concrete resurfacings makes them practically applicable for restoring or increasing the load-carrying capacity of existing pavement. Because of the limited use for resurfacing at this time, prestressed concrete must be considered as experimental or in the development stage.

CHAPTER 3. EXPERIMENTAL PROGRAM

This chapter deals with the experimental program. Details of the laboratory experiments and project information on the experimental section are given, together with a description of the field core experiments.

The experimental program on this research project consisted of two parts. First, a set of laboratory experiments were performed to help in identifying critical variables to be used on the main project at the Houston 610 South Loop. The second part of the project was the actual physical construction of the experimental sections, monitoring deflections and measuring crack width, etc., periodically. The scope of this report is limited to

(1) the laboratory experiments,

(2) the physical construction of the section, and

(3) the laboratory experiments performed on the field cores obtained at these sections.

LABORATORY EXPERIMENTS

The following experiments were performed in the laboratory.

(1) Slab Model Overlay Experiment;

(2) Cylinder Overlay Experiments;

    (a) Cylinder Overlay Experiment I

    (b) Cylinder Overlay Experiment II; and

(3) Bond Pull Out Test.

The details of each of these experiments are discussed in this section.

SLAB MODEL OVERLAY EXPERIMENT

The objective of this experiment was to study the behavior of bond strength at the interface with respect to

(1) different surface treatments, such as sand blasting and the original surface without any treatment,
(2) different bonding agents (cement grout and epoxy), and
(3) large temperature cycles.

Since this report covers only a part of a large project, the study scope was limited to basic procedures which would provide useful guidelines in selecting a suitable bonding material and an economical surface treatment at the interface. Thus, the study was not structured along statistical lines but, rather, consisted of a series of exploratory probes to determine useful information, to allow decision making in the main project.

The CRCP pavement on Houston Loop 610 has experienced distress including considerable transverse and longitudinal cracking. To simulate that condition, four slabs were selected which had a few hairline cracks on the surface. They represented poor pavement sections which had experienced fatigue failure due to severe traffic conditions. The slabs were of 6.0 feet by 3.0 feet. They were subjected to large temperature cycles to study the effect of possible debonding at the interface. Each slab was cut into two smaller slabs of 3.0 feet by 3.0 feet, to enable easier handling.

Experiment Design

The variables considered were (1) bonding medium (grout or epoxy), (2) environment, (3) surface preparation, (4) locations of cores, and (5) curing period.

Bonding Medium. The bonding media considered were epoxy resin and cement grout. The epoxy consisted of resin and hardener, which were mixed together. The cement grout had a water-cement ratio of 0.62 (seven gallons of water per bag of cement).

Environment. Two levels of environment were included in the experimental design. One was room temperature and the other was large temperature cycles, which consisted of a high temperature of 95°F and a low temperature of 30°F. The high temperature, 95°F, was the mean high temperature which Houston experiences and the low temperature, 30°F, was the mean low temperature. To create these environments four slabs were exposed, alternately, to hot and cold environmental chambers. The temperatures of these rooms were controllable adjusting the thermostats. Thermometers kept in these rooms were read every half hour to ensure that a of thermal equilibrium was reached. Four slabs were kept at room temperature and the remaining four were shifted between the hot and cold chambers every twenty-four hours.

Surface Preparations. Two kinds of surface preparations were considered: no treatment and sandblasting the surface. In actual construction, the surface has to be scarified before subjecting it to sand and air blasting, but it was not practical to scarify the slab surfaces to simulate field conditions, and it may be inferred that the field performance will be better than predicted from the laboratory findings.

Curing Period. Three curing periods were studied: one, seven, and twenty-eight days. It is an accepted practice in the field of portland cement concrete designs to study the strength after 24 hours, 7 days and 28 days. These strengths are useful for determining a feasible optimum time period for closing the sections for traffic after the placement of the overlay.

Figure 3.1 shows the factorial arrangement of the experiment in the four factors described above.

Preparation Procedure

Location of Cores. Cores were obtained from three locations: edge, corner, and interior. The cores were selected so that each variable was well represented. The locations of the cores are given in Fig A.1 in Appendix A.

Environmental Temperature	Curing Period	Bonding Treatment: Cement Grout		Bonding Treatment: Epoxy Resin	
	Surface Treatment	Normal	Sand Blasted	Normal	Sand Blasted
Normal (Room Temp.)	1 day				
	7 days				
	28days				
Low Temperature Cycle	1 day				
	7 days				
	28days				

Fig 3.1. Factors affecting interface bond strength.

The surfaces of four of the eight slabs obtained by cutting the original four slabs were sandblasted and the surfaces of the remaining four slabs were not subjected to any kind of special treatment. For the interface of two of the slabs with sandblasted surfaces and two of the slabs with unprepared surfaces, a uniform thickness layer of epoxy was spread evenly with a plastic spatula. Grout was spread uniformly on the surfaces of the remaining four slabs. Care was taken to insure that the dry surface absorbed the grout and that there was no bonding of the grout before the concrete was placed. The concrete used was the same asphalt specified for the Houston 610 project. The coarse aggregate gradation was as presented in Table 3.1.

CYLINDER OVERLAY EXPERIMENTS

Two sets of cylinder overlay experiments were performed. In the first set the objective was to compare the performances of interface surfaces with a grout layer and a non-grout surface. In the second set of experiment; the primary objective was to evaluate the performance of (1) a dry interface surface against that of a surface that was wet when an overlay was placed; (2) grout with "Daraweld" at the interface against a no-grout condition at the interface, and (3) high temperature placement against placement at room temperature.

Design of Experiment

Cylinder Overlay Experiment I. The variable considered was the bonding medium. The surfaces of six cylinders were sandblasted. The cylinders were 4 inches in diameter and 8 inches in length. On three of these cylinders, grout with a water cement ratio of 0.62 was brushed on the sandblasted surface. This was followed by a 2-inch-overlay layer. On the other three cylinders, a 2-inch concrete layer was placed directly on the sandblasted surface. The cylinders were kept outside the laboratory and were exposed directly to the environment to simulate field conditions. A wet cotton blanket was kept on top of the cylinders and adequate care was taken to insure that the blanket was wet throughout the duration of the experiment.

TABLE 3.1. COARSE AGGREGATE GRADATION

U.S. Sieve	Percent Passing (By Weight)
1 inch	100
3/4 inch	90-100
3/8 inch	20-55
No. 4	0-10

As the overlay section was to be opened to traffic on the fourth day, the cylinders were tested for direct shear at the interface on the fourth day.

Cylinder Overlay Experiment II. The variables considered in this study were (1) surface condition, (2) bonding media, and (3) environment. The factorial design of the experiment is shown in Fig 3.2.

(a) Surface Condition. The surfaces of twenty-one cylinders were sandblasted; it was not practical to scarify the surfaces of the cylinders using milling equipment, even though the actual surfaces of the test sections were to be milled to a depth of 1/8 inch. Thus, field results should be better than the laboratory results , since there should be better aggregate interlocking with milled surfaces.

Two surface conditions, dry and wet, were considered. The dry surface was expected to absorb the grout faster than the wet surface. This variable was considered to determine whether this phenomena would result in a significant difference in shear strength at the interface.

(b) Bonding Agent. Two bonding agents were used: one was a grout with a water-cement ratio of 0.62 with "Daraweld-C" added, and the other was a plain surface. Daraweld is a bonding agent for concrete. The manufacturer claims it will form a bond, between two surfaces, new and, or old and new concrete stronger than the concrete being bonded. It is a ready-to-use, non-setting, milk-white liquid with a viscosity only slightly greater than that of water and a weight of 90 pounds per gallon (Ref 5).

When mixed with cement and sand, Daraweld-C forms a strong, highly adhesive bonding agent which will adhere to most substances. Daraweld-C bonding when cured is expected to withstand intermittent or continued exposure to water. Mix proportions used were

Daraweld	1 part
Cement	5 parts
Sand	2.5 parts
Water	As required by specification

Environment Temperature \ Surface Condition / Bonding Medium	Dry		Wet	
	Grouted	Non-Grouted	Grouted	Non-Grouted
High Temperature Placement	X X	X X	X X	X X
Room Temperature	X X	X X	X X	X X

Fig 3.2. Factorial represention of cylinder overlay Experiment II.

The cement and sand should be mixed together first. Then the Daraweld-C and water are thoroughly mixed with the cement and sand. Since Daraweld-C bonding grout has a short pot life, it should be used within 30 to 40 minutes, and it should not be retempered.

(c) Environment. The experiment was also designed to study difference in shear strength at the interface when the surfaces are exposed to high temperatures as opposed to room temperatures, in order to be able to predict how the overlaid sections would behave in summer months, when the temperature reaches 90°F or higher. Twelve cylinders representing all the other variables were kept in a heat chamber at a temperature of 100° F and the remaining nine were kept at room temperature. The cylinders were designed to be exposed to these environments for four days as the test sections were to be opened to traffic on the fourth day after the overlay was placed.

BOND PULL OUT TEST

The restoration of pavements usually requires reconstruction of a portion or all of the surface. When new reinforcement is used in the concrete, there is an implied requirement for additional clear cover for the reinforcement. If bars can be placed directly on the existing surface, additional cover is provided besides expediting construction, and thereby reducing cost. But, with rebars placed directly on the surface of the pavement, the rebars do not have full encasement perimeter bonding the new overlay.

The objective of this experiment was to determine the degree of structural integrity of a slab reestablished through restoration methods and techniques and to study the degree of impact the position of rebars has on shear strength at the interface. The test program was designed to provide data for evaluating the effect of following conditions:

(1) surface preparation of the base slab's top surface prior to the placement of overlay

(2) the position of reinforcement, relative to the base slab surface, and

(3) the size of the bar used for the reinforcement.

The variables considered were

(1) thickness of overlay,

(2) curing period, and

(3) position of steel.

Thickness of Overlay

The experiment was designed to study the effects of thickness of overlay on the bond strength between reinforcement and concrete. To facilitate this study, two thicknesses, 2 inches and 3 inches, were selected. These thicknesses were selected to simulate the experimental section in Houston.

Curing Periods

As traffic was allowed on the experimental sections on the fourth day after the overlay was placed, it was necessary to study the strength at four days. The customary twenty-eight-day strength was also studied, and, in effect, two curing period variables were used, four days and twenty-eight days.

Position of Steel

As inferred previously the position of the steel is a critical variable. If no significant difference results from the position of steel with respect to the top of the old surface, considerable cost savings could be realized on construction projects, since it is much easier and more economical to roll the reinforcement grid into place directly on the concrete pavement surface rather than to place the reinforcement at mid-depth. To study this effect, the exact rebar mesh used at the Houston project was obtained and cut into single bars to be used in the 12 inch by 12 inch slabs cast in the laboratory. To simulate the actual site condition, a 3-inch length of transverse reinforcement was used on either side of the main reinforcement. The length of the main reinforcement was cut in such a manner that it provided an adequate grip for pulling the bars out.

Twelve slabs, 12 inches by 12 inches by 3 inches thick, were cast. The surfaces of these slabs were scarified before the final set of concrete. After four days of curing the surfaces of the slabs were air blasted to remove all loose particles. A grout with Daraweld-C was used as the bonding medium. The grout mix was brushed on with a bristle brush. Care was taken to insure that the grout thickness did not exceed 1/8 inch. On these 12 slabs, steel bars were laid on the 3-inch thick base slab before adding the overlay. Twelve more slabs, 12 inches by 12 inches by 6 inches, were cast with the rebars at mid-depth. The slabs were cured under normal laboratory conditions. So, in effect, two reinforcement locations were used: (1) the surface of the slab, and (2) mid-depth of the slab. The factorial representation of the variables is presented in Fig 3.3.

EXPERIMENTAL SECTION INFORMATION

The thin bonded overlay experimental section was located on I-610 in the eastbound lanes between Cullen Blvd. and Calais Street. The overlay was 1000 feet long and four lanes wide. The plan of the test section is shown in Fig 3.4. The concrete was placed in 200 foot test sections. Lanes one and two (the inside two lanes) were included in Phase 1 of the repair; Phase 2 consisted of the repairs of lanes three and four. The original pavement consisted of 8 inches of CRCP on 6 inches of cement-treated base. A 2-inch non-reinforced concrete overlay was placed on one of the sections, and, on two other sections, 2 and 3-inch reinforced concrete overlays were placed. On the remaining two sections, 2 and 3-inch fibrous concrete was placed. The grout was delivered in agitator trucks and sprayed on the dry surface immediately prior to paving. The water-cement ratio of the mixture was approximately 0.6, i.e., 7 gallons of water per bag of cement. The grout consisted of a water reducing plasticizer, Daraweld-C. A cross section of

Cover Thickness	2"		3"	
Curing Period	4	28	4	28
Condition of Steel				
on Surface of Old Slab	X X X	X X X	X X X	X X X
Mid-Depth of Slab	X X X	X X X	X X X	X X X

Fig 3.3. Factorial representation of bond pullout test.

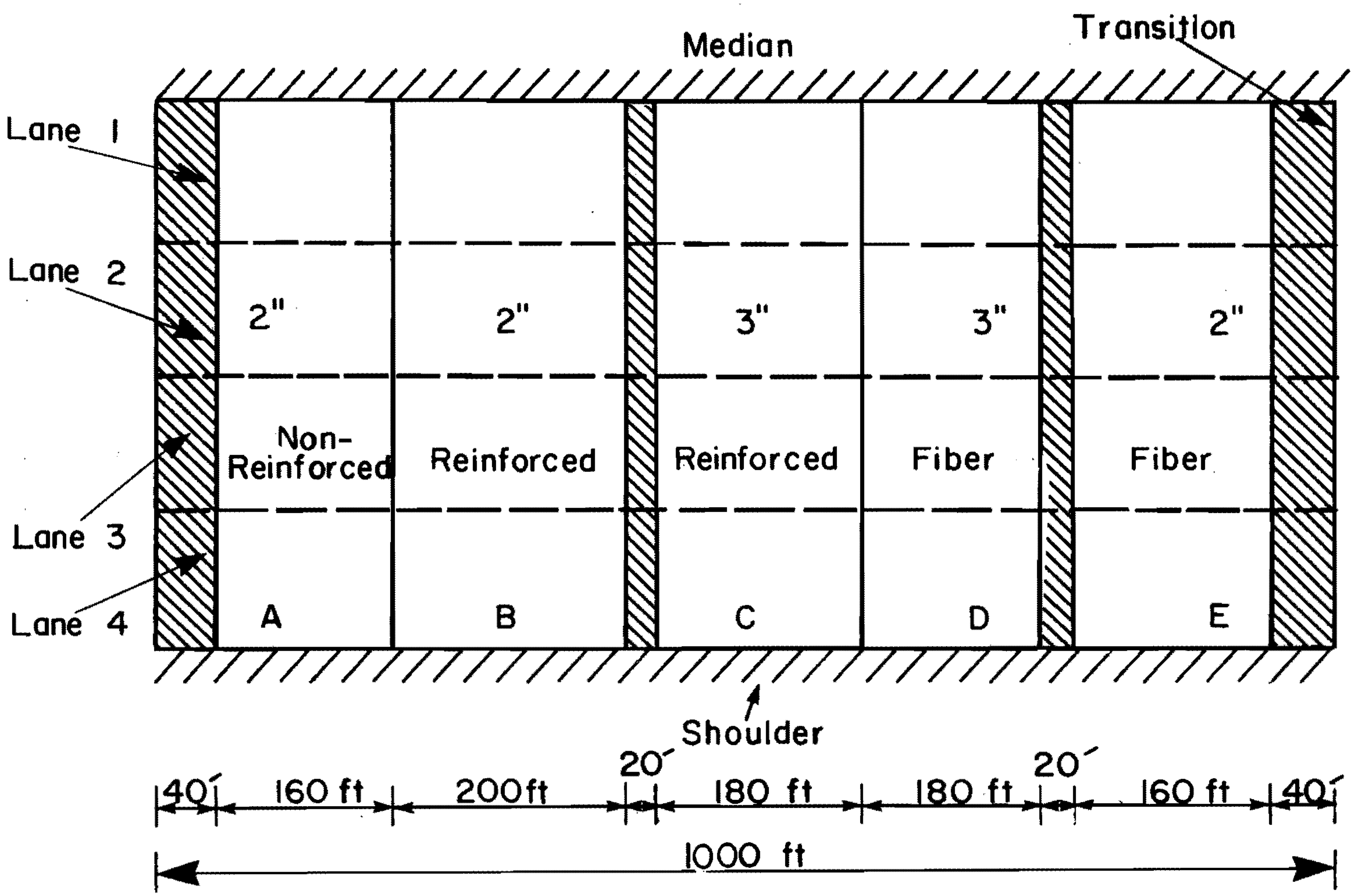


Fig 3.4. Plan of test section.

004 23

the typical pavement structure after the overlay was placed is shown in Fig 3.5.

DESCRIPTION OF FIELD CORE EXPERIMENTS

A month after the overlays were placed and the test sections were exposed to the Houston traffic and environmental conditions, cores were obtained by random sampling representing all the variables. The following tests were performed on the cores:

(1) thermal coefficient tests,
(2) direct shear tests at the interface, and
(3) splitting tensile tests.

COEFFICIENT OF THERMAL EXPANSION TESTS

When a new layer of concrete is bonded to an existing concrete surface, both the new and old concretes are assumed to behave as a composite having the same physical properties. However, due to the difference in the ages of the two materials, there is likely to be some difference in their properties. If the difference in the thermal properties of the new and old concrete is significant, then the two materials will expand or contract differently, which may cause additional stresses at the interface and, then, additional cracking.

This experiment was designed to determine whether there is a significant difference between the thermal properties of the old and the new concrete and also to determine their coefficients of thermal expansion.

One of the field cores obtained had a longitudinal crack and it was split in two. A thermometer was embedded between the two portions at the center of the core and electrically connected to a thermostat. The core was placed alternately in the hot and cold temperature chambers to determine how

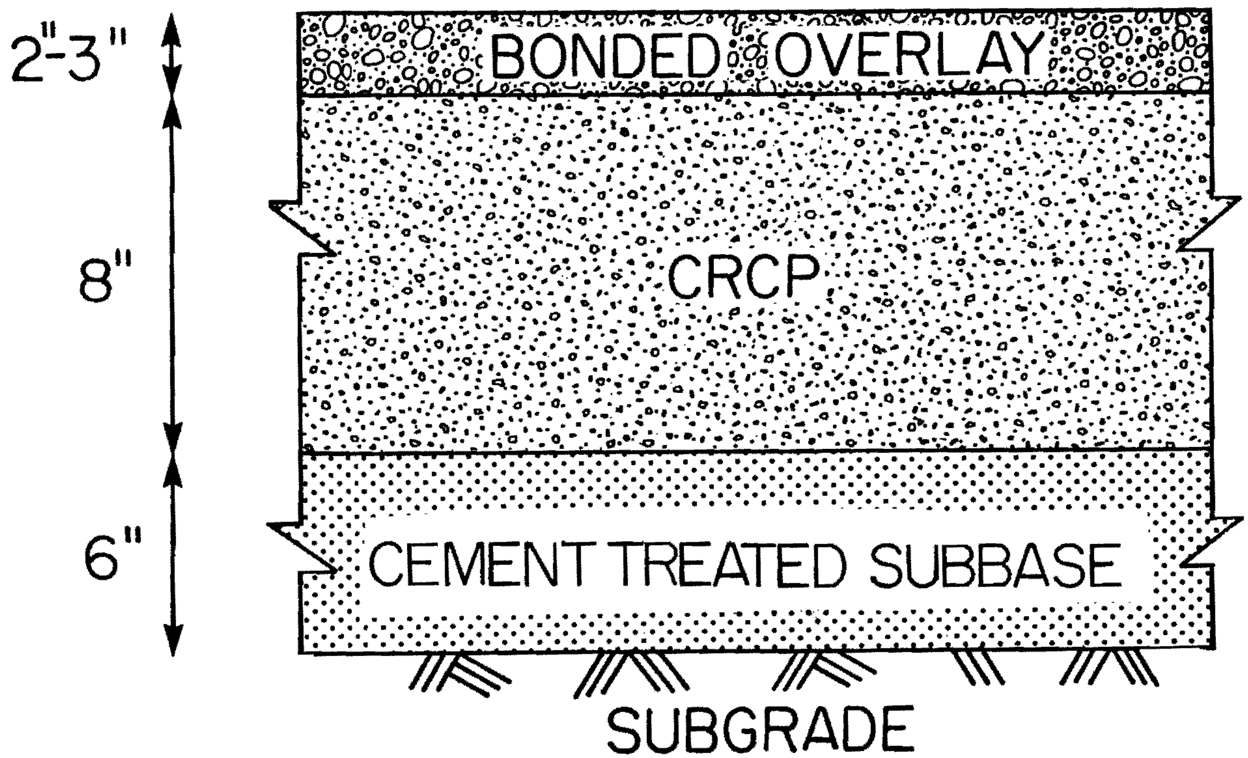


Fig 3.5. Cross-section of typical pavement structure.

long it takes the center temperature to achieve equilibrium, and this time was used as a standard to measure the temperature.

Three more cores were obtained and five plugs were affixed to the core with epoxy. The distances between Plug A and Plug B represented a length of 2 inches at room temperature on the overlaid portion. Similarly, the distance between Plug C and Plug D represented a length of 2 inches at room temperature on the old concrete portion. The distances were measured at different temperatures by means of a strain gauge. The range of temperatures selected was from 30°F to 95°F. A regression analysis using the SPSS program was performed and the results were tabulated.

The distance between Plugs D and E was 4 inches at room temperature and the distances were also measured by means of the strain gage at various temperatures within the selected temperature scale. The temperature in each of the cores was measured a thermocouple connected to the standard core, which had the thermometer embedded at the center. A regression analysis using SPSS was performed for the data obtained and the coefficient of thermal expansion of the old concrete was obtained:

$$\ell t = \ell o + \ell o \; \alpha t$$

where

ℓt = length at higher temperature,
ℓo = length at lower temperature,
t = difference between high temperature and low temperature.

Thus

$$\frac{\ell t - \ell o}{\ell o} = \alpha t$$

Letting

$$\frac{\ell t - \ell o}{\ell o} = K$$

then

$$K = \alpha t.$$

The slope of the regression between K and the temperature gave the coefficient of thermal expansion of the old concrete. This experiment was repeated twice for each core. The average of six slopes obtained was used to determine the coefficient of thermal expansion of the old concrete. The thermal coefficients of expansion similarly obtained were compared for significant differences. The detailed results are tabulated in Table A.8.

DIRECT SHEAR TEST

The shear strength at the interface of thin bonded overlay systems primarily contributes to the success and feasibility of this type of system. If this shear strength is greater than the actual stress which is experienced at the section due to traffic and environmental conditions, there is less chance of failure and the overlay will remain bonded to the old pavement. However, if the shear stress at the interface due to traffic and environment exceeds the shear strength of concrete at the interface, then the bond between old concrete and the overlay will fail, and the two will behave like two independent units and no longer can be considered as monolithic. Thus, the basic design assumptions will be violated.

To study the variations in the shear strength at the interfaces, twenty-nine cores were randomly selected along the length and breadth of the section. These cores represented all the variables. Again, it should be emphasized that since only a few cores represented each variable, no detailed statistical analysis was feasible, but a discussion on the mean of these results is discussed in Chapter 6. The device used to determine the shear strength at the interface was fabricated at the Center for Transportation Research.

SPLITTING TENSILE TEST

When portland cement concrete overlays are placed on existing rigid pavements and bonded at the interface, there probably is a large difference between the age of the existing concrete and that of the overlaid concrete.

This experiment was designed to study (1) whether a significant difference in the tensile strength exists between the new and the old concrete, (2) the variation of tensile strength with depth, and (3) the variation of tensile strength among the portland cement concrete, reinforced concrete, and the fiber reinforced concrete overlays.

Variables

After the shear strength of concrete was determined, ten of the cores were each cut into approximately 2-inch lengths with a diamond saw. The two variables were (1) overlay type and (2) depth of sample from top.

Overlay Type

The three different types of overlays which the samples represented were (1) plain concrete, (2) reinforced concrete, and (3) fiber reinforced concrete.

Depth of Samples from Top

Each core was divided into four sections indicated by A, B, C, and D, used

(1) A indicates overlay,

(2) B indicates 0 to 2 inches from interface,

(3) C indicates 2 to 6 inches from interface, and

(4) D indicates 6 to 8 inches from interface.

The forty samples obtained by cutting these cores were subjected to the indirect tensile strength test. The results are discussed in Chapter 6 and the measurements are tabulated in Table A.8.

CHAPTER 4. CONSTRUCTION OF EXPERIMENTAL SECTION

This chapter deals with the construction of the experimental section. The surface preparation, the sequence of placing concrete, the equipment used, and the control tests performed to ensure the quality of concrete are dealt with in this chapter.

SEQUENCE OF CONSTRUCTION

The existing concrete pavement surface was scarified by a milling machine to a depth greater than 1/8 inch. Then sand blasting equipment was used to remove all dirt, oil, and other foreign material, as well as any laitance or loose concrete from the surface and edges against which new concrete was to be placed. The entire surface was cleaned with compressed air just prior to the application of the bonding agent, which was a water-cement grout.

The prepared surface was dry to allow absorption of the bonding grout. The bonding grout was applied by means of brushes. Care was exercised by the contractor to insure that all parts received a thorough, even coating and that no excess grout collected in pockets. The grout application rate was limited to prevent drying of the grout before it was covered with the new concrete.

DESCRIPTION OF CONSTRUCTION

Generally, the concrete used had seven sacks of cement per cubic yard, 1/2 inch coarse aggregate, and 4.5 gallons of water per sack. The concrete was shipped 6 cubic yard to a load. However, for the fiber reinforced concrete, 8 sacks of cement per cubic yard and 5 gallons of water per sack were used. To control the temperature, ice was added to all loads.

On July 22, 1983, 200 feet of 2-inch non-reinforced concrete was placed in lanes 1 and 2. The concrete was supplied in six trucks. To each load, 150 oz of "Mighty 150", a plasticizing agent, was added. The average slump was 3.75 inches and the average air content was 4.0 percent. The average seven-day flexural strength obtained from the beam test was 823 psi.

On July 26, 1983, 200 feet of 2-inch steel reinforced concrete was placed in lanes 1 and 2. The wire fabric size used was 6 inches by 12 inches with end laps of 12 inches and edge laps of 6 inches. The concrete was transported in six trucks with an average slump of 4.3 inches and an average air content of 2.3 percent. An average of 150 oz of "Mighty 150" was added at the site to each truckload before the slump was recorded.

On July 28, 1983, a 3-inch steel-reinforced overlay was placed in lanes 1 and 2. The concrete was transported in eight trucks. The average slump recorded was 5.2 inches and the average air content was 5.2 percent. Two beams prepared from concrete from the fifth truck had an average seven-day flexural strength of 870 psi. To each of the truckloads an average of 150 oz of "Mighty 150" was added before the slump was recorded.

On August 3, 1983, 200 feet of 2-inch fiber-reinforced concrete was placed in lanes 1 and 2. The fibers were added in 66 pounds bags at the rate of 2 bags per minute. The specifications required 85 pounds of fibers per cubic yard of concrete. The concrete was transported in six trucks. The average slump was 4.5 inches and the average air content was 4.6 percent. Out of concrete transported in the fourth truck, two beams were cast which had an average seven-day flexural strength of 920 psi.

On August 15, 1983, 200 feet of 2-inch non-reinforced and 200 feet of 2-inch steel-reinforced concrete were placed in lanes 3 and 4. The average slump was 3.9 inches and the average air content was 4.0 percent. Two beams each were cast from the concrete transported by the sixth and the eighth truck. The average seven-day flexural strengths were found to be 730 psi and 798 psi, respectively.

On August 20, 1983, the work resumed, after the area was affected by Hurricane Alicia. The concrete was transported in nine trucks. The average slump was 3.6 inches and the average air content was 3.1 percent. Two beams were cast out of concrete transported by the second truck and the average flexural strength was 840 psi.

On August 27, 1983, 200 feet of 3-inch fiber-reinforced and 200 feet of 2-inch fiber-reinforced concrete were placed in lanes 3 and 4. The concrete was transported by fifteen trucks. Most of the concrete in the eleventh, twelfth, and thirteenth trucks was not used because the concrete screed had to be moved back to refinish some concrete. Part of the twelfth truck was dumped, but the rest was sent back to the plant. The average slump recorded was 4.8 inches and the average air content was 5.0 percent. One beam each was cast from concrete out of the eighth and the fifteenth trucks. The average seven-day flexural strengths were found to be 838 psi and 898 psi respectively. These two lanes were opened to traffic on the evening of September 3, 1983, giving the last concrete placed six curing days.

To ensure that the concrete reached its design strength, two control tests were performed during the construction phase of the overlay. One was the slump test and the other was the flexural test on the beams. The air content of the concrete was also measured. Table A.12 gives the results of the slump tests and the flexible strength tests. The slump ranged from 2.5 inches to 8 inches. The flexural strength ranged from 730 psi to 992 psi; the mean strength was 872 psi.

CORES

Twenty-nine cores, randomly distributed throughout the section, were obtained as shown in Table A.10. The cores exhibited rigid bond at the interface. There were not many visible voids in the overlaid portion of the concrete.

CHAPTER 5. RESULTS OF TESTING PROGRAM

This chapter describes the apparatus developed at the Center for Transportation Research to test the shear strength at the interface of the core. The results of the experimental programs are also given in this chapter.

INSTRUMENTATION

Apparatus to Measure Direct Shear

In all the laboratory experiments except the bond pull out test and in the experiments with the field cores, the shear stress at the interface was measured and defined as a function of bond strength. Several types of instruments have been used in the past to determine direct shear on cylinders by various agencies. After investigating available test apparatus, it was decided to develop an instrument which would facilitate measuring the direct shear stress at the interface.

Figures 5.1(a) to 5.1(d) illustrate the instrument developed by the Center for Transportation Research. Figure 5.1(a) shows the sample in the machine, ready to be tested. Figure 5.1(b) shows the load being applied. Figure 5.1(c) shows a specimen failing in shear. Figure 5.1(d) shows a sample after failure.

The instrument consisted of a flat piece of steel (14.5 inches by 4.5 inches by 0.375 inch) welded to a semicircular section of pipe 4.5 inches long and 4.0 inches in diameter. Another steel plate (3.5 inches by 4.5 inches by 0.375 inch) was welded to another semicircular section of pipe with a diameter of 4.0 inches and length of 4.5 inches. The semicircular sections of pipes were each also held onto the plates by means of two rectangular plates welded to either side of the semicircular sections of pipes. These plates gave more rigidity to the instrument. Four holes were drilled in both plates, aligned in such a manner that a bolt could be easily seated though

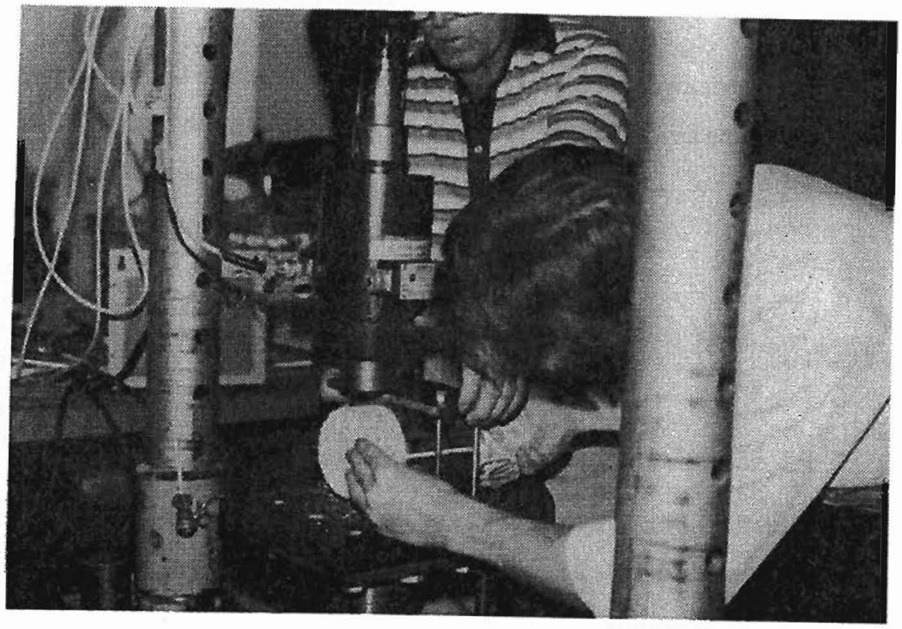


Fig 5.1(a). Sample being placed.

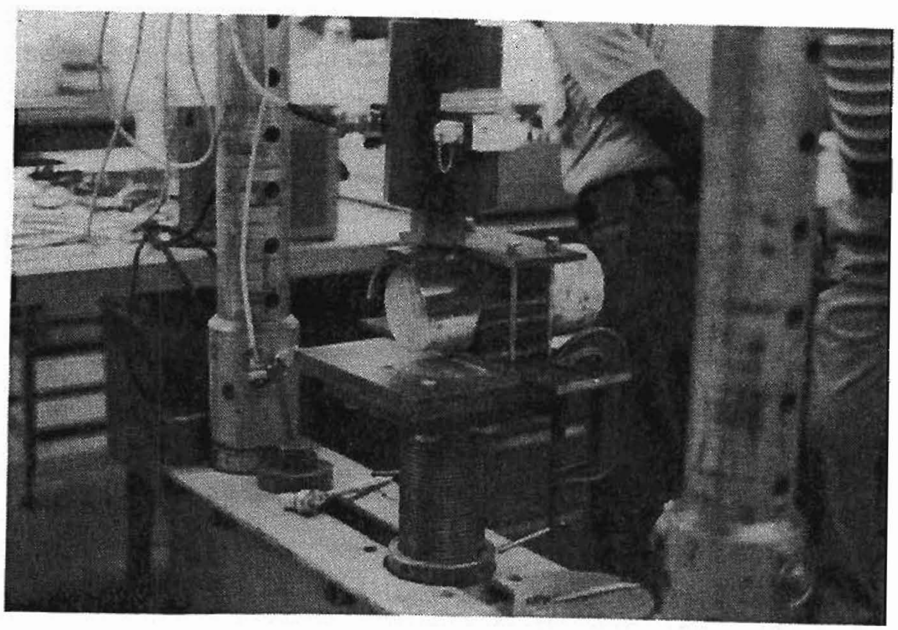


Fig 5.1(b). Sample being tested.

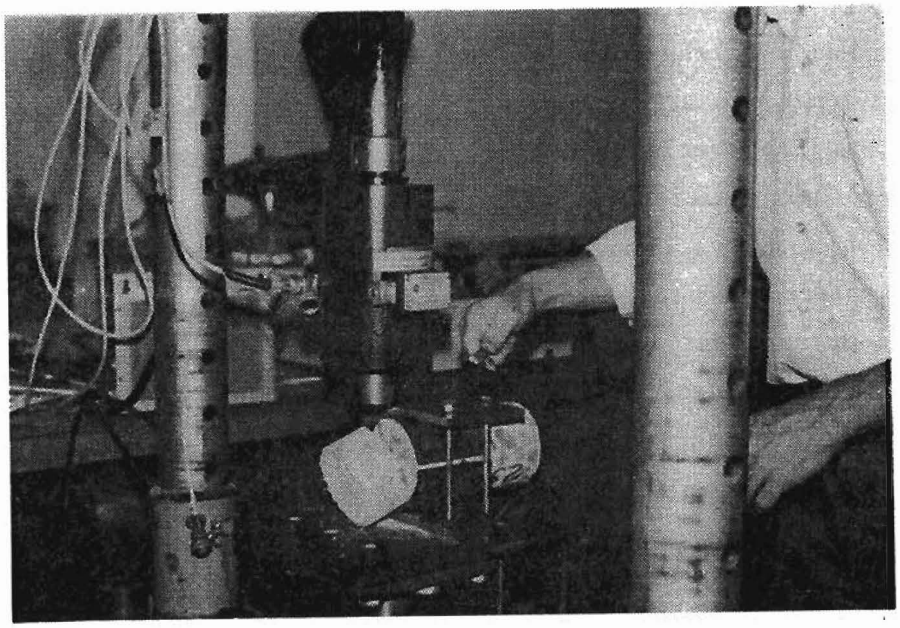


Fig 5.1(c). Sample at failure.

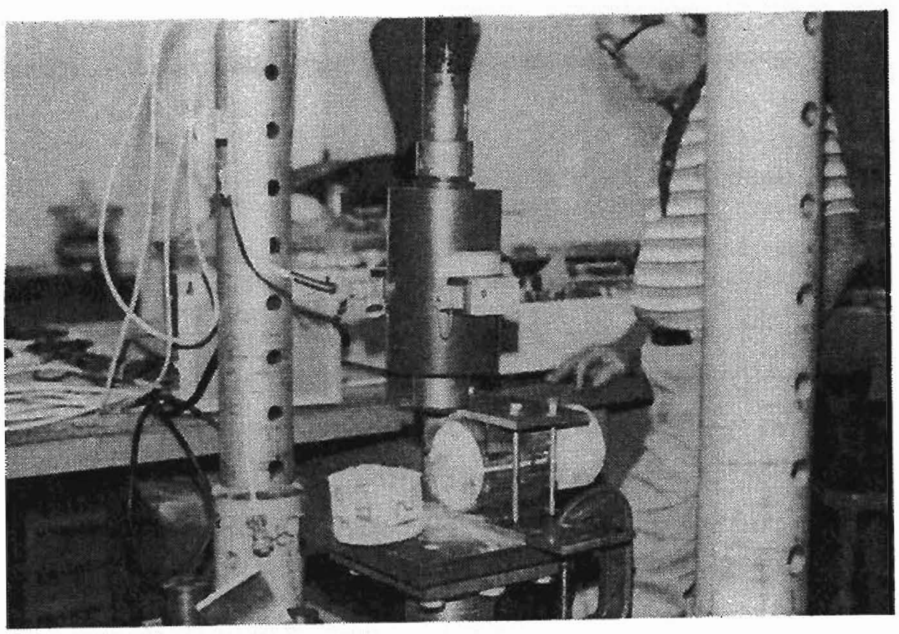


Fig 5.1(d). Sample after failure.

the holes. The holes in the longer plate, which was also the bottom plate, were threaded so that four bolts could be bolted to keep the sample between tne two semicircular section of pipes. When the sample was housed between the pipes, the overlaid portion of the core visible in Fig 5.1(b) projected out. Another semicircular section of pipe, with a diameter of 4 inches and length of 3 inches, which had a thick steel plate 1/4 inch thick welded to it, was placed above the projecting area of the core. The thick steel plate had a depression at the center so that it could easily house a small sphere through which it was feasible to transfer the load.

This instrument was clamped to the table of a universal testing machine by means of two C-clamps and the load was applied in a uniform manner. The plotter ploted the loading automatically on graph paper as the loading progressed. The load at failure was obtained from the graph paper.

The diameter of each core was measured by a surface gauge accurate to one-thousandtn of an inch. A surface gauge was attached to a stand and initially set to zero when the plunger touched the top surface of a 4-inch brass scale. The core was rolled under the plunger and the reading $\pm$ 4 inches gave the actual diameter of the core. For these experiments, the core diameter was obtained as an average of four measurements at the interface.

BERRY STRAIN GAGE

A Berry strain gage was used to measure the distance between two plugs affixed onto the field cores which were used to determine the thermal coefficient of expansion of the concrete. Five plugs were affixed onto the cores by means of epoxy resin. Five plugs were used on each core. Two plugs were affixed on the overlaid region 2 inches apart. Two more plugs were affixed 2 inches apart on the surface of the original pavement. These two sets were used to compare the thermal coefficient of expansion of the old concrete with the new concrete. At a distance of 6 inches from one of these plugs, another plug was attached. This was done to insure that a better measure for the coefficient of thermal expansion. The length of the standard bar and the distances between these plugs were measured by the use of a Berry strain gage, at different temperatures.

LABORATORY EXPERIMENTS

Four laboratory experiments were performed. The results are given below.

Slab Overlay Experiments

Figure A.1 in the Appendix shows the locations of cores. The letters E, C, and I denote the locations of cores: Edge, Corner or Interior; and the subscripts denote the number of days after which the core was taken. The cores were 4 inches in diameter. The recorded data of the observations are in Table A.1.

Attempts to core on the first and seventh days proved to be a failure. Due to the heavy vibrations of the coring machine, the core broke at the interface, thus destroying the specimens for determining the first and the seventh day shear strengths at the interface. However, the mean shear stress at the interface for the sixteen cores obtained after the 28th day was 204 psi. The mean shear stress at the interface for the cores obtained from the slabs which were subjected to high temperature cycles was 237 psi and for those which were kept at room temperature was 173 psi. The mean stress for cores obtained from corner locations was 180 psi, for those from the edge, 167 psi, and for those for the interior, 271 psi. The lowest shear stress at the interface recorded was 115 psi, and the highest was 324 psi.

A subjective attempt to study the effect of corner curling involved using a "Swiss-hammer" at different locations on the slabs. Due to the non-repetitive nature of the experiment and due to the lack of reliability of the instrument, no useful conclusion could be made, but, in general, the results did show weaker bonding in the corners and edges than at the interior.

Cylinder Overlay Experiment I

The recorded data for this experiment are given in Table A.2. The mean shear stress at the non-grouted interface was 227 psi and at the grouted interface was 218 psi. The standard deviation of the shear stress at the interfaces without grout was found to be 29 psi, and that for the grouted interfaces was found to be 5 psi. The coefficient of variation, which is a measure of relative dispersion expressed as a percentage, for the direct shear at the interface for the grouted cylinder was 2.3 percent and for the non-grouted cylinder was 12.8 percent. The lowest shear stress recorded was 191 psi.

Cylinder Overlay Experiment II

The recorded data for these experiments are given in Table A.3. The maximum shear stress, 356 psi, was found on the cylinder which had a dry surface and grout with Daraweld-C as a bonding medium and was exposed to a temperature of 100° F. The least stress, 111 psi, was obtained for the cylinders which had a wet surface and no layer of grout at the interface. The mean shear strength for the cylinders with a layer of grout at the interface was 272 psi. The mean shear strength for the cylinders without a layer of grout at the interface was 127 psi, when both the cylinders were exposed to high temperatures.

Bond Pull Out Test

All bars failed in tension before they could be pulled out after four days, indicating that the bond strength is higher than the tensile strength of steel, regardless of the positioning of the bars. The detailed results are given in Table A.7. Figures 5.2(a) and 5.2(b) show the sample during testing and at failure, respectively.

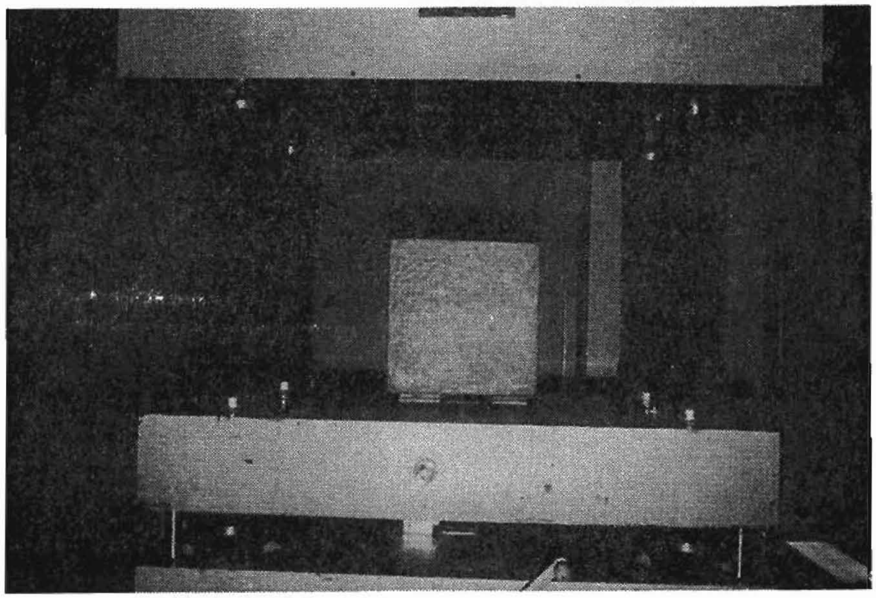


Fig 5.2(a). Sample being tested.

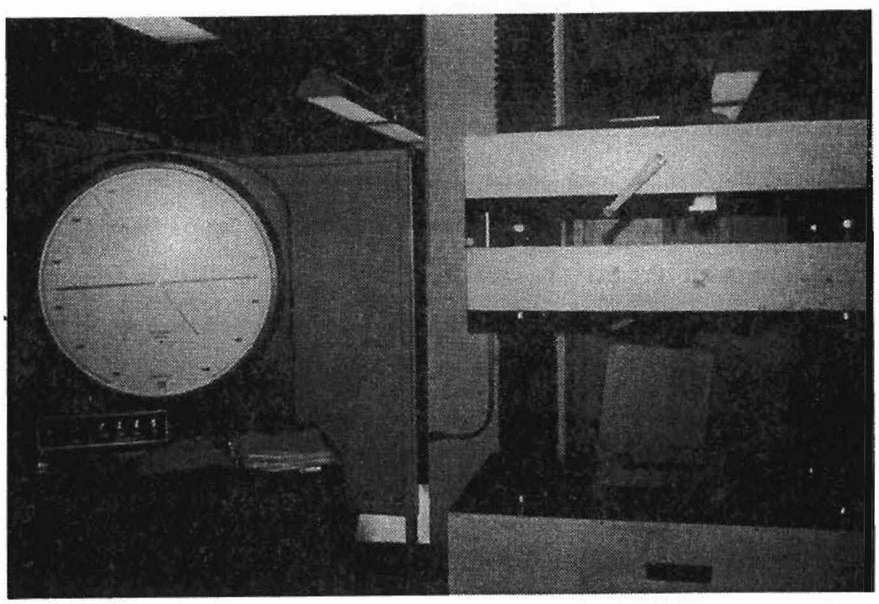


Fig 5.2(b). Sample at failure.

FIELD CORE EXPERIMENTS

The results of the three laboratory tests performed on the field cores are given below.

Coefficient of Thermal Expansion

The results of this test are shown in Table A.8 The mean thermal coefficient of expansion of concrete was found to be 5.333 x 10^{-6}/°F.

Direct Shear Test

The detailed observations for this experiment are given in Table A.5. The average shear strength at failure was 202.0 psi. The highest strength at the interface was exhibited by the core which had a 2-inch plain concrete overlay and was located on one of the outer lanes. The interface of this section had a grout with Daraweld-C as a bonding agent.

The lowest shear strength 79 psi, at the interface was found on core 10, which was obtained from the section which had a 3-inch reinforced concrete overlay was on the inside lane, and also had a grouted layer at the interface.

The mean shear strength at the interface for cores with a plain concrete overlay was found to be 275.0 psi. The mean shear strength at the interface for cores with reinforced concrete was found to be 185 psi and for steel fibrous concrete was found to be 150 psi.

The cores with a 2-inch overlay had a mean strength of 223 psi, and those with a 3-inch overlay had a mean strength of 105 psi.

The cores from inner lanes had a mean strength of 186 psi and those obtained from outer lanes had a mean strength of 220 psi. Cores with grout with Daraweld-C as a bonding agent had a mean shear strength of 135 psi, and those with no grout at the interface had a strength of 213 psi. The lowest shear strength at the interface was found to be 79 psi, which is about 300 percent higher than the critical stress that would be experienced at the interface.

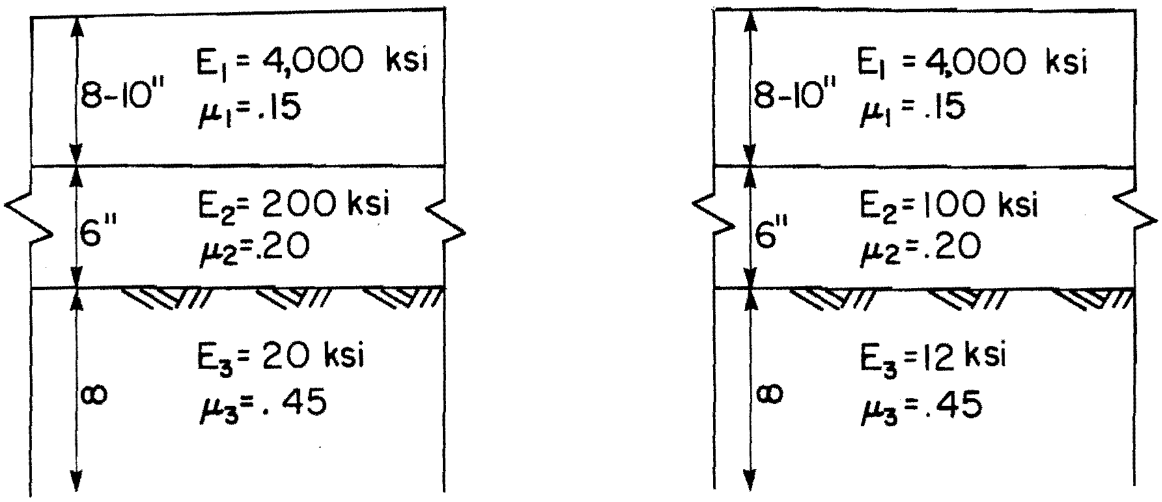
Splitting Tensile Test

The data for this experiment are given in Table A.6. The cores with plain concrete as the overlay had a mean tensile strength of 543 psi at the interface. The cores which had a reinforced concrete overlay had a mean strength of 458 psi at the interface. The mean strength at interface for the cores with a fibrous concrete overlay was 520 psi. The existing CRCP had a mean tensile strength of 498 psi with a standard deviation of 89 psi.

Samples from core 26 with a full-depth crack exhibited relatively low tensile strengths (347 psi and 360 psi). Another core, with a crack but with a fibrous concrete overlay had a strength of 491 psi.

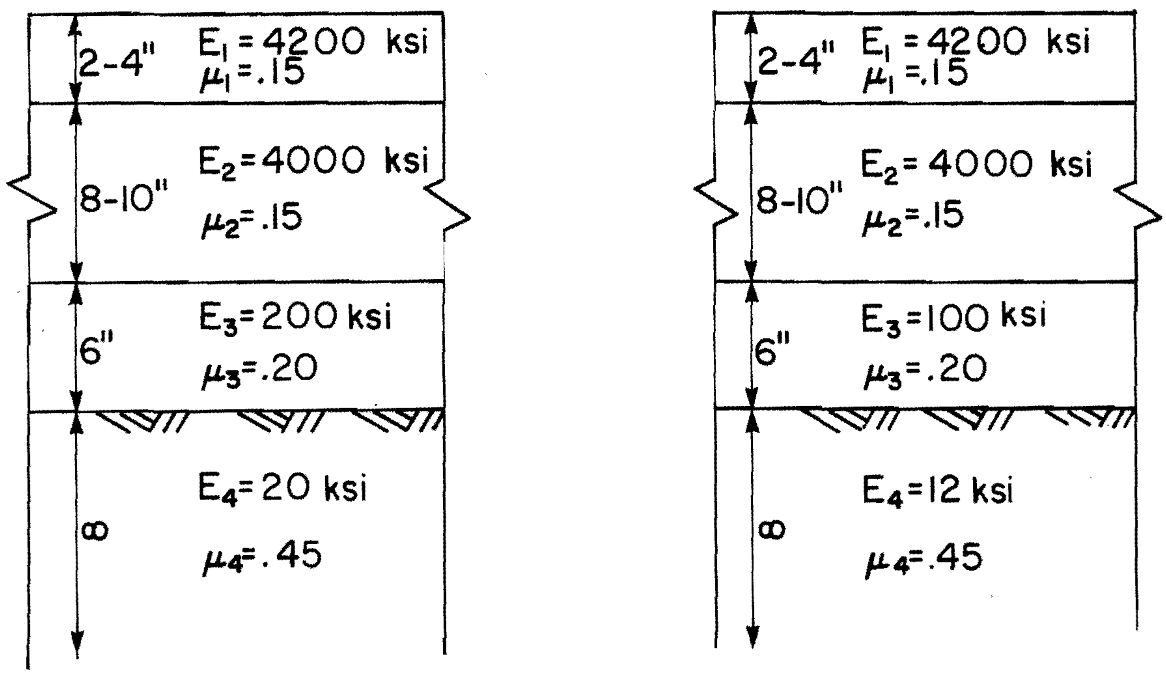
COMPUTER ANALYSIS RESULTS

Theoretical analysis to determine critical shear stresses at the interface were done using a computer program called ELSYM5. Analysis were done for continuously reinforced pavement and jointed reinforced concrete pavement. In each case, overlays of 2 and 4-inch thickness under strong and weak subgrade support conditions were modelled. The cross sections of the pavements assumed are given in Figs 5.3 and 5.4. The only cause of distress considered in this study was loading. A simulated 18,000 pound single axle load was considered. Temperature and moisture differential were not taken into account in evaluating the worst stress at the interface. These stresses could, however, be easily modelled and analyzed using finite element analysis technique. A quick computation under the simplified conditions applicable to the case under study, namely same coefficient of thermal expansion of 0.53×10^{-5} for overlay material and existing slab, and a typical shrinkage strain of 0.39×10^{-3} in./in., yields a combined interface shear stress of 157 psi (including shear due to wheel load). The range of the shear stresses at the interface was from 16 psi to 24 psi, with a mean shear stress of 19 psi and a standard deviation of 3 psi.



(a) Strong support. (b) Weak support.

Fig 5.3. Rigid pavement structures analyzed (before overlay).



(a) Strong support. (b) Weak support.

Fig 5.4. Rigid pavement structures analyzed (after overlay).

CHAPTER 6. DISCUSSION OF RESULTS

In this chapter both the laboratory and the field core results are discussed in general and summarized.

LABORATORY EXPERIMENTS

Slab Overlay Experiment

Since the mean shear strength at the interface for the sixteen cores was 204 psi and the worst condition shear stress at the interface found from computer analyses was 24 psi, the thin bonded PCC overlay is feasible and has a factor of safety against shear failure of over 8. The worst case of shear strength obtained was 115 psi. Even this would yield a factor of safety greater than 4.

The cores obtained at interior regions generally exhibited more strength at the interface than cores obtained from either the corners or the edges. The overlaid portion of the slab separated from the original slab in those slabs which had epoxy as the bonding medium. This may be attributed to the fact that the epoxy may have hardened before the concrete was placed or that the performance of epoxy resins is affected by the semi-liquid form of fresh concrete. Among the slabs exposed to temperature cycles, the one with grout at the interface had better bonding than the one with epoxy, which separated from the existing surface.

Cylinder Overlay Experiment I

The mean shear strength of concrete at the non-grouted interface was higher than at the grouted interface by 9 psi, but the grouted interface exhibited a lower standard deviation, 5 psi, against the non-grouted interface, with 29 psi.

The coefficient of variation also was lower for the grouted cylinders than for the cylinders without grout at the interface, but the lowest stress obtained was 191 psi for a cylinder without grout at the interface. Since the worst stress obtained from the computer analysis was 24 psi, the factor of safety is greater than 7. Thus, it may be economical and feasible to place a concrete overlay directly on the treated old surface rather than to apply grout before placing the overlay. Figure 6.1 shows the mean shear stress at interface for Cylinder Overlay Experiment I.

Cylinder Overlay Experiment II

A summary tabulation of this experiment is given in Table 6.1. The maximum shear strength, 356 psi, was found on the cylinder which had a dry surface and grout with Daraweld-C as a bonding medium and was exposed to a temperature of 100°F. The least strength , 111 psi, was obtained for the cylinder which had a wet surface and no grout at interface. It is seen that the presence of grout on a wet surface affects the bond shear at the interface significantly. Even the shear for the worst condition, which was 111 psi, had a factor of safety of more than 4, since the theoretically predicted shear stress is only 24 psi. This result was also established in the previous experiment. The shear stress for each cylinder is tabulated in the Appendix. Exposure to high temperature does not seem to have a great effect on the mean shear strength at the interface in any of the cases, except on those cylinders which had a wet surface and did not have grout at the interface prior to the placement of the overlay, which had a 30 percent reduction in strength at the interface.

There was also a significant difference in the mean shear strength at the interface between the cylinders which had wet surfaces with grout (with a mean shear strength of 272 psi) and those with wet surfaces without grout (with a mean shear strength of 127 psi) when exposed to the same high temperatures. The shear strength at the interface for the cylinders which had a dry surface was generally higher. This may be due to the fact that the grout was absorbed by the dry surface and resulted in a stronger bond.

The increase in the shear strength at the interface from applying grout does not seem to be high enough to economically justify the use of grout. If

Fig 6.1. Cylinder overlay Experiment I.

TABLE 6.1. CYLINDER OVERLAY EXPERIMENT II: SHEAR STRENGTH IN PSI

SURFACE COND. / BONDING MEDIUM / TEMPERATURE	DRY		WET	
	GROUTED (PSI)	NON GROUTED (PSI)	GROUTED (PSI)	NON-GROUTED (PSI)
HIGH TEMPERATURE PLACEMENT	$\bar{x} = 289$	$\bar{x} = 243$	$\bar{x} = 272$	$\bar{x} = 127$
ROOM TEMPERATURE	$\bar{x} = 282$	$\bar{x} = 217$	$\bar{x} = 268$	$\bar{x} = 196$

the surface is dry, i.e., the overlay is being placed under dry weather conditions, it may be economical to omit the grout. On the other hand, if the overlay is being placed under wet weather conditions, it is advisable to apply grout before placing the overlay.

Bond Pull Out Test

All bars failed in tension before they could be pulled out. This test demonstrated that the location of rebars has no bearing on the bond strength at the interface, and that the bars will not lose anchorage when laid on existing surface and then covered with overlay.

The bond strength at the interface was not evaluated as a part of this experiment. It is evident from the test results that the capacity at the interface was adequate for the bars tested. Additional tests are recommended to determine a numerical value or range for the bond capacity at the interface. Figure 6.2 shows the mean load at failure for the different variables considered.

FIELD CORES EXPERIMENTS

The results of the field core experiments are discussed in this section.

Thermal Coefficient Tests

The thermal coefficient of expansion was found to be $5 \times 10^{-6}/°F$. There is no significant difference in the thermal coefficients of expansion of the new and the old concretes. As a result, there seem to be no serious problems at the interface due to the concretes' expanding differently and inducing additional tensile stresses.

Direct Shear Test

Since the cores from the inner lane had a mean strength of 186 psi and those obtained from outer lanes had a mean strength of 220 psi, it could be

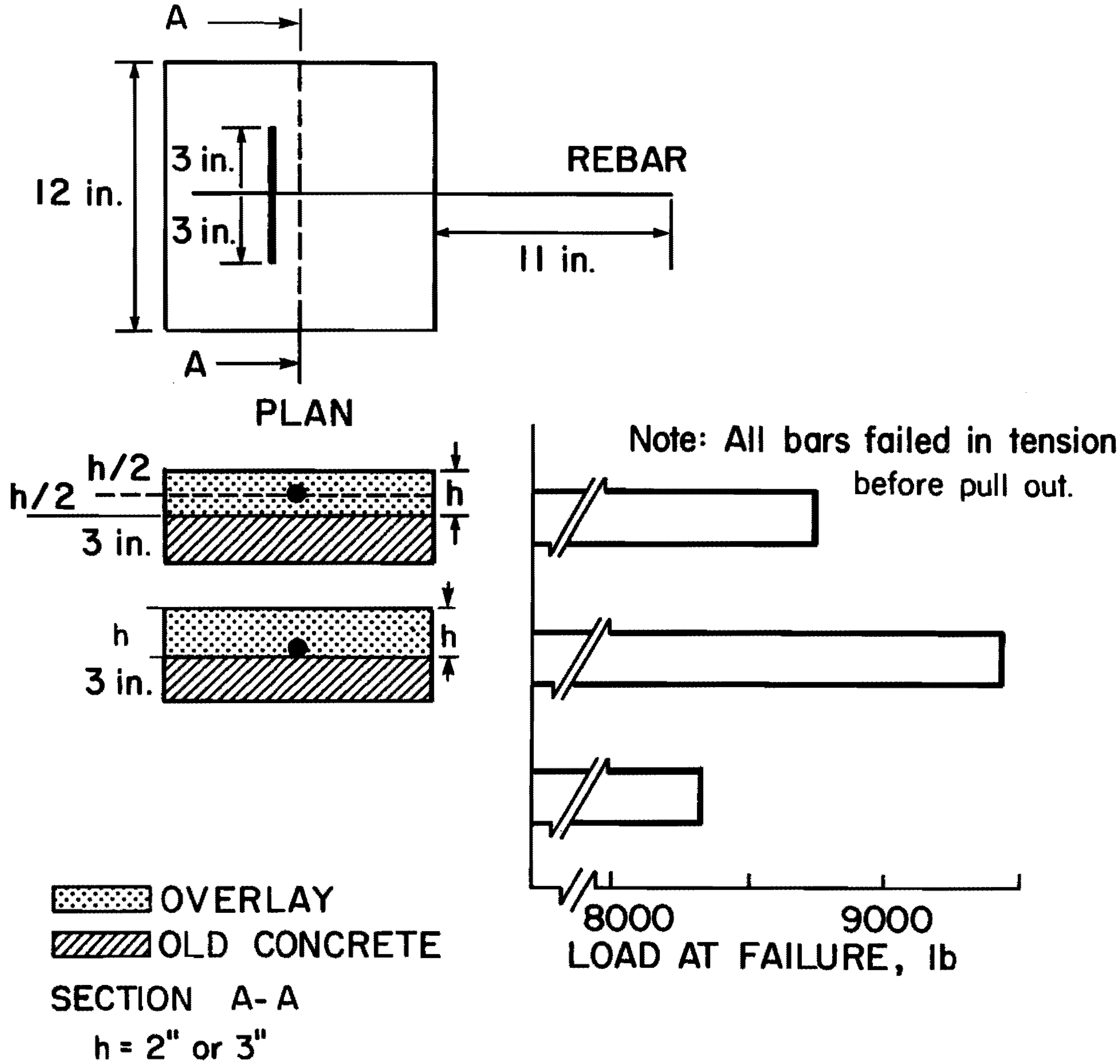


Fig 6.2. Bond pull out test.

concluded that lane location does not have a significant effect on the shear strength at the interface. Figure 6.3 shows the mean shear strength obtained under the different conditions.

Since the cores with grout and Daraweld-C as a bonding agent had a mean shear strength of 135 psi, and those with no grout at the interface had a strength of 213 psi, it may be concluded that the dry surface without grout during dry weather conditions results in to higher shear strength at the interface. This result agrees with the laboratory findings that the dry surface with no grout at the interface gave rise to higher shear strength at the interface.

The lowest shear strength obtained was 79 psi and as against the worst shear stress expected at the interface of 24 psi, giving a factor of safety greater than 3.

From the results, it could be inferred that the performance of the plain concrete overlay was better than most of the reinforced concrete overlay. Further experiments must be performed to verify whether the contact area of reinforcement affects the bonding of an overlay.

SPLITTING TENSILE STRENGTH

The cores with plain concrete exhibited the highest tensile strength followed by the fibrous and the reinforced concrete cores in that order (see Table A.8). There was a full-depth crack in core number 26. This may be the reason it gave low strengths. The samples from a cracked core of fibrous concrete gave a higher strength. This may be because the fibers were oriented in different directions. Normal concrete may be weaker in tension in a direction normal to the crack.

Due to the improved performance of fibrous concrete, further tests should be conducted with repeated loadings to determine how the fibrous concrete behaves in fatigue. If this could be proved to be a viable alternative to reinforced concrete or a plain concrete overlay, the considerable labor cost involved in placing reinforcement mesh could be easily saved.

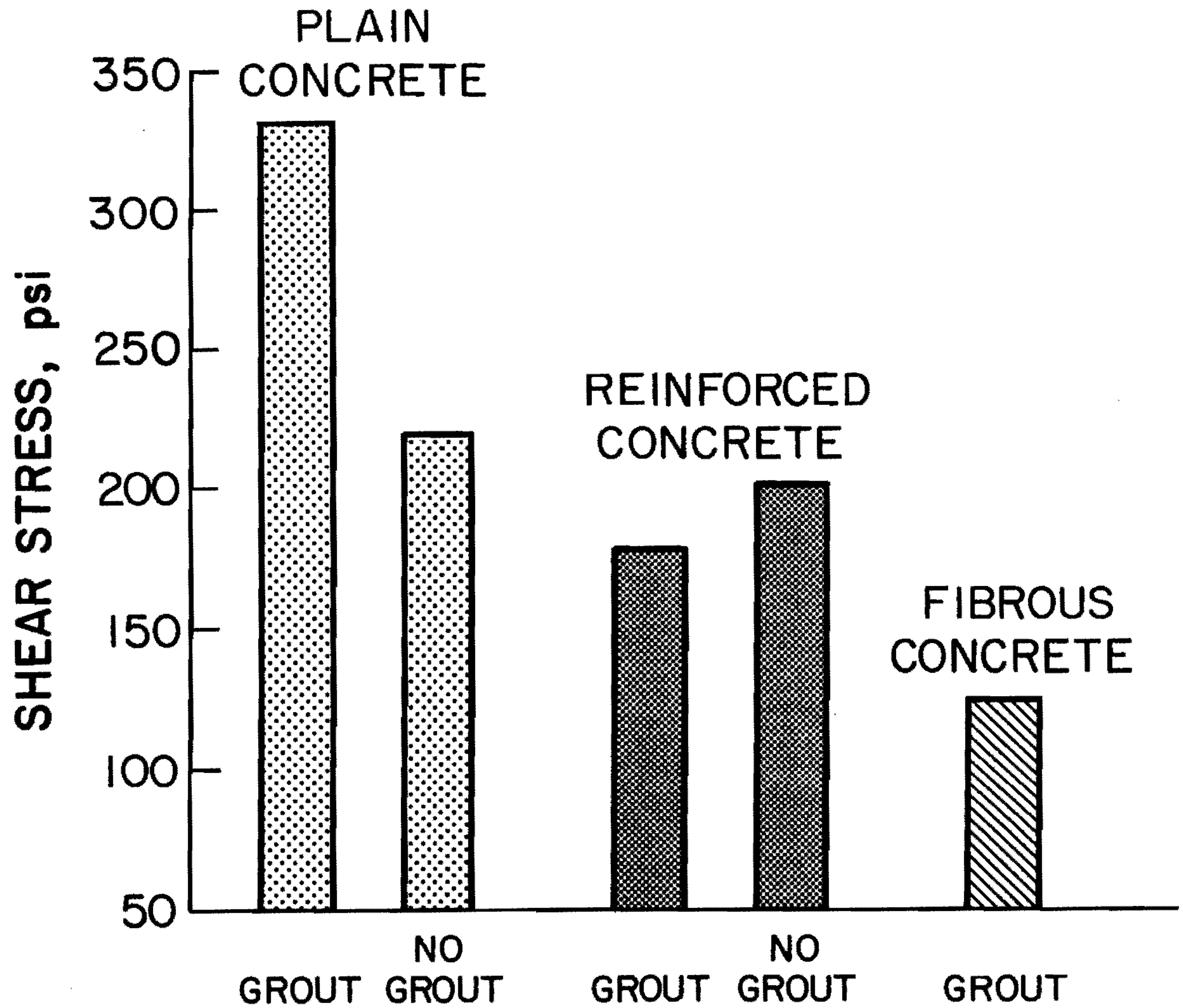


Fig 6.3. Field cores.

The tensile strength of concrete seems to increase with depth. All samples tested showed this. In the region immediately below the interface, the average tensile strength was found to be 472 psi, and the tensile strength for the bottom most layers was found to be 550 psi. This may be due to the fact that bottom layers receive more compaction and are consolidated more, by the weight of the upper layers. The mean tensile strengths of the different types of concrete are given in Fig 6.4. The results of this experiment are given in Table A.6.

The tensile strength of the RC overlay was found to be about 16 percent less than existing concrete at a depth of 8 inches. The tensile strength of fibrous concrete overlay was found to be about 10 percent less than the existing concrete at a depth of 8 inches.

These results suggest that fibrous concrete is likely to perform better as an overlay than reinforced concrete at least in the initial stages. Further experiments must be conducted to determine whether these differences will decrease with age.

SUMMARY

The summary of the results is given in this section.

(1) The performance of epoxy was affected by the wet concrete. Possibly the epoxy hardened before the overlay was placed, thus affecting its bonding capacity.

(2) Cement grout proved to be a better bonding agent than the epoxy.

(3) The bonding at the interior was better than at the edge and corner.

(4) The grouted interface may withstand thermal cycling effects better than the epoxy at the interface.

(5) A grouted surface gave a reduced possibility of failure due to a smaller coefficient of variation of the shear stress at interface.

(6) The least shear strength obtained at the laboratories was much greater than the worst shear at the interface based on theoretical predictions.

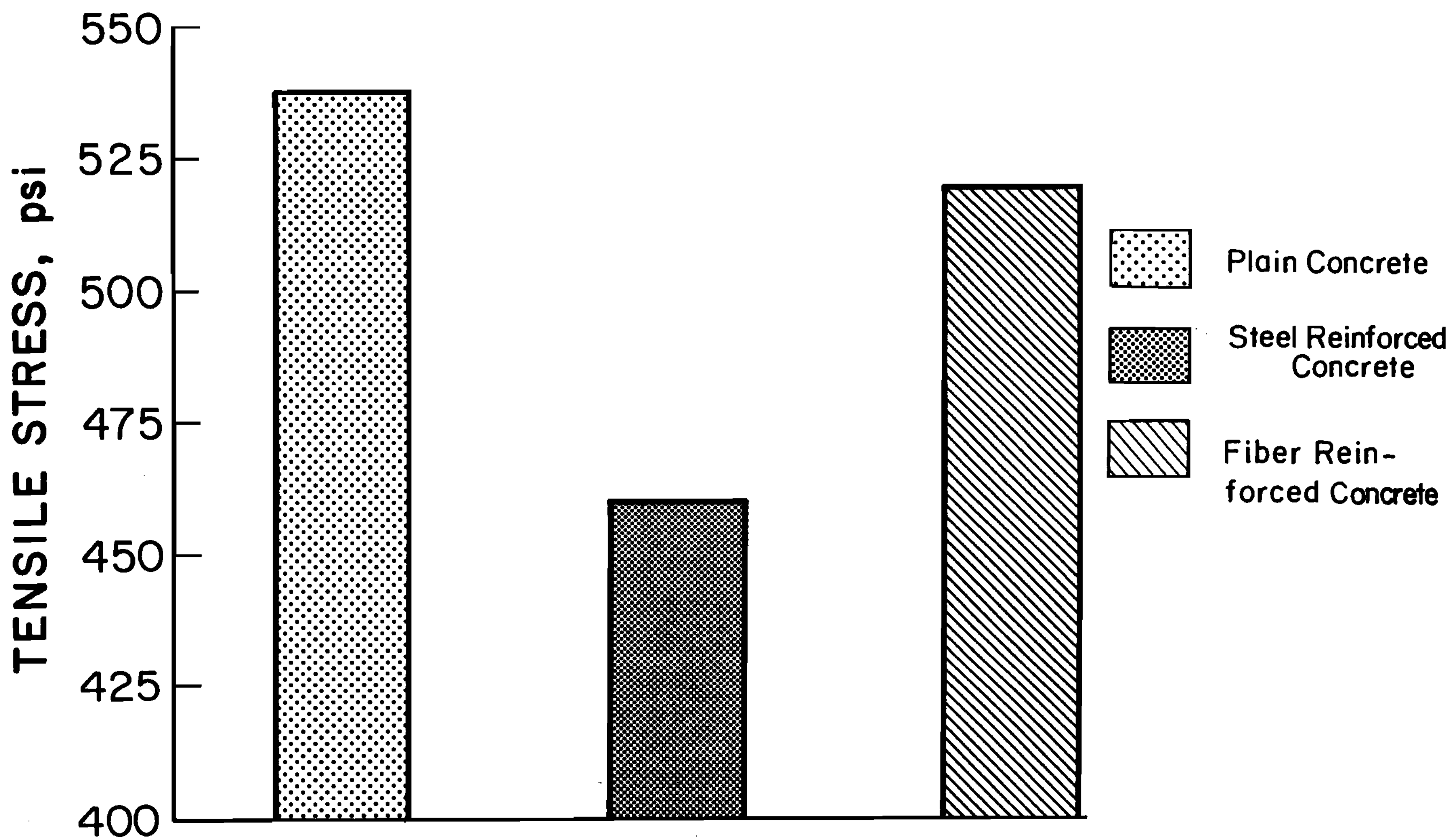


Fig 6.4. Splitting tensile test on field cores.

(7) The dry interface gave a higher shear stress at the interface.

(8) High temperature placement did not seem to have a greater effect on the shear stress at the interface.

(9) If the overlay is being placed during dry weather conditions, the grout can be omitted, but, if the overlay is being placed during wet weather conditions, it is better to have a layer of grout.

(10) The location of rebars, i.e., at the original surface or at mid-depth of the overlay, was not critical as far as bond at interface is concerned. Hence, the bar can be placed at the surface.

(11) The coefficient of thermal expansion of old concrete was found to be $5.333 \times 10^{-6}/°F$. There was no significant difference in the coefficients of thermal expansion of new and old concretes.

(12) When no bonding agent was used, the shear strength obtained when overlaid on a dry surface was higher than that obtained when overlaid on a wet surface.

(13) The performance of plain concrete seemed to be better than that of reinforced concrete overlay. Further experiments must be conducted to verify whether the contact area of reinforcement affects the bonding of overlay.

(14) The most critical shear strength obtained was 300 percent more than the most critical shear stress obtained through computer analysis.

(15) Plain concrete exhibited the highest tensile strength and the reinforced concrete the lowest at the interface.

(16) Fibrous concrete had a higher tensile stress than the reinforced concrete.

(17) The tensile strength of overlay and that of the concrete at 12 inches from the resurfaced layer were closer in value in the case of fibrous concrete reinforced concrete, which indicates that a fibrous concrete overlay is more likely to behave as a monolithic unit than a reinforced concrete overlay.

CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

The conclusions that emerge from the several tests and the theoretical analysis described in previous chapters are presented here. These conclusions are based on limited tests and apply only when the conditions of these tests regarding the materials used, environmental aspects and design specifications are met. After presenting the conclusions, a set of recommendations is given.

CONCLUSIONS

(1) Adding an overlay to an aged pavement improves the structural quality of the pavement as measured by the reduction in deflection both at cracks and at mid-span positions.

(2) Overlaying on dry surface results in better bond strength at the interface than overlaying on a wet surface. Specifically, overlaying when the surface is wet results in the weakest interface bond strength. Under this condition, it is advisable to apply a grouting agent or to dry the surface before overlaying. If the surface is dry, there is no need for using a grouting agent.

(3) Roughening the surface as by milling or scarifying helps produce a better bond.

(4) The effect of positioning overlay reinforcements on interface bond strength is insignificant. Hence reinforcing bars can be placed on the original surface in the interests of cost saving.

(5) Fiber reinforced overlay is a good alternative to plain or bar reinforced PCC overlay.

(6) The experience with TBCO as gained during this research is encouraging, leading to the conclusion that TBCO appears to be a feasible alternative to other methods of resurfacing.

RECOMMENDATIONS

This study has answered some questions, but there are many more questions of interest to pavement designers, constructors and administrators. It is suggested that the following studies be taken to understand the scope and limitations of TBCO as a rehabilitation resurfacing strategy:

(1) the experimental section should be monitored for a length of time to gain an understanding of the performance behavior of TBCO;

(2) more experiments should be designed and analyzed to know the relative merits of:

   (a) various types of coarse aggregates,

   (b) different grouting agents,

   (c) surface preparation techniques,

   (d) using TBCO at different stages of decay of the existing pavement,

   (e) using TCCO under different environmental, traffic and support conditions, and

   (f) relative degrees of dryness vis-a-vis grouting agent usage.

(3) relative economics of using alternatives need to be investigated, e.g., is it cost effective to dry a wet pavement (before overlaying) than to apply a grouting agent?

(4) the effects of temperature gradient, such as due to different layers being at different levels from surface, needs to be investigated.

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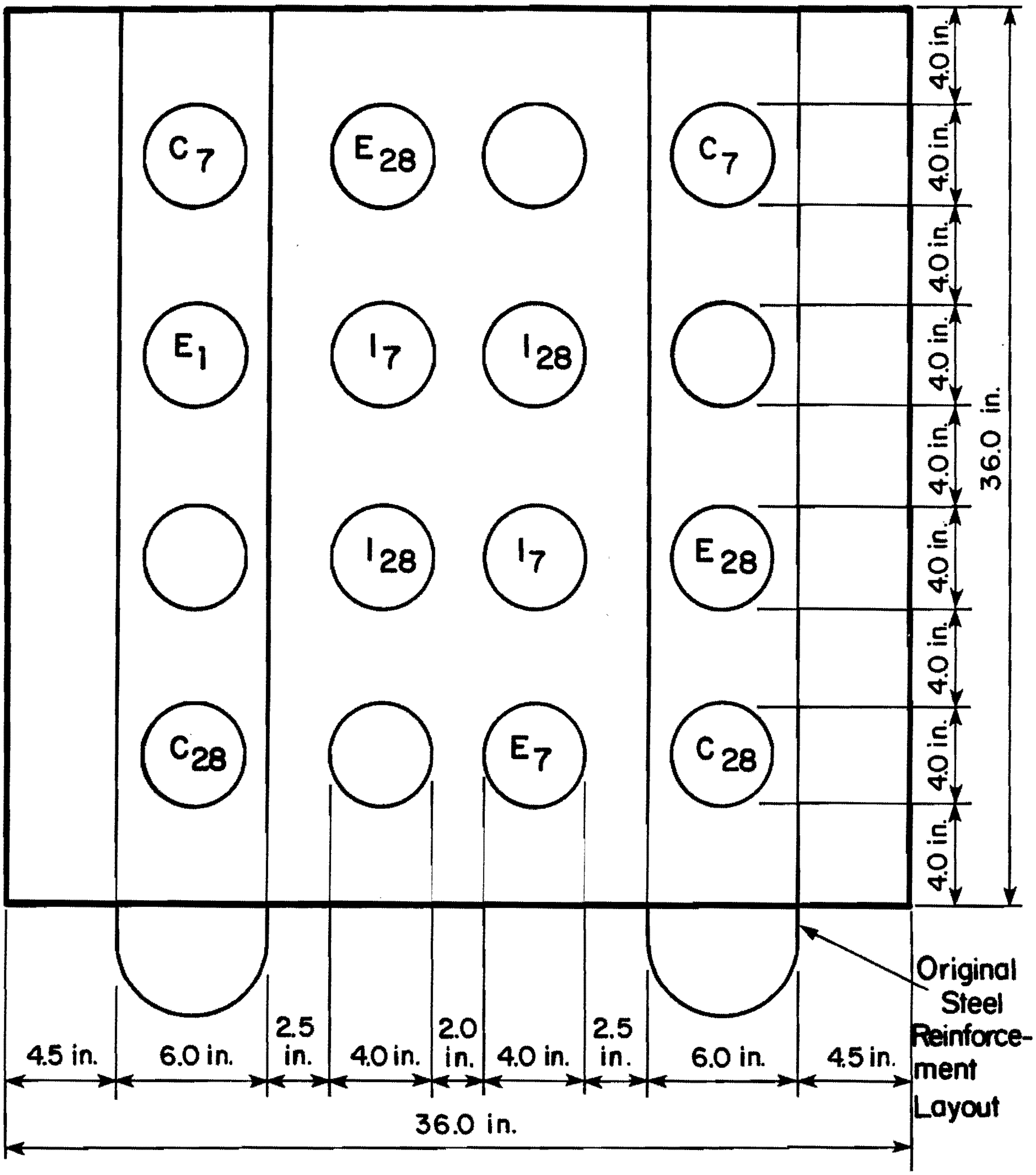
APPENDIX A



Legend:

C: Corner

E: Edge

I : Interior

Subscripts : Curing Period in Days.

Fig A.1. Locations of cores.

004 27

TABLE A.1. RESULTS OF SHEAR STRESS OF CORES OBTAINED FROM SLAB (28 DAYS)

Code: SB: Sand Blasted
G: Grout (cement)
R: Room Temperature
C: Cycled (cold to hot temperature)

Serial No.	Identification Code	Load at Failure (lb)	Shear Strength (psi)
1	SBGRI	1775	141
2	SBGRE	2275	181
3	SBGCI	3675	293
4	SBGCE	2025	161
5	SBGCC	2250	179
6	SBGRI	3925	312
7	SBGCC	4025	320
8	SBGCI	3600	287
9	SBGCC	1700	135
10	SBGRE	1450	115
11	SBGRE	2250	179
12	SBGRC	1950	155
13	SBGRC	2000	159
14	SBGCE	2475	197
15	SBGRI	4075	324
16	SBGRC	1650	131

TABLE A.2. RESULTS OF CYLINDER OVERLAY EXPERIMENT I

Serial No.	Surface Treatment	Load at Failure (lb)	Diameter (in.)	Shear Stress (psi)
1	No Grout	2800	4.002	231
2	No Grout	3275	4.025	261
3	No Grout	2400	4.001	131
4	Grouted	2250	4.003	213
5	Grouted	2800	4.001	228
6	Grouted	2650	4.002	211

TABLE A.4. SHEAR TEST ON CORES

Core No	Core Diameter	Area 2 (inches)	Load at Failures (lb)	Shear Strength (psi)
1	4.006	12.609	3200	253.79
2	4.004	12.597	saved	saved
3	4.004	12.597	saved	saved
4	4.015	12.666	3050	240.80
5	4.015	12.666	4400	347.39
6	4.011	12.641	4500	355.98
7	4.011	12.641	5150	407.40
8	4.013	12.653	3000	237.10
9	4.013	12.653	3650	288.47
10	4.011	12.641	1000	79.11
11	4.000	12.571	2800	222.73
12	4.006	12.609	3800	301.37
13	4.021	12.703	3025	238.13
14	4.014	12.659	1050	82.94
15	4.016	12.672	saved	saved
16	4.020	12.697	2100	165.39
17	4.010	12.634	3100	247.37
18	4.015	12.672	1800	142.05
19	4.014	12.659	saved	saved
20	4.008	12.622	1450	114.88
21	4.023	12.717	saved	saved
22	4.012	12.647	1400	110.70
23	4.010	12.634	1200	94.98
24	4.019	12.691	1650	130.01
25	4.015	12.666	2600	205.27
*26	4.010	12.634	1200	94.98
27	4.012	12.647	2100	166.05
28	4.045	12.672	2500	197.29
29	4.008	12.622	saved	saved

*Indicates a core with crack.

TABLE A.3. CYLINDER OVERLAY EXPERIMENT II: SHEAR STRESS AT FAILURE (PSI)

Surface Condition	Temperature	Bonding Agent: Grouted (psi)	Bonding Agent: Non Grouted (psi)
Dry	High	356 (1)	245 (2)
		266 (3)	190 (6)
		246 (11)	293 (7)
		$\bar{x}$ = 289	$\bar{x}$ = 243
Dry	Room	295 (15)	253 (12)
		322 (18)	155 (13)
		230 (21)	242 (14)
		$\bar{x}$ = 282	$\bar{x}$ = 242
Wet	High	222 (4)	142 (8)
		280 (5)	111 (9)
		313 (10)	
		$\bar{x}$ = 272	$\bar{x}$ = 127
Wet	Room	271 (17)	265 (19)
		264 (16)	126 (20)
		$\bar{x}$ = 268	$\bar{x}$ = 196

004 19 Figures in parentheses indicate core numbers.

TALBE A.5. SHEAR STRENGTH (PSI): FACTORIAL REPRESENTATION OF SHEAR STRESS RESULTS OF FIELD CORES

		Overlay Type					
		Plain Concrete		Reinforced Concrete		Fibrous Concrete	
Bonding Agent	Thickness (inches)	Inside	Outside	Inside	Outside	Inside	Outside
Grout and Daraweld	2	238 (13)	347 (5)	356 (6)	165 (16)	111 (22)	247 (17)
		---	407 (7)	233 (11)	---	95 (26)	166 (27)
		---	---	301 (12)	---	---	---
		---	---	83 (14)	---	---	---
Grout and Darsweld	3	---	---	79 (10)	115 (20)	130 (24)	---
		---	---	---	95 (23)	---	---
No Grout	2	254 (1)	---	241 (4)	---	---	---
		205 (25)	---	237 (8)	---	---	---
		197 (28)	---	142 (18)	---	---	---

Note: Figures within brackets are core numbers.

TABLE A.6. RESULTS OF THE INDIRECT TENSILE STRESS OF THE CORES

Sl No.	Core No.	Plain Concrete A	Plain Concrete B	Plain Concrete C	Plain Concrete D	Reinforced Concrete A	Reinforced Concrete B	Reinforced Concrete C	Reinforced Concrete D	Fibrous Concrete A	Fibrous Concrete B	Fibrous Concrete C	Fibrous Concrete D
1	2						543						
2	5			423	550								
3	7			405	647								
4	8							480					
5	9					553		511	439				
6	10					467	442	532					
7	12					408		443	686				
8	14						501	631	480				
9	17									673			538
10	18						430		525				
11	20					403	478						
12	22												575
13	24									343	411	541	701
14	25	543			416								
15	26									*491	*360	*347	
16	27									437	483	369	506
17	29									654	548		

Note: A: overlay; B: 0" to 2" from interface; C; 2" to 6" from interface; D: 6" to 8" from interface.

*Indicates a core with cracks.

TABLE A.7. BOND PULL OUT TEST

Rebar Size	Sl No	2-in. Overlay	3-in. Overlay	Mid-depth	Load at Failure (lb)
#3	1			X	9,040
#3	2	X			9,000
#2	3		X		7,900
#2	4			X	8,000
#3	5	X			10,380
#3	6		X		9,300
#2	7		X		7,800
#3	8	X			9,000
#2	9			X	8,000
#2	10			X	8,080
#3	11			X	10,400
#3	12			X	10,440

TABLE A.8. THERMAL COEFFICIENT OF CONCRETE CORES

Core Number	Temperature (°F)	Distance, "DE"	Thermal Coefficient of Expansion
24	40 70 92 118	6.0018 6.0024 6.0033 6.0042	5.2×10^{-6}
23	40 70 90 118	6.0015 6.0028 6.0036 6.0048	7.0×10^{-6}
10	40 70 84 118	6.0843 6.0852 6.0857 6.0071	5.9×10^{-6}
24	37 60 77 100	6.0014 6.0019 6.0023 6.0030	4.2×10^{-6}
23	37 60 74 100	6.0805 6.0806 6.0810 6.0825	5.3×10^{-6}
10	37 60 74 100	6.0842 6.0849 6.0854 6.0866	6.5×10^{-6}

Mean thermal coefficient of expansion = $5.3 \times 10^{-6}/°F$

TABLE A.9. CALCULATED MAXIMUM SHEAR STRESSES (PSI) AT THE INTERFACE

Overlay Thickness (in.)	Existing Pavement			
	CRCP (8 in.)		JRCP (10 in.)	
	Strong	Weak	Strong	Weak
2	21	24	18	20
4	18	16	18	16

Note: * Grand mean, N = 19 psi
** Standard deviation, σ = 3 psi

TABLE A.10. SUMMARY OF CONSTRUCTION SCHEDULE

Out Lane 4 / Lane 3	In Lane 2 / Lane 1	
Date: 8/15/83 Slump: Min 2.75" Max 4.50" Mean 3.9" Test: 730 psi 798 psi Core #: 7, 19, 28, 1, 5, 25, 16*, 9* Interface:	Date: 7/22/83 Slump: Min 3.25" Max 4.0" Mean 3.75" Test: 955 psi Core #: 13 Interface	2" NRC
Date: 8/26/83 Slump: Min 2.50" Max 4.50" Mean 3.61" Test: 840 psi Core #: 20, 23 Interface	Date: 7/26/83 Slump: Min 3.0" Max 6.0" Mean 4.3" Test: 878 psi Core #: 6, 8, 11, 12, 14, 15, 18, 21 Interface	2" RC
Date: 8/27/83 Slump: Min 3.75" Max 6.00" Mean 4.8" Test: (1) 838 psi (2) 898 psi Core #: 3, 17, 27, 29 Interface	Date: 7/28/83 Slump: Min 2.5" Max 6.5" Mean 5.1" Test: 992 psi Core #: 2, 4, 10 Interface	3" RC
	Date: 8/1/83 Slump: Min 3.75" Max 8.0" Mean 5.2" Test: 870 psi Core #: 24 Interface	3" FC
	Date: 8/3/83 Slump: Min 3.0" Max 5.25" Mean 4.5" Test: 920 psi Core #: 26 Interface	2" FC