SHRP-C-373

Optimization of Highway Concrete Technology

David Whiting, Mohamad Nagi, Paul Okamoto

Construction Technology Laboratories, Inc.
5420 Old Orchard Road
Skokie, Illinois

Tom Yu, David Peshkin, Kurt Smith, Michael Darter

ERES Consultants, Inc.
8 Dunlap Court
Savoy, Illinois

James Clifton, Lawrence Kaetzel

National Institute of Standards and Technology
Gaithersburg, Maryland

Strategic Highway Research Program
National Research Council
Washington, DC 1994

SHRP-C-373
Contract C-206
ISBN 0-309-05751-5
Product Numbers 2027, 2028

Program Manager: Don M. Harriott
Project Manager: Inam Jawed
Program Area Secretary: Carina S. Hreib
Copyeditors: Sally Baird
Susan Neill
Production Editor: Katharyn L. Bine

January 1994

key words:
aggregate
bridges
durability
curing
expert system
fatigue
microstructure
nondestructive testing
overlays
pavements
portland cement concrete
rehabilitation
repair
strength

Strategic Highway Research Program
2101 Constitution Avenue N.W.
Washington, DC 20418

(202) 334-3774

1.5M/NAP/194

Acknowledgments

The research described herein was supported by the Strategic Highway Research Program (SHRP). SHRP is a unit of the National Research Council that was authorized by section 128 of the Surface Transportation and Uniform Relocation Assistance Act of 1987.

The authors acknowledge the assistance of the Georgia Department of Transportation, the Ohio Department of Transportation, the Kentucky Transportation Cabinet, and the Missouri Highway and Transportation Department in making field sites available for this study and assisting in collection of traffic data and field testing; Prof. Donald J. Janssen, University of Washington, who performed all T 161 B freeze-thaw testing for specimens prepared at the field test sites; and Ms. Sally Baird and Ms. Susan Neill of Walcoff & Associates for the editing of the document.

Contents

List of Figures

List of Tables

Abstract

This report provides (1) state-of-the-art information on new developments in portland cement concrete materials and applications to users in the highway agencies, especially young engineers just beginning their careers in the field; and (2) new materials and testing methodologies for a variety of important highway and bridge rehabilitation applications. The report includes a condensation of the state-of-the-art synthesis of concrete highway technology prepared as part of this research project. Descriptions of evaluations of test methods for in-place concrete density with the use of nuclear methods and determination of water content of fresh concrete using microwave drying are also incorporated in the report. The report gives summaries of guidelines for avoiding thermal effects in concrete pavement slab placements and packing-based aggregate proportioning for concrete mixtures. The report contains a detailed description of field evaluations of a variety of concrete mixes used for early opening of full depth concrete pavement repairs and bridge deck overlays. Opening times ranged from 2 to 24 hours. Brief descriptions of the methodology and content of the HWYCON expert system, available from SHRP, are also included. Finally, a description of the contents of audiovisual implementation packages for highway personnel dealing with materials testing and pavement and bridge rehabilitation is included.

Executive Summary

The performance of portland cement concrete (PCC) pavement and structures is dependent on the design of the structure; the composition and quality of concrete materials; the care with which the concrete is placed, consolidated, finished and cured; and the environment in which it is placed. Many members of the post-World War II generation of highway engineers, who have great practical knowledge and experience in these areas, have retired. The next generation of engineers needs to learn now about concrete materials technology to produce quality concrete pavements and structures. Design aspects are covered in detail in most modern university curricula, but discussions of materials, properties, and performance are often left to on-the-job training. The objective of this project was to present the new engineer with information about portland cement concrete and its constituent materials and how they relate to concrete repair and rehabilitation. New types of cements and mixtures were studied for their applicability to facilitate the early opening of concrete pavements (one of the greatest problems facing today's engineers). Information regarding this and other topics, such as diagnosis of distress, testing of aggregates and concrete, proper design of concrete mixes, and repair and rehabilitation, was compiled and assembled into computer-accessible expert systems, educational videotapes, and training manuals. This report alerts the reader to the existence of these training aids.

State-of-the-Art Concrete

As part of this project, a state-of-the-art synthesis has been prepared. Entitled *Synthesis of Current and Projected Concrete Highway Technology* (Whiting et al. 1993), the synthesis relates that, in recent years, significant developments have taken place in the availability of materials that can be utilized in the production of concrete. The simple mixture of cement, aggregate, and water also may include combinations of special cements, chemical admixtures, mineral admixtures, special aggregates, and fibers. Air-entraining agents are generally used to develop air-void systems appropriate for durability requirements. Chemical admixtures may be used to increase compressive strength, control rate of hardening, accelerate strength, improve workability, and improve durability. Mineral admixtures such as fly ash and silica fume have been used to produce stronger concretes and concretes with less permeability. New cements capable of very rapid strength gain and enhanced durability have recently been introduced, and conventional cements can be treated with a variety of admixtures and mixtures designed in such a manner to obtain equivalent high performance.

As a result, highway engineers are increasingly using these materials to enhance concrete properties for new construction, repair, and rehabilitation.

The continued development and evaluation of the concrete technology discussed above will have an enormous impact on the concrete paving industry. Engineers must be able to build concrete pavements rapidly, open them to traffic soon after construction, and to rely on such to be long-lasting designs. With these advances, concrete will become a more attractive material for use not only in new design but also for rehabilitation. Because the work on the interstate and primary system has shifted from new construction to rehabilitation, use of concrete for rehabilitation is crucial. Specific areas of rehabilitation addressed in this synthesis and which will be positively influenced by the advancement of concrete technology include

- reconstruction
- full-depth repairs/slab replacements
- partial-depth repairs
- concrete overlays (including bridge deck overlays)
- concrete recycling.

Current practices and recommendations for improvements in each of these areas are described in the synthesis. The need for early opening of repaired pavements made possible through the use of special materials and concretes with high rates of early strength gain is stressed. These rapid-setting concretes will allow rapid repair and rehabilitation of pavement and bridge components and improve the long-term performance of concrete against adverse exposure conditions and increased magnitude and frequency of traffic loading.

In addition to selection and design of repair applications, constructibility is another important issue addressed in the synthesis. Procedures for mixing and batching are described, and the prospect of more highly automated systems is noted. The use of slipform pavers is described, along with characteristics of equipment used to place, consolidate, and finish the slab. Dowel insertion and zero-clearance pavers are being used more frequently, especially in heavy traffic areas. Final treatment of the pavement, including proper curing and texturing, is important if the highway is to achieve a long-lasting and safe riding surface.

The synthesis addresses other developments in job site testing and quality control. This includes measurement of concrete water, chloride, and air-void content as well as maturity monitoring and other methods for determining in-place strength gain. New methods for measurement of concrete density and voids are discussed. Finally, a chapter on statistical quality assurance describes acceptance plans that account for variability in both testing and materials and allow for adjustment of pay factors based on contractor performance.

Development of Test Methods for Fresh Concrete

Under this project, four areas dealing with control methods for fresh concrete and mix optimization were investigated. These included (1) evaluation of a technique for field determination of the water content of fresh concrete with the use of microwave drying, (2) development of a guide for prediction of temperature effects in newly placed concrete slabs, (3) development of a handbook for use in optimizing the proportioning of aggregates for concrete mixes, and (4) development of a technique for measuring the density of fresh concrete at various depths within a freshly placed concrete slab.

The development of a technique for determination of the total water content of fresh concrete was completed under C-206. This involves the drying of fresh concrete for up to 14 minutes in a 900-watt microwave oven. It is necessary to remove the test sample at intervals from the oven to allow for hand crushing and distribution of the sample in the drying pan. The test uses a 1.5-kg test sample and can be carried out by relatively inexperienced personnel after only a brief period of training. Single-operator precision was $\pm$ 2.7 lb/yd^3 ($\pm$ 1.6 kg/m^3). The method has been applied to conventional paving concretes, latex-modified and silica fume concretes for bridge overlays, and concretes used for rapid opening of pavement repairs, all with good results. A proposed standard American Association of State Highway and Transportation Officials (AASHTO) test procedure is appended to this report.

The effects of temperature and moisture early in the life of concrete strongly influence early strength development and long-term durability. A guide developed under C-206 (Andersen, Andersen, and Whiting 1992) demonstrates how knowledge of concrete temperature, air temperature, and concrete mix characteristics allows the user to determine whether temperature-induced problems may be expected. Guidelines for avoiding such problems are also included.

The guide includes a series of tables for Types I, II, and III cements as well as for concretes containing Class F fly ash. The tables can be used to predict temperature effects for concrete mixes containing between 525 and 750 lb/yd^3 (312 and 445 kg/m^3) of cement in the mix. The tables were prepared from the output of a computer simulation of heat flow and strength development. The simulation was carried out during the first 72 hours after placement. The following parameters must be entered for the tables to be used: type of cementitious material, content of cementitious material, concrete temperature, air temperature, and thickness of concrete pavement. Examples of the use of the tables for various combinations of parameters are included in the guide.

The proportioning of sand to coarse aggregate has been found to have an important effect on the properties of both fresh and hardened concretes. Produced under C-206, *A Guide to Determining the Optimal Gradation of Concrete Aggregates* (Andersen and Johansen 1993) describes the application of particle packing theory to the production of densely packed aggregate skeletons in a concrete mixture. The guide states that workability of concrete will be maximized at the point of maximum packing of aggregate particles, as at this point the amount of cement paste will be minimized. The improved aggregate gradation and reduced paste content can reduce occurrences of segregation and bleeding and lead to a more durable concrete. This will also lead to a more economical concrete mixture.

This guide consists of tables relating characteristics of the aggregate particle size distribution and the packing density to the packing of two or more different materials. The tables can be used to efficiently blend coarse aggregate fractions or mixtures of sand and coarse aggregate.

The manual includes a description of test methods and procedures, instructions on use of the tables, and a summary of the theoretical background. A limited amount of work was executed using the packing tables compared with more standard methods of aggregate proportioning. While some promising results were obtained, a more comprehensive evaluation is warranted before the tables can be recommended for widespread use.

A commercially available twin-probe nuclear density gage (Troxler model 2376) was evaluated for applications where information on density and degree of consolidation as a function of depth in pavement slabs is desired. The gage utilizes a Cs-137, 5-mCi (0.19-GBq) source; a scintillation tube detector; and a scaler/ratemeter to obtain the transmitted counts. A small, 1-μCi (37-kBq) secondary source was mounted by the manufacturer on the detector to stabilize the Cs-137 peak during testing. The source and detector are positioned 12 in. (305 mm) apart in 2-in. (50-mm)-diameter guide tubes designed with cone points to allow penetration of the fresh concrete. A calibration curve of counts versus density was developed by testing of metallic standards supplied with the gage as well as with a series of concrete standards ranging from 70 to 150 lb/ft^3 (1,120 to 2,400 kg/m^3).

The gage was tested in a series of void-free concretes and found to have an average accuracy of approximately $\pm$ 1.2 lb/ft^3 (19 kg/m^3). When applied to specimens containing steel dowel bars, the gage was able to accurately determine the density of successive layers of concrete to within 0.8 in. (20 mm) of the dowel bar. When the path of the gage came closer to the dowel bar, the recorded density greatly increased. Opposite effects were seen with simulated voids in the concrete, the density decreasing greatly as the gage path came close to the void areas. A proposed standard AASHTO test procedure is included as an appendix to this report.

Also included is a summary of other products developed in the SHRP Concrete and Structures Program. A brief description of the product and its utility are given for each of the products selected. The products treated include new tests for concrete permeability, fluorescent optical microscopy, methods of detection of alkali-silica reaction (ASR) products in concrete, new tests for determining the reactivity of aggregates, methods for designing ASR-safe concrete mixtures, identification of aggregates susceptible to d-cracking, a modified AASHTO T 161 freeze-thaw procedure, methods of measurement of damping constant of concrete, the pulse-echo technique for flaw detection, and high-performance concrete mixes. Listed for each of the products are references to supporting documentation and the SHRP product number to be used in conjunction with the 1992 *SHRP Product Catalog*.

Field Studies of New Test Procedures, Materials, and Criteria for Concrete Rehabilitation

Field studies were carried out to evaluate concrete materials and mixtures that show great promise for use in early opening applications and where concrete is subject to extremely adverse environments. The applications chosen included full-depth pavement repairs, where traffic must be permitted to pass onto the pavement in a relatively short time, and bridge deck overlays, where early opening would aid in reducing the need for extended bridge closures.

Concrete materials and mixtures for these studies were chosen on the basis of development carried out in other SHRP studies, independent research and development programs, and industry participation. These included Very Early Strength (VES) and High Early Strength (HES) mixes developed under SHRP C-205 (*Mechanical Behavior of High-Performance Concrete*), Fast-Track concrete mixtures developed originally by the Iowa Department of Transportation (DOT), and mixtures currently being utilized by the Georgia and Ohio DOTs. All of these mixes utilized AASHTO M 85 cements and locally available materials as well as admixtures meeting AASHTO M 194 requirements. Mixtures utilizing the proprietary cement materials Pyrament and Rapid-Set cement were also included in the research program because these materials are becoming more widely available.

Demonstration sites were selected in the states of Georgia and Ohio for early opening of full-depth pavement repairs. The basic design of the repairs was to place ten repairs of the same size with each of the materials being tested at each of the two sites selected for the experiment. The Georgia site was located on a 3-mile (4.8-km) stretch of eastbound Interstate 20 (I-20) west of Augusta. The Ohio site was located on a 4-mile (6.4-km) stretch of east and westbound SR 2 east of Vermilion. Three early opening mixes (VES, Fast-Track, and a Georgia DOT mix) were evaluated at the Georgia site on a total of 60 replacement sections, making 20 repairs with each mix. Eight early opening mixtures (two Pyrament cement, two Rapid-Set cement, HES, VES, Fast-Track, and an Ohio DOT mix) were evaluated on a total of 80 replacement sections at the Ohio site.

Prior to commencement of field work, laboratory trial batches were prepared with materials shipped from the field sites. These batches were used to establish early-age correlations between such parameters as maturity, compressive strength, and modulus of rupture. Relationships between pulse-velocity and strength were also developed. During the actual field placements, these relationships were used by the investigators to track the strength development in the slabs and aid in determining time of opening. Predicted strengths were verified by on-site testing of insulated test cylinders, cylinders cured by temperature-matched curing, and cores removed from the test slabs. Opening time ranges included 2 to 4 hours, 4 to 6 hours, and 12 to 24 hours, depending upon the particular materials and mixtures being used. In most cases, measured compressive strengths in excess of 2,000 psi (13.8 MPa) and moduli of rupture (MOR) exceeding 300 psi (2.1 MPa) were achieved within the target opening times.

The assessment of the effects of early opening of pavement slabs was made on the basis of fatigue analyses using Miner's approach. The effect of concrete strength at the opening on dowel bearing stress was also examined. Traffic count and distribution, truck weights, and traffic wander data were collected at the pavement field sites. These were used to compute pass-coverage ratios (p/c) needed for the fatigue analysis. The fatigue damage accumulated during the first 14 days of service after opening to traffic was considered the fatigue damage attributable to early opening in this study. Both load- and temperature-induced stresses were considered.

Analyses were conducted using laboratory test results and the data collected during installation to estimate the fatigue damage attributable to early opening and to determine the strength needed at opening to assure acceptable performance. From an applications perspective, the determination of strength needed for opening to traffic was the most important aspect of the experiment. On the basis of results, it appears that the strengths required for early opening to traffic—a flexural strength (MOR) of 300 psi (2.1 MPa) or a compressive strength of 2,000 psi (13.8 MPa)—are reasonable under most conditions for repairs up to 12 ft (3.7 m) in length. The compressive strength is a more accurate indicator of early opening for very short sections—6 ft (1.8 m) or less—because dowel bearing stress is the critical factor in these cases. Through long-term monitoring of test sections, the performance data that are needed to develop more precise guidelines for early opening may be obtained.

The field studies demonstrated that considerable care needs to be exercised if successful early openings are to be achieved. Little margin for error exists in working with many of these materials, and any delays may impact adversely the ability of workers to adequately place, consolidate, and finish the concrete. Working times are short (often less than 45 minutes) and concrete haul times should be minimized. The large amount of cement in these mixes causes more generation of heat and more rapid loss of workability than in conventional concretes. Many of the mixes are sensitive to relatively small changes in amount of water batched. Successful openings can be achieved, in some cases within 2 to 4 hours; however, personnel must be made aware of the special handling characteristics of these concretes prior to their use on any particular job.

Bridge deck overlay applications were also investigated. Latex-modified concretes (LMC) utilizing Type III cement and silica fume concretes were placed at four separate locations. Two bridges were selected in Ohio on I-270 in Columbus and US 52 southeast of Cincinnati. Two additional bridges were selected in Kentucky on I-265 over KY 22 east of Louisville and on US 41 over KY 351 in Henderson. At each site, both LMC Type III and silica fume concrete placements were made on separate lanes of twin structures. Results showed that, using Type III cement, LMC can be opened to traffic within 24 hours of placement. Cold conditions (ambient temperature less than 50°F [10°C]) may require longer curing periods. While silica fume concrete (using Type I cement) requires longer curing times, strengths were significantly higher, and permeability to chloride ions was lower than for LMC Type III mixes.

Follow-up surveys were conducted at both bridge deck and pavement sites. At bridge sites, both visual surveys of deck condition and sounding of deck surfaces for hollow areas were carried out. Results indicated that overlays were generally sound, with distress being identified at only one of the four overlay sites. In this instance, some minor cracking had occurred on both the latex-modified and silica fume sections as well as some apparent disbonding of overlay material near the construction joint between two overlay placements.

At pavement repair sites, follow-up testing consisted of visual distress surveys, coring, and falling weight deflectometer (FWD) measurements. The visual distress survey of the Georgia sections showed no cracking or other obvious signs of distress after 2 months of service. Core strengths ranged from 10% to 20% lower than 28-day lab-cured cylinder strengths.

Only limited microcracking was detected in a core removed from one of the sections. At the Ohio site, a significant number of repairs were found to exhibit longitudinal cracks. All cracking occurred at midslab, with the major contributing factor appearing to be high temperature gradients generated in the period after placement, which most likely induced significant curling stresses in the slabs. Extensive microcracking was noted in cores removed from many of these sections, indicative of early tensile stresses which may have been induced by the high temperatures developed in the slabs.

Expert Systems

Expert systems for cement and concrete applications have experienced limited success since their introduction in the 1980s. These systems can be characterized as requiring long development times, being based on limited knowledge and having only limited resources available to develop them. More recently, however, computer software tools and hardware have provided both enhanced resources for the design or integration of different knowledge forms and improved user interfaces, resulting in reduced development time. This new environment, along with the need to synthesize vast quantities of cement and concrete knowledge, will lead to a greater acceptance of expert systems and will play an increasingly important role in decision making for highway activities.

In this project, an expert system was developed to assist state highway departments in (1) identifying and determining the cause of materials-related distresses; (2) selecting materials to avoid such problems as alkali-aggregate reactivity, freeze-thaw action, corrosion of reinforcing steel, and sulfate attack; (3) selecting materials for recycled concrete, permeable bases, and Fast-Track concrete; and (4) selecting materials for full-depth and partial-depth repair, bonded and unbonded overlays, and diamond grinding and milling. Structures addressed in the system include jointed and continuously reinforced pavements, bridge decks, and other highway structures (e.g., columns, piers, and parapet walls). The target audience for the computerized knowledge systems includes inspectors and engineers, concrete specifiers, and decision makers whose knowledge about the activities ranges from novice to mid-level. Operation of the system involves input from the user, who describes field observations of distressed structures or specifications for concrete design. The expert system then gives conclusions about the cause(s) of the problem and makes recommendations on the design or method that best fits the description. The system also makes recommendations pertaining to tests and laboratory procedures to confirm conclusions and recommendations.

The computerized system requires an IBM-compatible desktop or portable personal computer. Software requirements include MS-DOS and Microsoft Windows. The system is distributed in a run-only format that allows the user to operate the system. The system is designed using an object-oriented architecture which makes the system conducive to the addition of new knowledge and permits modifications of the existing knowledge and operational features. Various forms of knowledge are used in the system. They include (1) facts and rules of thumb, (2) pictures and drawings, (3) explanatory information, and (4) bibliographic information. A document describing the installation and providing an overview of the system was developed, along with a comprehensive reference manual describing the conceptual design, knowledge structure, and procedures for modifying the computerized system.

Implementation Packages for Highway Personnel

A set of implementation packages was prepared to allow for transfer of many of the products developed under the SHRP concrete research program to state highway department personnel. The SHRP program was funded not only to find solutions to many of the problems being encountered in highway engineering and maintenance but also to convey this information to the practitioners in state and local highway agencies quickly and in a form that will be readily understood and applied. The research team selected some of the most important problems and appropriate solutions, and these are presented to the highway engineering community in a series of implementation packages that are practical and understandable.

A total of four implementation packages were produced. Each package consisted of the following materials: (1) educational videotapes utilizing professional narration and produced at actual construction sites or laboratory facilities (in the cases where laboratory procedures were being demonstrated), totaling twelve tapes in all; (2) for each tape, a parallel set of 35-mm slides with a moderator's outline for use in those cases where video facilities might not be available or for large audiences where video monitors would not be readily visible; (3) illustrated users' manuals that detail the equipment, materials, and procedures covered by the tapes (the manuals are intended to be used as reference materials that would include details not appropriate to the video presentations); (4) instructors' guides for each session so the packages can be used later by personnel not immediately involved with initial development of the materials; and (5) where appropriate, pocket guides that can be used in the field that list the materials and equipment, testing requirements, and construction methods necessary to carry out the particular applications and testing being discussed.

Package 1, *Durability Testing of Concrete and Aggregates*, illustrates the various new methods that can be used to evaluate the durability of concretes and aggregates and therefore enable more selectivity in choice of materials by state agencies. While concentrating on alkali-silica reactivity and freeze-thaw testing—the primary emphases of the SHRP concrete program—the users' manual contains additional information on other areas of concrete durability such as corrosion, including testing for chloride content and sulfate attack.

Package 2, *Quality Control of Concrete On-Site*, includes both traditional methods of control, such as proper sampling and use of temperature, slump, and air content test procedures, and new methods developed by SHRP and others. The latter include rapid determination of concrete water content using microwave drying; use of *A Guide to Evaluating Thermal Effects in Concrete Pavements* (Andersen, Andersen, and Whiting 1992); use of maturity, pulse velocity testing, and temperature-matched curing to estimate in-place strength gain; and use of the twin-probe nuclear density gage to locate areas of incomplete consolidation in concrete pavements.

Package 3, *Rehabilitation of Highway Concrete*, consists of five videotapes covering a number of important procedures used to repair concrete pavements and bridges. The topics covered include partial-depth spall repairs, full-depth joint repairs and full slab replacements, bonded and unbonded concrete pavement overlays, and rigid concrete bridge deck overlays. The availability of rapid-setting concrete materials, which make concrete a cost-competitive material and allow rapid opening of repair sections for these types of repairs, is emphasized. The tapes include detailed action sequences of all the repair techniques.

Package 4, *Early Opening of Full-Depth Concrete Repairs*, is designed to address a topic that has garnered a wide interest in recent years, i.e., early opening of concrete pavement repairs. With the use of early opening technology, concrete mixes can be designed and curing techniques used that will promote development of required opening strengths at intervals ranging from 24 hours down to 6 hours or less. As a result, concrete can now be used for the repair of projects for which it would not have been used in the past because of the lengthy cure times needed. The most efficient methods for concrete removal and dowel bar insertion into adjacent slab faces are discussed. Scheduling so that all activities are completed without delays is emphasized. The criteria for opening and the use of in-place test methods for determination of opening times are also included in the video.

1

Introduction

Needs

The goal of the concrete research of the Strategic Highway Research Program (SHRP) was to develop the means needed to produce more durable concrete for a variety of highway applications. This research consisted of six projects: C-201, *Concrete Microstructure*; C-202, *Eliminating or Minimizing Alkali-Silica Reactivity*; C-203, *Resistance of Concrete to Freezing and Thawing*; C-204, *Non-Destructive Testing for Quality Control/Condition Analysis of Concrete*; C-205, *Mechanical Behavior of High Performance Concrete*; and C-206, *Optimization of Highway Concrete Technology*. These new materials, test methods, and procedures will allow highway practitioners to produce concrete that meets the challenges of increased traffic loadings and severe environments experienced by today's highways and the highways of the future.

Significant changes in materials and testing methodology occurring in the concrete and highway fields require validation before they are ready to be implemented into daily practice. These changes include the use of materials such as silica fume, latex modifiers, rapid-setting cements, high-performance concretes, and a variety of chemical admixtures, among others. Test methods for determination of the components of fresh concrete, rate of strength gain, susceptibility to cracking, consolidation, and other properties have been developed under SHRP and other programs but are in need of field validation.

Perhaps the most important need created by the large amount of information generated under SHRP is that of a system of information exchange that can be used to transmit the information to those who will ultimately use the new materials and procedures in their daily work. The information must be synthesized and made available in formats compatible with modern technology. Computer-assisted knowledge bases offer a means for access to a large amount of information in a rapid manner. Media such as videotape and slide presentations allow highly technical subjects to be portrayed in an easy-to-understand format. In the project being described in this report, all of these approaches were utilized to develop a series of implementation packages that can be used to transmit the latest in concrete highway technology to highway agencies most in need of this information.

Objectives and Approach

The following set of objectives and scope were set as the goal for this project.

Objective: Synthesize existing information on current and newly developed materials and practices for highway construction and present these in a state-of-the-art document to be made available by SHRP. Scope: This information was obtained from ongoing SHRP projects as well as from literature sources and results of other research studies in the highway field.

Objective: Evaluate and validate newly developed test procedures for determining critical properties of both fresh and hardened concretes. Scope: The procedures having the most promise based on preliminary results obtained in SHRP C-204 were selected for further evaluation. Procedures that can be used in actual field applications were given priority for this study.

Objective: Evaluate and validate these procedures and newly developed materials and concrete mixtures in actual field installations. Scope: On the basis of information developed during compilation of the synthesis document, these installations were chosen as representative of the most commonly employed rehabilitation techniques that utilize portland cement concrete materials.

Objective: Develop a methodology for diagnosis of concrete problems, selection of materials, and repair applications that can be rapidly accessed by users in highway agencies. Scope: A computer-based format, such as a computer-assisted knowledge base or expert system, was seen as highly desirable, considering the increasing use of computers in almost all aspects of modern technology. An expert system shell, suitable for use on personal computers, was used to develop this product.

Objective: Transmit new developments in the concrete field to users in highway agencies in an easily accessible and understandable format. Scope: A set of implementation packages was produced covering the latest developments in concrete materials testing, field quality control practices, rehabilitation of highways and bridge decks, and early opening of highway repairs. These packages heavily utilized videotape and slide presentation format and were designed to be practical and understandable for persons at all levels of highway agencies.

2

State-of-the-Art Concrete Highway Technology

Needs

The performance of portland cement concrete (PCC) pavement and structures is dependent on the design of the structure; composition and quality of the concrete materials; the care with which the concrete pavement is placed, consolidated, and finished; the proper curing of the pavement; and the surrounding environment. It must also be recognized that specifications on many construction materials do not completely define the composition or performance of any given materials when utilized in concrete. Careful selection and testing of materials at the local level is necessary to assure performance under the conditions to be expected during actual construction. The retirement of many members of the post-World War II generation of highway engineers having great practical knowledge and experience in these areas has created a need for rapid education of the next generation in many aspects of concrete materials technology, which are important for production of quality concrete highways and structures. While design aspects are covered in detail in most modern university curricula, materials, properties, and performance are often overlooked and left to on-the-job training. The objective of the synthesis developed under SHRP C-206 and entitled *Synthesis of Current and Projected Concrete Highway Technology* (Whiting et al. 1993) was to afford the new engineer with a primer covering many aspects of portland cement concrete, its constituent materials, and its applications to important areas of concrete repair and rehabilitation. This chapter consists of what is, in essence, a digest of the synthesis. Those wishing to review the various areas covered by the synthesis in more detail are strongly urged to obtain the original publication. Numerous references to original work in the various areas covered by the synthesis can also be found in the publication.

It is recommended that highway agencies obtain multiple copies of the synthesis and utilize them in training of both new and existing personnel in areas where a sound knowledge of concrete technology is required.

Cements

Cements meeting AASHTO M 85 (ASTM C 150) are still the most widely used binder materials in PCC. Types I and II cement are most widely available and can be obtained in low-alkali versions where alkali-aggregate reactivity is a concern. The chemical composition and physical characteristics of cement can affect many important performance parameters, including placeability, strength gain, drying shrinkage, permeability, resistance to sulfates, and corrosion of reinforcing steel. Although tests are carried out on the finished cement in an attempt to control its production, the relationship between cement and concrete performance is still not wholly quantified. This is especially true when admixtures are used, making the preparation of trial batches an important prerequisite in the start-up of any new concrete project. The performance of admixtures is known to be influenced by the aluminate and alkali contents of the cement, its fineness, and the amount and type of sulfate compounds added to regulate set.

Cements other than Types I and II are available, though utilized to a much smaller extent, such as when high early strengths are desired (Type III) or where high resistance to sulfate salts is needed (Type V). Cement can be blended with mineral admixtures to produce portland pozzolan or portland blast-furnace slag cements. Use of these materials can reduce mix water requirements, improve workability, decrease permeability, inhibit alkali-aggregate reaction, and reduce heat generated in large sections or in hot weather. In the United States, however, concrete producers prefer to add these pozzolans at the batch plant to maintain control over the percentage of the material added to, or used as replacement for, the cement. This separate addition also allows for an increased economic advantage, as most pozzolanic materials are generally less expensive than portland cement.

Cements to be used for more specialized purposes are also available. These include expansive cement (generally available as Type K cement or shrinkage-compensating cement), which contains hydraulic cement compounds that expand during hydration and thus can compensate for volume shrinkage generally associated with portland cement. Expansive cements have been used with apparent success in both highway and airfield construction, although to a relatively limited extent.

Special cements also include those capable of very rapid strength gain. Typically, strengths of ordinary portland cement are not adequate for structural purposes within the first 24 hours when placed and cured under ambient conditions. Cements such as high-alumina cements, magnesium phosphate cements, regulated set cements, gypsum-based cements, sulfo-aluminate-based cements, and blended pozzolanic cements using alkali-activated alumino-silicates are available and have been used in a variety of applications where it is necessary to open a pavement or structure within a very short period of time. A number of these are marketed as proprietary products. More research and testing is needed on these cements, however, especially as regards the influence of early age service on long-term durability and performance.

Aggregates

Often wrongly described as the inert filler in a concrete mixture, aggregates compose 70 to 80% of the volume of a typical concrete mix and are an important component of the overall material. Properties such as size, gradation, and shape of aggregates have an important influence on water demand, workability, strength, and durability of concrete. Gradation is one of the most important aggregate characteristics affecting its performance. Too many fines or a shortage of material on one or more sieve sizes can lead to a poor distribution, requiring excess water for placement that may result in bleeding and segregation of the mix. By aggregate beneficiation or by blending two or more separate gradings of aggregate, some of these deficiencies may be overcome. Tables based on particle packing theory (Andersen and Johansen 1993) may aid in obtaining an optimal blend of aggregates with a minimum void content and maximum workability. It must be recognized that experience with local materials is necessary for final proportioning of field concrete mixtures.

Aggregates also contribute to abrasion resistance and friction of concrete surfaces. The latter is especially important for pavement and bridge deck applications, and aggregates that consistently provide good friction are preferred. Friction of the coarse aggregate is especially important after wear is sufficient to expose the coarse particles in the surface of the pavement.

There is an increasing shortage, however, of high-quality, durable aggregates (ASTM 1976). This can be attributed to stringent acceptance requirements, zoning restrictions on aggregate production operations, pollution control regulations, and shipping expense. Aggregate resources can be conserved and extended through the use of marginal aggregates, which still meet the desired performance requirements, beneficiation of low-quality aggregates, adjustment of specifications (so as to emphasize performance rather than prescription), use of waste materials such as phosphate slimes or mine tailings, and production of aggregates from recycled concrete. Such materials as slags and municipal waste are being studied as possible alternate sources of aggregate. Even with traditional naturally occurring aggregates, problems such as D-cracking and alkali-silica reactivity remain serious concerns. Such problems indicated a need for new tests designed to detect sources susceptible to these processes. Where alternate sources cannot be located, methods for utilization of susceptible aggregates are being developed. These include the use of pozzolans and chemical inhibitors for avoidance of alkali-silica reactivity and reduction of maximum size of aggregates susceptible to D-cracking.

Admixtures

Admixtures are defined as any substances other than water, cement, aggregates, or fibers that are added to a concrete batch immediately before or during mixing. Admixtures can be used to accelerate or retard setting time, to reduce water content and improve strength, to increase slump, or to reduce cement content. Admixtures can enhance finishability and make concrete more manageable under difficult conditions. Admixtures can increase resistance to freezing and thawing, inhibit alkali-aggregate reactivity, improve resistance to sulfates, increase resistance of reinforcing steel to corrosion, reduce heat generated during curing, and make possible the placement of concrete under very hot or cold conditions.

Chemical admixtures are widely used and may be present in more than 80% of the concrete placed today. If air-entraining agents are included in the total, it is likely that close to 100% of the concrete in this country contains one or more admixtures. Air-entraining agents impart a system of tiny microscopic bubbles to the concrete, thus allowing pressures generated upon freezing to be relieved. The amount and stability of the entrained air system is a function of the fineness and alkali content of the cement, the concrete mix design, means of placement and consolidation, and the presence of other types of admixtures in the mix. Other types of chemical admixtures include accelerators, retarders, water reducers, and superplasticizers (or high-range water reducers). These aid in the placement of concrete in cold or hot weather and impart a longer working life to concrete under a variety of conditions.

Mineral admixtures, though slowly gaining acceptance in the highway industry, are now being increasingly used to meet energy and waste disposal requirements. The most common mineral admixtures are by-product substances such as fly ash and ground-granulated blast furnace slag (GGBFS). Fly ash can be and is used extensively as a partial replacement for cement in levels generally up to 25% of the cement content of the mix. In addition to its economic benefits, a high-quality fly ash can improve concrete workability, reduce bleeding, reduce heat generation and permeability, and contribute to long-term strength gain. GGBFS can be used at even higher replacement levels and offers similar benefits. Silica fume is a pozzolanic material that is being increasingly used in concrete, primarily for the production of very high strength (more than 10,000 psi [69 MPa]) and very low permeability concretes such as those required for thin bridge deck overlays. Other finely divided materials such as boiler slags, municipal wastes, and cement kiln dust (CKD) are being investigated for future use in concrete.

Concrete Production

Even the best materials will fail if incorporated into a concrete mixture in an improper manner or if the concrete is subsequently incorrectly mixed or transported. Rational mix design procedures, which rely on proportioning of concrete to meet the demands of placeability, strength, and durability, offer the best means of achieving desired performance. Mix designs that rely on fixed amounts of cement and other materials for various classes of concrete are not flexible enough to meet many of today's changing demands. The ACI 211 mix proportioning procedures (ACI 1992c) are widely accepted and, except for various refinements having to do with more exact definitions of aggregate gradation and relative proportions of fine and coarse materials, should serve the industry well into the future. Alternate cementitious materials such as fly ash and GGBFS are increasingly used and are included in recent proportioning procedures. Computer-aided mix design procedures and expert systems in this area are expected to become widely available in the years ahead.

Accuracy of batching and mixing is necessary for achieving consistent concrete properties. Cement and aggregates should be weigh batched, and water and admixtures should be batched by weight or volume. Procedures and equipment leading to improved consistency in production include individual batching, pneumatic gate control, oscillatory gates, belt conveyor discharge into the weigh hoppers, and pressurized admixture dispensers. Maintaining the integrity of separate storage facilities for all fine powders used in the batch (for instance, cement and fly ash) is also important if serious errors are to be avoided. Future trends include the use of automated controls and materials selection programs, which automatically discharge the proper amount of material for the requested concrete mix. While central mixing facilities are preferred for increased uniformity on large jobs, truck mixing can be used advantageously, provided care is taken in loading the truck and proper mixing procedures are followed.

Highway Pavement Construction

Although some pavements are still placed using fixed forming procedures, the majority of projects now follow slipform paving practice. Concrete is centrally mixed and delivered by dump truck or ready-mix truck, with front-discharge trucks being preferred where access is restricted. The auger distributors meter out a proper head of concrete across the entire width of pavement. As the paving machine advances, a battery of vibrators enters the discharged concrete and consolidates it. Oscillating extrusion finishers pass over the consolidated concrete and extrude it at the proper shape. The Clary screed and pan float then follow for final finishing. This highly automated process leads to smooth and consistent pavement production.

Dowel bars at joints are placed either by using preassemblies fixed to the surface or by a component of the paving machine. The automatic dowel bar inserters (DBIs), introduced in recent years, have considerable advantage in eliminating the labor and effort required by conventional assemblies. Such automated methods have also been applied to tie bars and to continuously reinforced concrete pavements. Other improvements in the paving process include the use of zero-clearance pavers, which allow the paver to be confined to a single lane while traffic can flow freely on adjacent lanes. Final magnesium finishers also improve the quality of the surface by automatically closing the surface immediately behind the pan. Computers have had an impact upon the paving industry in the form of pavers designed to automatically control alignment and profile.

To provide the required friction, pavements must be textured. This can be carried out either on fresh or hardened concrete. Transverse tining using spring steel tines is normally carried out on freshly placed concrete. This can be combined with longitudinal artificial turf or burlap texturing. The drag precedes the tining operation. Texturing of hardened concrete can be carried out on new pavements but is more typically applied to existing pavement as a rehabilitative procedure. It can be carried out by diamond grinding, grooving, sandblasting, or water blasting. Sawed transverse grooving appears to offer the best frictional performance.
The final step, curing of concrete, includes maintenance of sufficient internal moisture to allow the cement to hydrate so as to reach its intended strength as well as to protect concrete from either overheating or freezing. White-pigmented curing compound is the best choice for long-line paving jobs, although other techniques such as soaked burlap or fogging can be used on smaller placements such as bridge decks, overlays, and pavement repairs. Thermal effects

can be offset by careful control of concrete temperature at placement (Andersen, Andersen, and Whiting 1992). Additionally, such effects may be reduced by using lower cement contents and cement replacement materials, such as fly ash, that lower the heat generated by concrete during curing. Under cold weather conditions, insulating blankets or heated concrete may be required to prevent freezing and allow concrete to obtain the desired level of strength (ACI 1992a).

Applications of Concrete Highway Technology

The continued development and evolution of concrete technology will have an enormous impact on the concrete paving industry. Concrete pavements must be able to be rapidly constructed, to be opened to traffic soon after construction, and to be a reliable, long-lasting design alternative. With these advances, concrete will become a more attractive material for use not only in new construction but also in rehabilitation. Because the work on the Interstate and primary system has shifted from new construction to rehabilitation, the ability to use concrete in the rehabilitation area is crucial. Specific areas of rehabilitation that will be positively influenced by the advancement of concrete technology include the following areas.

Reconstruction

For badly deteriorated pavement sections, total reconstruction is generally the preferred rehabilitation option. However, it is generally desirable to maintain traffic flow as much as possible to minimize disruptions, particularly in urban areas. To maintain traffic flow during reconstruction, so-called Fast-Track technology has been developed. Concretes can be placed, cured, and opened to traffic in as few as 4 to 6 hours. These mixtures utilize high early strength cements, high cement contents, and low water-cement ratios (w/c) to accelerate the setting process. Curing blankets can be used to maintain heat of hydration in the slabs and further contribute to early strength gain. This technology has been successfully applied to intersections, access roads, and airport runways, where it is difficult (or impossible) to detour traffic for more than a very short time.

Full-Depth Repairs/Slab Replacements

Where deterioration is confined to joints or other well-defined portions of the pavement and where the concrete pavement is exhibiting relatively low amounts (less than 10 to 20%) of severe joint spalling or slab cracking, the most cost-effective method of rehabilitation is full-depth repair or slab replacement. In these cases, early opening may also be called for, especially on heavily trafficked roads where only a few sections are being repaired and detours would lead to extreme congestion of the traffic flow. Repairs with early opening mixes can be made in 2–4 hours using either accelerated versions of the early opening mixes previously noted or mixes using specialty cements capable of achieving complete cure in as few as 2 hours. The premium paid for such materials is often offset by the need for reduced traffic control costs as well as the reduced inconvenience to the driving public.

Partial-Depth Repairs

Partial-depth concrete repairs are performed to address shallow joint spalls. Too often, however, these spalls are patched with asphaltic materials due to their rapid placement and ability to be opened to traffic immediately. However, these materials commonly have limited durability and short service lives. Therefore, it is important that reliable and more durable cementitious materials and procedures be developed that can be placed quickly and opened to traffic as soon as possible. Such materials have been evaluated in the SHRP program, and recommendations for their use will soon be available (Good Mojab et al. 1993).

Concrete Overlays

Both bonded and unbonded concrete overlays are seeing more application as rehabilitation alternatives. Bonded concrete overlays are intended to provide additional pavement structure to concrete pavements with little structural deterioration; unbonded concrete overlays are constructed on deteriorated concrete pavements but use a separation layer (bondbreaker) to prevent the underlying pavement distress from reflecting through the new pavement. The bonded systems require a much more distress-free substrate and therefore more extensive preoverlay repairs. However, such systems may be more applicable where clearances present a problem and can be completed more rapidly, especially where rapid substrate preparation methods such as cold-milling and shotblasting are employed. Concrete overlays are also an important method for rehabilitation of deteriorated bridge decks, where durable and impermeable materials using such admixtures as latex emulsions and silica fume are a necessity.

Concrete Recycling

The emphasis on reuse of waste materials and the need to protect the environment have led to a renewed interest in concrete recycling. Recent advances in pavement removal and processing equipment make it possible to produce recycled aggregates from deteriorated pavements. Recycled concrete pavement can be used for subbases, bases, or shoulders, or it can be used as aggregate in new concrete. Significant savings can be achieved through the use of recycled concrete, particularly in urban areas, where disposal costs may be high. By beneficiation of recycled aggregate, by using smaller top sizes, or by blending with virgin natural aggregates, almost any concrete pavement can be reused. Recycled aggregate is now being used in many instances where normal aggregate would formerly have been used in pavement reconstruction projects.

Other Developments

This synthesis also addresses new developments in job site testing and quality control of concretes. These areas have been sorely neglected over the years, with only slump, air content, and compressive and flexural strengths normally emphasized as concrete control parameters. With the use of new technology, it is possible to measure water and cement content of concrete on site, determine chloride content of materials, and determine in-place density. Such methods as maturity monitoring and in-place strength testing can be used to monitor rate of strength development by essentially nondestructive means. Voids and other defects in hardened concrete can be located by such methods as penetrating radar and impact-echo testing.

Finally, although implementation has so far been very slow, there is a gradual progression away from so-called recipe methods of specification and towards performance-related specifications coupled with application of statistical quality assurance schemes (SQA). In SQA, acceptance plans are developed that account for the variability in both materials and testing. Both agencies and contractors can reach agreement on the particular scheme to be used based on trade-offs between project cost and acceptable risks to both parties. The pay factors can be coupled to contractor performance, and bonuses may also be applied. The advantages of performance specifications and SQA are twofold. First, they recognize that it is the final product that is of importance to the public. Second, while variabilities are inherent in materials and construction, these variables can be dealt with in a logical and equitable manner based on sound statistical principles rather than rigid compliance to unrealistic prescriptions, which are often idealized and not implementable in practice.

This synthesis concludes with an appendix summarizing developments in highway technology in European countries. During the past few years, this country has come to realize (often by learning painful lessons) that we are indeed part of a global economy and that neither our practices nor our policies can be isolated from those of other areas of the world. While economic, technical, social, and environmental conditions are indeed different in other countries, there is much to be learned from the experiences of our counterparts abroad. This appendix should be of interest to all in the highway field.

3

Development of Test Methods for Fresh Concrete

Current Status and Needs

Job site testing of concrete has, for the most part, changed little during a number of generations. Although interpretation of compressive strength testing has been codified in ACI 214 (ACI 1992b), the basis of acceptance of concrete still remains the compressive strength cylinder or flexural beam cured for a specified time under largely artificial conditions. Before the concrete hardens, the bases for acceptance remain the slump test (AASHTO T 119) and air content (AASHTO T 152 and T 196), where appropriate. These tests are simple to perform and well established throughout the industry; however, complete reliance on these tests alone represents a failure to recognize some important contributors to concrete performance. With respect to fresh concrete, it must be recognized that the slump test, as a simple measure of consistency, does not afford a measure of the potential of a concrete to reach its specified strength or its intended durability. Slump is a poor measure for prediction of strength variations of concrete from load to load on a large job (Shilstone 1988). Mix proportions, especially the ratio of water to cementitious components, have a large impact on ultimate performance, and the ability to measure these properties in fresh concrete would allow the screening of concretes deficient in the proportions that were specified. Knowledge of the true water-cement ratio (w/c) of the fresh concrete shortly after delivery, for instance, would enable a more accurate prediction of strength and durability to be made. Consolidation of concrete can also have a dramatic influence on strength and other mechanical properties (Whiting et al. 1987). A reliable method for measuring consolidation of fresh concrete throughout the depth of a pavement slab would allow revibration of poorly consolidated areas prior to hardening of the concrete.

Research in these areas was initiated in SHRP C-204, Nondestructive Testing for Quality Control/Condition Analysis of Concrete, but efforts were terminated before useable products could be developed. Work in these areas was continued under SHRP C-206, and field procedures that are described in this chapter were developed. Additionally, under

SHRP C-201, Concrete Microstructure, bases for predicting temperature effects in freshly placed concrete slabs and for using packing theory to proportion concrete were developed. These efforts were also continued in C-206 with the aim of developing useable products for state quality control personnel.

Determination of Water Content of Fresh Concrete

Previous Work

Highway agencies and AASHTO officials have recognized the need for methods to measure the water and cement contents and w/c of concrete since the early 1970s. Several methods were developed and evaluated in the last 20 years (Whiting 1993). Unfortunately, none of these methods completely satisfied the needs of highway engineers and concrete users. Most are complex, and many are expensive. The main challenges for these methods are to be field applicable, simple, and rapid. Unfortunately, the existing techniques have several drawbacks, including the need for a skilled operator, overly long test times, and difficulties in application under field conditions. Although an ideal objective would be to simultaneously measure cement and water contents or at least measure them separately and then compute w/c, the SHRP C-204 investigators evaluated most of the available technology in this area and concluded that it is not possible at this time to develop a direct method for measuring w/c that would satisfy the requirements of such a technique. At the same time, they stated that, because the cement content is well controlled and batched in modern ready mix plants, developing a rapid and reliable method for measuring water content and then calculating the w/c by knowing the cement content would be sufficient, considering the complexity and expense of the existing field methods for determination of cement content. The C-206 investigators concurred with this conclusion, for most of the changes in w/c of as-delivered concrete are caused by the extra water that may be introduced into the concrete as wash water in the mixing truck, may be added by the truck driver as the mix stiffens, or may be derived from excessive water that might be held in the aggregate.

Various techniques have been investigated for determining water content of fresh concrete. These range from complex chemical methods (Kelly and Vail 1968) to simple hot-plate drying (Tom and Magoun 1986). The most promising technique appears to be one based on use of a microwave oven to rapidly dry the fresh concrete. This technique was first developed by the North Dakota State Highway Department (1978). It involved gravimetric determination of water content by boil-off in a microwave oven equipped with a defrost cycle. The defrost cycle was claimed to be needed to prevent overheating of the sample and fusion of the cement. To perform the test, a 1-kg sample was placed in a ceramic dish, weighed, and placed in the oven. The sample was then dried to constant weight, using the defrost mode for approximately 1 hour. Average error for a large number of mixtures was reported as approximately 3 lb/yd^3 (1.8 kg/m^3) at 95% confidence for an average water content of 250 lb/yd^3 (148 kg/m^3). Average standard deviation was reported as 1.25 lb/yd^3 (0.74 kg/m^3). This method was a direct determination of water and was inexpensive ($200 to

$500 for purchase of the oven). Unfortunately, it was relatively slow, requiring up to 60 minutes. What was still needed was a method that could yield a result within the same time period as that normally required to determine slump and air content of concrete. One inspector can normally make these latter determinations within 15 minutes.

A microwave oven with a ceramic ashing block assembly was used in a recent study (NCHRP 1990) on measurement of water content. A measurement was obtained within 10 minutes. The only problem with this approach was the limited size of sample required (500 g), which might be applicable for mortar but not for concrete containing coarse aggregate.

The SHRP C-204 investigators conducted a series of tests with the use of a commercial microwave oven with adjustable power settings (up to 1,400 watts) and with the use of different sample sizes (500; 1,000; 1,500; 2,000; 2,500; and 3,000 g). They concluded that sample size has a major effect on the accuracy of water content measurement and that drying a 1.5-kg sample in the microwave oven with a power setting of 900 watts for 10 minutes was the optimum. The objective of the follow-up study conducted under C-206 and described in this chapter was to further refine the C-204 technique and study its applicability to a variety of concretes under both laboratory and field conditions. Field results are described in Appendix A.

Objective

The objective of this research program was to evaluate the microwave oven method for measuring water content of as-delivered fresh concrete. The objective was to be achieved by gravimetric determination of the water contents of a series of concrete mixes prepared in the laboratory by drying the concrete in the microwave oven and comparing these measurements with the batch weights of water used in making these mixes. Field ruggedness of the procedures was verified by testing under actual field conditions at placements of bridge deck overlays and full-depth pavement repair sections (see Appendix A).

Concrete Materials and Mixes Evaluated

Materials selected for the laboratory evaluations consisted of the following: (1) a Type I cement from Lafarge Corp.; (2) a predominantly subangular natural sand consisting of a mixture of siliceous and calcareous minerals with specific gravity of 2.71, absorption of 0.80%, and fineness modulus (FM) of 3.01; (3) an angular to subangular quartzite coarse aggregate having a maximum topsize of 1 in. (25 mm), with specific gravity of 2.63 and absorption of 0.32%; (4) a subangular to subrounded fossiliferous siliceous limestone coarse aggregate having a maximum topsize of 1 in. (25 mm), with specific gravity of 2.45 and absorption of 3.82%; and (5) a crushed dolomitic limestone coarse aggregate having a maximum topsize of 1 in. (25 mm), with specific gravity of 2.70 and absorption of 1.40%. Admixtures included the following: (1) Force-10,000 silica fume (slurry form); (2) Daravair, a neutralized Vinsol resin air-entraining agent; and (3) WRDA-19, a Type F high-range water reducer. In addition, LMC mixes were produced using DPS Modifier A, a styrene-butadiene latex emulsion produced by Dow Chemical Co. Fly ash used was a Class F ash from the Indiana-Michigan Power Co., Rockport, Indiana plant.

Conventional concrete mixes with a cement content of 600 lb/yd^3 (356 kg/m^3) were used, with an initial water content of approximately 300 lb/yd^3 (178 kg/m^3). Water content was varied to obtain three general ranges of slump (low, moderate, and high). Three types of aggregates with low, moderate, and high water absorptions were used in this investigation to study the effect of aggregate absorption characteristics on the method.

Silica fume mixes, with 700 lb/yd^3 (415 kg/m^3) of cement and 70 lb/yd^3 (41.5 kg/m^3) of silica fume (solids basis) and the three types of aggregates, were investigated. In addition, aggregates with water absorptions of 0.32% and 3.82% were used in mixes incorporating fly ash and latex. In fly ash mixes, 120 lb/yd^3 (71 kg/m^3) of fly ash was used in combination with 480 lb/yd^3 (285 kg/m^3) of cement. In latex-modified concrete mixes, a latex solids-to-cement ratio of 0.15 and a cement content of 660 lb/yd^3 (392 kg/m^3) were used. Properties of concrete mixes used in this study are presented in Table 3.1. Total water contents per batch are included for comparison with recoveries described later in this chapter.

Equipment and Procedures

Equipment and materials used for this test method included the following: (1) a microwave oven with a power setting of 900 watts (the microwave oven purchased for this work, SHARP model R-9H93, has ten power levels, ranging from 100% to 10% of the maximum power); (2) a heat-resistant glass tray (9 in. [230 mm] in diameter and 2 in. [50 mm] deep), with a capacity of 4.5 lb (2,000 g) of fresh concrete; (3) a fiberglass cloth; (4) a 5,000-g-capacity balance; (5) a small metal scraper; and (6) a grinding pestle.

Test procedures used to dry a fresh concrete sample in the microwave oven were performed as follows. A piece of fiberglass cloth large enough to cover the tray and overhang the edges by approximately 6.0 in. (150 mm) was placed on the tray. The tray and the cloth were then weighed together. A sample of the fresh concrete weighing approximately 3.3 lb (1.5 kg) was obtained immediately after being mixed, placed on top of the fiberglass cloth in the tray, and wrapped with cloth. The tray and the concrete sample were then weighed. The microwave oven was set to the highest power (900 watts), the sample in the tray was placed in the oven, and the drying process was started. It was initially proposed that the sample should be dried continuously for 10 minutes and weighed immediately after being removed from the oven. After some trial mixes, it was observed that, in the first 5 minutes of drying, the concrete sample changed into a solid mass, with some moisture and vapor trapped inside it. This impeded full drying of the sample. To avoid this problem, it was decided to take the sample out after 5 minutes of drying and unwrap it, break up the mass with the edge of the scraper until coarse aggregate is separated from the mortar, and then grind the mortar with the pestle to expose a larger surface area for completion of the drying process. This procedure was performed carefully to avoid losing any material and did not take more than 45 seconds. After that, the fiberglass wrap was placed back over the sample and put back in the oven for another 5 minutes. The tray and sample were weighed immediately after removal from the oven, and the weight of the dry concrete sample was obtained. The sample was then dried for a final 2 minutes, and the weight was compared with the weight after 10 minutes of drying.

Table 3.1. Properties of mixes used in microwave method development.

Mix Designation	Type of Concrete[1]	Aggregate Absorption (%)	Total Water Content (lb)	Slump (in.)	Air Content (%)	Unit Weight (lb/ft^3)
I-1	Conventional—M	0.32	10.68	3.5	5.2	148.0
I-3	Conventional—H	0.32	11.14	5.2	6.4	144.6
I-4	Conventional—L	0.32	9.43	0.2	4.0	151.4
II-6	Conventional—M	1.89	10.70	3.0	6.0	148.6
II-8	Conventional—H	1.89	10.95	4.5	6.2	149.0
II-9	Conventional—L	1.89	9.65	1.0	5.2	151.6
III-1	Conventional—M	3.82	11.44	2.6	5.6	140.7
III-2	Conventional—H	3.82	12.10	6.6	7.0	139.0
III-4	Conventional—L	3.82	10.44	0.6	4.0	145.6
SF-II	Silica Fume—M	1.89	9.32	4.0	3.5	155.3
SF-III	Silica Fume—M	3.82	8.84	6.3	5.5	151.2
F-II	Fly Ash—M	1.89	10.48	4.4	8.0	139.9
F-III	Fly Ash—M	3.82	10.78	2.4	7.2	139.5
LX-I	LMC—M	1.89	8.54	5.0	8.0	137.4
LX-II	LMC—M	3.82	9.55	4.8	7.4	143.0

[1]M = moderate slump; H = high slump; L = low slump
Note: 1 lb = 0.4535 kg; 1 in. = 25.4 mm; 1 lb/ft^3 = 16.02 kg/m^3

The difference between initial weight of the fresh sample (W_i) and final dry weight was then the water content of the sample (W_s). The water content per cubic yard (cubic meter) of concrete (W) was calculated as follows.

$$W = W_s/W_i \times UW \times 27$$

where

W = water content of fresh concrete (lb/yd^3 [kg/m^3])
UW = unit weight of fresh concrete (lb/ft^3 [kg/m^3])
and 27 is a constant (not used for SI calculation).

For the laboratory mixes, as the total mass of the concrete batch was accurately known, measured water content was calculated simply by multiplying percentage water in the sample (as determined via microwave drying) by the mass of the entire concrete batch.

Test Results

The procedure described above, with the exception of the intermediate grinding steps, was essentially taken from that developed under the C-204 contract. It was first applied to the concrete mixes shown in Table 3.1 to develop initial data that might point to possible means of improving the method. Measured water contents at 10 and 12 minutes, as well as recoveries expressed as percent of initial water content of the batches, are shown in Table 3.2. At the 12-minute mark, recoveries range from 91% to 97% of the total water content of the batches. There seemed to be no relationship of recovery either to initial water content of the mix or absorption of the coarse aggregate used. Recoveries appeared to occur toward the low end of the range for the fly ash and LMC mixes, but only two of each of these types of concretes were tested. The failure to arrive at the expected water contents led the investigators to surmise that longer drying times may be necessary. Removal of the final amounts of water, however, is very slow and would not be consistent with the goal of producing a method that would be useable for field control. It would not be practical to delay discharge of the concrete beyond about 20 minutes, as this would create slowdowns in job production and add considerably to the cost of construction.

While drying time would certainly influence recoveries obtained, it was also felt that sampling errors might contribute to differences between measured and batched water contents. In an attempt to eliminate these errors and thereby more accurately determine percent recoveries, selected batches of concrete exactly the same size as the required sample (i.e., 1,500 g) were produced by handmixing in the dish used to hold the sample during drying. In this manner, the water content of the test specimen was exactly known, and possible sampling errors were eliminated. Mixing and drying procedures were executed as follows.

The dish was lined with fiberglass cloth as in the original procedure. Coarse and fine aggregates and the cement were weighed out and placed in the tray. Water was then added, and materials were mixed by hand with a spatula following the standard mixing procedures used previously. Immediately after being mixed, the sample was wrapped with the fiberglass cloth, weighed, and placed in the microwave. At the end of 10 minutes of being dried, the sample was unwrapped and stirred with the spatula or scraper, weighed, and returned to the microwave; the 2-minute-interval drying and weighing procedure was repeated three times for a total drying time of 16 minutes. Results are shown in Table 3.3.

Table 3.2. Measured water contents using microwave drying.

Mix Designation	Type of Concrete[1]	Aggregate Absorption (%)	Water Content (lb) at: 10 min. drying	12 min. drying	Recovery (%) at: 10 min.	12 min.
I-1	Conventional—M	0.32	9.66	10.10	90	95
I-3	Conventional—H	0.32	9.68	10.14	87	91
I-4	Conventional—L	0.32	8.65	8.76	92	93
II-6	Conventional—M	1.89	9.44	9.74	88	91
II-8	Conventional—H	1.89	9.90	10.51	90	96
II-9	Conventional—L	1.89	9.10	9.36	94	97
III-1	Conventional—M	3.82	9.88	10.80	86	94
III-2	Conventional—H	3.82	10.10	11.10	84	92
III-4	Conventional—L	3.82	9.61	10.00	92	96
SF-II	Silica Fume—M	1.89	8.39	8.76	90	94
SF-III	Silica Fume—M	3.82	8.18	8.38	93	95
F-II	Fly Ash—M	1.89	9.43	9.64	90	92
F-III	Fly Ash—M	3.82	9.49	9.78	89	91
LX-I	LMC—M	1.89	7.96	7.98	93	93
LX-II	LMC—M	3.82	8.56	8.84	90	93

[1]M = moderate slump; H = high slump; L = low slump
Note: 1 lb = 0.4535 kg

Table 3.3. Measured water contents of test sample batches.

Mix Designation	Type of Concrete[1]	Aggregate Absorption (%)	Recovery (%) at: 10 min.	12 min.	14 min.	16 min.
III-2	Conventional—H	3.82	83	93	*	99
II-6	Conventional—M	1.89	92	97	99	100
I-3	Conventional—H	0.32	94	98	99	99
I-4	Conventional—L	0.32	96	98	99	100
III-4	Conventional—L	3.82	90	97	99	100
I-1	Conventional—M	0.32	95	97	98	99

[1]M = moderate slump; H = high slump; L = low slump
Note: 1 lb = 0.4535 kg
*Not determined

These results indicate that, by following the recommended procedures for this oven, recoveries very close to 100% are achieved within 14 to 16 minutes of drying. This is an acceptable interval and indicates that previous deviances from expected recovery were most likely the results of batching and sampling errors—which, of course, are unavoidable when heterogeneous material such as concrete is being tested. The method itself does remove virtually all the water that is actually present in the sample. The work on these sample batches, however, indicated that a somewhat longer drying time was optimal; therefore, a 14-minute drying time was selected for all subsequent work.

Selected full-size batches were then repeated and tested for water contents at 10, 12, and 14 minutes. The results are presented in Table 3.4. In these cases, recoveries at 14 minutes are closer to 100%. In one case, mix III-4, recovery was 104%. It was recognized that this is a field method, and accuracies close to ± 3% should be acceptable for field control. A procedure is needed to simply, rapidly, and inexpensively determine whether significant amounts of water have been added to the mix in excess of the prescribed specifications of the design. The investigators believed at this point that the method had met this objective and that further refinements were not necessary. Field trials of the method are described in Appendix A.

Table 3.4. Measured water contents using increased drying time.

Mix Designation	Type of Concrete[1]	Aggregate Absorption (%)	Recovery (%) at:		
			10 min.	12 min.	14 min.
I-1	Conventional—M	0.32	93	96	97
I-4	Conventional—L	0.32	95	97	98
II-8	Conventional—H	1.89	93	97	98
II-9	Conventional—L	1.89	95	97	98
III-2	Conventional—H	3.82	84	92	97
III-4	Conventional—L	3.82	97	102	104

[1]M = moderate slump; H = high slump; L = low slump
Note: 1 lb = 0.4535 kg

Operator Effects and Precision

To be practical, the method should be capable of being conducted by personnel other than those who developed it. Four batches were selected to repeat the measurements performed by the method developer; an entry-level technician was selected to perform the repeated measurements. The technician was instructed in the procedure and worked with the developer on three trial mixes before making his independent measurements. Because only one microwave oven was available, the measurements were conducted sequentially. Previous work had indicated that delays of up to 30 minutes from time of initial mixing to time of testing had no significant effect on the measurement. Results using the two operators are shown in Table 3.5. There are essentially no differences between the two operators, indicating that relatively inexperienced personnel can successfully perform the procedure.

Time limitations allowed for generation of only limited precision data on the method. A conventional concrete mix was prepared (designation II-6), and an extended set retarder was added at a dosage of 1.2 oz/lb (7.8 ml/kg) of cement to prevent hydration of the mix during the test period. A series of samples was taken over a 2-hour period, and water contents were measured with the use of the microwave technique. Standard deviation of the measurements was 2.7 lb/yd^3 (1.6 kg/m^3). With the use of precision terminology recommended by ASTM, results of two properly conducted tests by the same operator on the same material should not differ by more than 7.6 lb/yd^3 (4.5 kg/m^3) in only 1 case in 20 (95% confidence limit). For a typical conventional concrete paving mix containing approximately 300 lb/yd^3 (178 kg/m^3) of water, this would amount to an expected maximum difference of about 2.5% of the mix water, certainly acceptable for field control purposes.

Table 3.5. Comparison of recoveries between operators.

Mix Designation	Type of Concrete[1]	Aggregate Absorption (%)	Recovery (%) at 14 min. by operator: 1	2
I-3	Conventional—H	0.32	95	96
III-2	Conventional—H	3.82	100	101
III-4	Conventional—L	3.82	98	99

[1]M = moderate slump; H = high slump; L = low slump
Note: 1 lb = 0.4535 kg

Summary and Recommendations

The microwave oven procedure for determination of water content of fresh concrete involves drying of a 1,500-g sample of fresh concrete in a 900-watt microwave oven equipped with a turntable to provide uniform drying. The total test time is approximately 16 minutes, including time for sample manipulation and check weighing twice during the drying process. The technique was found to be sufficiently reproducible for purposes of field control and independent of the effects of absorption of aggregates or consistency of concrete. The method can be applied to latex-modified and silica fume concretes as well as more conventional mixes. It is recommended that the method be adopted by agencies on a trial basis so that it may be further evaluated with a wider variety of materials and conditions than could have been included in the limited test program described in this report.

Guidelines for Predicting Temperature Effects in Fresh Concrete

The effects of temperature and moisture early in the life of concrete strongly influence early strength development and long-term durability. A guide, developed by SHRP under C-206 (Andersen, Andersen, and Whiting 1992), demonstrates how knowledge of concrete temperature, air temperature, and concrete mix characteristics allows the user to determine whether temperature-induced problems may be expected. Guidelines for avoiding such problems are also included.

Thermal conditions, if not addressed, can lead to significant problems, including the following:

- cracking caused by large temperature differentials between the interior of the concrete and the external environment;
- strength loss caused by the freezing of concrete before it has reached sufficient strength;
- strength loss caused by high internal temperatures within the concrete mass.

To avoid such problems, the rate of heat development within the concrete as well as its dissipation to the external environment must be known. The hydration of cement is an exothermic process in which heat is liberated during the reaction of cement with water. The amount of heat liberated is primarily a function of the composition of the cement, the fineness of the cement, and the temperature at which hydration takes place. Cements having a high tricalcium silicate (C_3S) content and a high fineness, such as Type III cements, generate more heat early in their hydration. Cements having relatively low C_3S content and low fineness, such as certain Type II cements, can have relatively low heats of hydration. Pozzolanic admixtures, particularly Class F fly ashes, can be used to replace part of the cement and thereby lower the heat of hydration. Class C fly ashes can have considerable cementitious properties and as such liberate heat during hydration. If used to replace part of the cement, Class C fly ashes would not necessarily reduce the overall heat generated to any appreciable extent (Barrow et al. 1989).

Development of heat in a concrete mass due to hydration of cement is a function of the maturity of the concrete. Maturity is a time/temperature function that requires a knowledge of the activation energy of the cement being used. Activation energies for a variety of cements were determined in this study and were used to develop relationships between heat development and maturity. Relationships between strength gain and maturity for these cements were also determined. Maturity is normally expressed relative to a reference condition. For instance, a maturity of 72 hours indicates that a particular concrete has reached a degree of hydration (and corresponding heat development) equivalent to 72 hours of curing at a reference temperature of 68°F (20°C).

As heat is generated within the concrete, an exchange with the external environment begins. Heat transport occurs both from the top of a pavement slab into the air and from the bottom of the slab into the base. Heat is transported by thermal conduction, convection, and radiation. Thermal conduction is controlled primarily by the thermal conductivities of concrete and base course and by the existing temperature gradients. Convection is controlled by the top surface convection coefficient and the difference between the temperature of the concrete and the temperature of the air. Radiation is controlled by a radiation constant and the surface temperature.

The balance between heat generation and heat loss is controlled by a number of parameters, knowledge of which allows prediction of thermal effects under a wide range of conditions. The parameters used in this study include type of cement (and its rate of heat development), amount of cement used per unit volume of concrete, presence of pozzolanic admixtures, concrete placement temperature, air temperature at placement, energy of activation of cement, wind velocity, pavement thickness, thermal properties of the base course, and the surface covering used on the pavement (if any) after finishing. To obtain a reasonable number of tables, the base properties, wind velocity, and thermal conductivity were assumed at constant levels. Because most concrete pavements are light colored (white curing compounds being widely used), the effects of solar radiation on surface temperatures are expected to be small and are not included. Surface protection was assumed to be initiated 3 hours after placement for temperatures below 40°F (5°C).

The tables were prepared from the output of a computer simulation of heat flow and strength development with the use of the parameters described above. An example of one such thermal effects table, reproduced from the guide, is shown in Figure 3.1. The simulation was carried out during the first 72 hours after placement. The following parameters must be entered for the tables to be used:

- Type of cementitious material. Cement Types I, II, and III are included in the tables. Tables showing instances when 20% of the cement was replaced by a Class F fly ash are also included. The tables are not meant to be applicable to other types of cement or specialty cement products. Additionally, they may not apply in cases where Class C fly ashes are used, as these ashes may in themselves generate heat during hydration of the concrete.

- Content of cementitious material. Total content of cementitious material ranges between 525 and 750 lb/yd^3 (311 and 445 kg/m^3). The tables should not be applied to instances where cementitious material contents outside of these limits are encountered.

- Concrete temperature. Concrete temperatures from 50° to 100°F (10° to 38°C) in increments of 10°F (5.5°C) can be used. Concrete temperatures must be within this range for the tables to be applied.

- Air temperature. The tables are applicable in conditions where ambient air temperatures fall between 0° and 100°F (−18° and 38°C).

CEMENT TYPE I, CONTENT 625 LBS/CU.YD.

AIR TEMP. [Degree F]	CONCRETE TEMPERATURE [Degree F] 50	60	70	80	90	100	SLAB THICKNESS [Inches]
0	EF	EF/TD	EF/TD	TD	TD	HT/TD	20
	EF	EF	EF	TD	TD	TD	16
	EF	EF	EF	EF	TD	TD	12
	EF	EF	EF	EF	EF	*	8
20	EF	*	TD	TD	TD	HT/TD	20
	EF	*	*	*	TD	TD	16
	EF	*	*	*	*	TD	12
	EF	EF	EF	*	*	*	8
40	*	*	TD	TD	TD	HT/TD	20
	*	*	*	TD	TD	TD	16
	*	*	*	*	*	TD	12
	*	*	*	*	*	*	8
60	*	*	*	*	TD	HT/TD	20
	*	*	*	*	*	TD	16
	*	*	*	*	*	*	12
	*	*	*	*	*	*	8
80	*	*	*	*	HT/TD	HT/TD	20
	*	*	*	*	*	HT/TD	16
	*	*	*	*	*	*	12
	*	*	*	*	*	*	8
100	*	*	HT	HT	HT/TD	HT/TD	20
	*	*	*	HT	HT	HT/TD	16
	*	*	*	*	HT	HT	12
	*	*	*	*	*	*	8

NOTES:

*	satisfactory thermal conditions
TD	risk of too large Temperature Differences within the concrete slab
EF	risk of Early Freezing
HT	risk of too High Temperatures within the concrete slab

Figure 3.1. Example of a thermal effects table.

Note: ° F = 1.8 × ° C + 32; 1 in. = 25.4 mm; 1 lb/yd^3 = 0.594 kg/m^{3}y.

- Thickness of the pavement. A wide range of pavement thicknesses can be handled by the tables. The range of 8 to 20 in. (0.2 to 0.5 m) includes most common design thicknesses. Thin overlays are outside of the scope of the tables.

The output of the tables consists of one of the following four symbols for any given combination of inputs:

* = Satisfactory thermal conditions

TD = Risk of differentials of temperature within the concrete slab that are too large

EF = Risk of early freezing

HT = Risk of temperatures within the concrete slab that are too high

Large temperature differentials can be minimized by reducing the temperature of the concrete mix, reducing the amount or changing the type of cement in the mix, reducing the pavement thickness, or insulating the slab so the differential temperature is reduced. All of these cautions (except the use of insulation) may also be applied to reducing the actual temperature generated within the concrete. Risk of early freezing can be avoided by increasing the temperature of the concrete, increasing the amount of cementitious materials, increasing pavement thickness, and insulating the pavement slab.

Figure 3.1 illustrates the many situations that may arise depending on the temperatures and amount of various components. The mix chosen for this example is typical of many used for highway pavements. The table illustrates that, when thick pavements are placed under very hot (100°F [38°C]) conditions and concrete temperature is not controlled, there is a risk of overheating of the slab. This could be minimized, for instance, by reducing the concrete temperature at placement or by substitution of Class F fly ash for cement; the guide includes tables showing the effect of substitution of 20% fly ash for cement. Other examples are given in the guide. A detailed discussion of the fundamental background and input variables is also included.

Packing-Based Aggregate Proportioning

The proportioning of sand to coarse aggregate has been found to have an important effect on the properties of both fresh and hardened concretes. Work carried out under SHRP C-201, Concrete Microstructure, demonstrated that theories of particle packing (Johansen and Andersen 1991) can be used to determine the optimal blend of a mixture of aggregates that will produce the maximum packing of constituents. The handbook produced under C-206 entitled *A Guide to Determining the Optimal Gradation of Concrete Aggregates* (Andersen and Johansen 1993) describes the application of particle packing theory to production of densely packed aggregate skeletons in a concrete mixture. The report states that workability of concrete will be maximized at the point of maximum packing of aggregate particles, as at this point the amount of cement paste will be minimized. The improved aggregate gradation and reduced paste content can reduce occurrences of segregation and bleeding and lead to a more durable concrete; they should also lead to a more economical concrete mixture.

This handbook consists of tables relating characteristics of the aggregate particle size distribution (denoted by D) and the packing density (as determined by a simple physical test parameter) to the packing of two or more different materials. The tables can be used to efficiently blend coarse aggregate fractions or mixtures of sand and coarse aggregate. The manual includes a description of test methods and procedures, instructions on use of the tables, and a summary of the theoretical background.

Three sets of tables are included in the handbook. These are (1) tables for blending two types of coarse aggregate, (2) tables for blending three types of coarse aggregate, and (3) tables for blending sand and coarse aggregate. The first two sets of tables are used when a producer has more than one type of coarse aggregate and wishes to optimize the coarse gradation prior to final blending with sand. The last set of tables is used to determine the percentage of coarse aggregate by total volume of aggregate. The tables are not meant to substitute for ACI 211 (ACI 1992c) or other standardized methods of concrete mixture proportioning. They serve simply as a substitute for determination of the aggregate volumes per unit volume of concrete. To use the handbook in conjunction with ACI 211, the amount of water per unit volume of concrete necessary to achieve the desired slump is selected, the w/c necessary to achieve the specified strength is selected, and the volume of cement is calculated from these two quantities. After the volumes of water, cement, and entrained air have been determined, the remaining volume is then taken up by coarse and fine aggregate. The ratio of the coarse and fine aggregate is then determined using the handbook.

Before the tables are used, a standard sieve analysis is performed on each aggregate to be blended. Results (as percent passing) are plotted onto semilogarithmic percentile paper, and the best straight line is drawn through the points. The characteristic particle diameter (D) is then determined from the diameter corresponding to 63 wt.% passing of the material.

The packing density of the aggregates is then established by determining the void content of each material using the dry rodding procedures described in AASHTO T 19. The dry weight packing density (PHI) is then computed as

$$\text{PHI} = 1 - (\%\ \text{voids}/100)$$

An example of use of the tables is given in Table 3.6, which treats the case of a blend of sand and a single coarse aggregate. To start, sieve analyses are performed on the coarse and fine aggregates, and the characteristic diameters (D) are determined. The packing densities (PHI) of both aggregates are then determined. The procedure for use of the tables is as follows: (1) the characteristic diameter of the sand (D) (in this case 0.06 in. [1.5 mm]) is located in the table (see marked entries in the first column); (2) the characteristic diameter of the coarse aggregate (D) (0.47 in. [11.9 mm]) falling within this previously marked region of the table is located (see marked entries in the third column); (3) the packing density of coarse aggregate (PHI) (in this case 0.55, which falls in the same region as the characteristic diameters previously located) is then located (see marked entries in the fourth column); (4) the packing density of the sand (PHI) (in this case 0.65), falling in the same region of the table as the packing density of coarse aggregate, is then located (see the single marked entry in the second column); and (5) the volume of coarse aggregate lying on the same line is then read from the table. In this case, it is 68% by volume. The final column shows the maximum packing density for the blend of sand and coarse aggregate. In this case, the density is 0.74, meaning 74% of the combined volume is occupied by solid material and 26% is void space.

A limited laboratory study was conducted to evaluate this method based on strength improvement criteria. Because resources were limited, only the third set of tables (combination of sand and coarse aggregate) was selected for this study.

Materials selected for the laboratory evaluation consisted of (1) a Type I cement from Lafarge Corp.; (2) a predominantly subangular natural sand consisting of a mixture of siliceous and calcareous minerals having specific gravity of 2.69, absorption of 0.9%, and FM of 3.00; and (3) a partially crushed dolomite coarse aggregate having a maximum topsize of 0.75 in. (19 mm) with specific gravity of 2.70 and absorption of 1.4%. The only admixture used was a 2% solution of neutralized Vinsol resin air-entraining agent.

Characteristic diameters (D) and packing densities (PHI) for the two aggregates were determined using the techniques outlined above. The characteristic diameter for the coarse aggregate was 0.64 in. (16.3 mm) and for the sand was 0.044 in. (1.12 mm). The percent voids for the coarse aggregate was 41.8, and PHI was 0.582. The percent voids for the sand was 31.5, and PHI was 0.685.

A practical mix with 564 lb/yd^3 (335 kg/m^3) of cement and 0.48 w/c was designed based on ACI 211 procedures (Mix A). Fine and coarse aggregates were proportioned based on fineness moduli (Table 6.3.6 of ACI 211.1). The mix was adjusted for a slump of 2 $\pm$1 in. (50 $\pm$25 mm) and air content of 6 $\pm$1%.

For evaluation of the packing tables, the same cement content and w/c were used. After the characteristic diameter and packing density of the aggregates were determined and the third set of tables was used (combination of sand and coarse aggregate), coarse aggregate expressed as a percentage of the total volume of aggregate was obtained. While the nearest value was 70% (70% coarse and 30% fine), another value of 64% (64% coarse and 36% fine) was also close enough to the interpolation from the tables to also be selected. Because the purpose of this study was to evaluate these tables, it was decided to use both values, and two mixes were designed (Mix B and Mix C). Six cylinders 6 $\times$ 12 in. (152 $\times$ 305 mm) were made for each mix, with one-half being tested at 7 days and one-half at 28 days for compressive strength.

The slumps of these two mixes (B and C) and the ACI mix (A) fell between 2 and 3 in. (50 and 75 mm), and air content fell between 5% and 7%. The workability of all three mixes was good. The compressive strengths for the three mixes at 7 and 28 days are shown in Table 3.7.

Comparing mix C (designed using the packing tables) with mix A (designed using ACI 211 procedures) shows that the compressive strength was improved by 7% for 7-day tests and by

Table 3.6. Use of packing-based aggregate proportioning table.

Sand		Coarse Aggregate		Volume % Coarse Aggregate	Maximum Density
D-inches	PHI	D-inches	PHI		
0.06	0.50	0.43	0.70	86	0.74
0.06	0.55	0.43	0.70	86	0.75
0.06	0.60	0.43	0.70	84	0.76
0.06	0.65	0.43	0.70	82	0.76
0.06	0.70	0.43	0.70	82	0.77
0.06	0.50	0.47	0.50	68	0.68
0.06	0.55	0.47	0.50	66	0.69
0.06	0.60	0.47	0.50	64	0.71
0.06	0.65	0.47	0.50	62	0.73
0.06	0.70	0.47	0.50	60	0.74
0.06	0.50	0.47	0.55	74	0.70
0.06	0.55	0.47	0.55	72	0.71
0.06	0.60	0.47	0.55	70	0.73
0.06	0.65	0.47	0.55	68	0.74
0.06	0.70	0.47	0.55	66	0.75
0.06	0.50	0.47	0.60	78	0.71
0.06	0.55	0.47	0.60	76	0.72
0.06	0.60	0.47	0.60	75	0.74
0.06	0.65	0.47	0.60	72	0.75
0.06	0.70	0.47	0.60	72	0.76

Note: 1 in. = 25.4 mm

Table 3.7. Compressive strengths of ACI versus packing table (PT) mixes.

	Compressive Strength (psi) at:	
Mix Designation	7 days	28 days
A (ACI)	3,855	4,848
B (PT)	4,047	5,155
C (PT)	4,123	5,329

Note: 1 psi = 0.0069 MPa

10% after 28 days. It would appear that there are some benefits to be realized from these procedures, but there are not enough to recommend major changes in current practices. The investigators would recommend that a more comprehensive series of tests be instituted, particularly in those cases where it is necessary to blend two or more coarse aggregates to achieve a proper gradation. Additional work along these lines would be beneficial.

Twin-Probe Nuclear Density Device

Previous Work

Concrete consolidation can have a significant effect on strength and fatigue life performance. A quick, reliable method is needed to evaluate plastic concrete consolidation throughout the depth of a pavement slab that will allow revibration of poorly consolidated areas prior to concrete hardening. Fresh concrete, when initially placed, contains entrapped air. Consolidation, generally achieved through mechanical vibration, is needed to eliminate entrapped air voids that can result in a weak, porous, and nondurable material.

Significant decreases in compressive and flexural strength with decreases in consolidation have been reported. Decreases in compressive strength of up to 30% and 60% have been reported for 5% and 10% decreases in consolidation, respectively (Glanville et al. 1947; Kaplan 1960). Flexural strength reductions of 24% for a 5% decrease and 45% for a 10% decrease have been reported (Kaplan 1960).

Poor consolidation significantly reduces the expected pavement service life by (1) reducing concrete strength and fatigue life, (2) decreasing load transfer and increasing faulting and pumping at doweled joints, and (3) decreasing crack spacing in continuously reinforced concrete pavement.

Investigators on SHRP Project C-204, *Non-Destructive Testing for Quality Control/Condition Analysis of Concrete*, concluded that a commercially available twin-probe nuclear density gage could be used to evaluate plastic concrete consolidation in a horizontal strata in areas where consolidation problems may occur. These include areas between vibrator paths, near the bottom of concrete pavements, below reinforcing steel, and below transverse joint dowel bars. The twin-probe gage was previously demonstrated for evaluating the density of plastic concrete along horizontal strata (Iddings and Melacon 1981). The effective bandwidth in which density is determined was estimated to be 1 in. (25 mm) thick. Laboratory testing indicated that density could be evaluated to within 1 in. (25 mm) of reinforcing steel. A limited field testing program in Iddings' study demonstrated the use of the twin-probe nuclear gage in monitoring plastic concrete consolidation. For 95 field tests on plastic concrete, the average absolute error between nuclear gage density and rodded unit weight was 1.6 lb/ft^3 (25.6 kg/m^3). This corresponded to a 1.1% measurement error.

Objective

The objective of this research program was to evaluate the use of the commercially available Troxler 2376 twin-probe nuclear density gage for determining plastic concrete consolidation levels. The objective was achieved by laboratory testing of several concrete block specimens with known densities. Lightweight aggregates and foam blocks were used to simulate honeycombed concrete and large entrapped air voids. Tests were conducted away from and below steel dowel bars to establish guidelines for evaluating consolidation in horizontal strata below steel reinforcement and dowel bars.

Nuclear Density Testing

The measurement of in situ density of a variety of construction materials has been facilitated through nuclear gage technology. Gamma ray photons are generated during decay of a Cs-137 source. Gamma rays are chargeless electromagnetic radiation that have zero mass and that travel at the speed of light. As the density of the material increases, the intensity of the gamma rays transmitted through the material decreases. The use of nuclear density gages to monitor concrete consolidation has been demonstrated and applied on a limited basis (Whiting and Tayabji 1988; Teng 1972; Bower and Gerhardt 1971; Mitchell et al. 1979). There are two basic operation modes: direct transmission and backscatter. These are similar in principle but differ significantly in their operations, and each has certain advantages and disadvantages.

The direct transmission method with a standard nuclear gage positions the material to be tested between the radiation source in a probe tip and the detector in the gage body. The main disadvantage of using this approach is that density is determined with depth on a diagonal, not horizontal, strata, as desired. The backscatter method positions both the detector and radioactive source in the gage body on the plastic or hardened concrete surface. The radiation detector receives lower energy, scattered gamma rays. The main disadvantages of this gage are that only near surface density is evaluated and large air gaps between the gage and surface can influence count rates. Typically, backscatter readings are influenced by density in the top 1 or 2 in. (25 or 50 mm). Backscatter measurements are commonly used in monitoring the density of thin-bonded, low-slump concrete or asphalt overlays.

Equipment and Procedures

The objective of this task was to evaluate the use of the Troxler 2376 twin-probe nuclear density gage for evaluating plastic concrete consolidation levels. A diagram of the gage is shown in Figure 3.2. The equipment includes a pulse height discriminator module, a scaler and ratemeter, a 5-mCi (0.19-GBq) Cs-137 source, and a detector consisting of a thallium-activated sodium iodide [NaI(Tl)] crystal. The cesium-radioactive source produces gamma photons of 662 keV (1.06×10^{-13} joule) initial energy. The scintillation counter is designed to count gamma photons of energies of about 662 keV (1.06×10^{-13} joule). The scaler timer digitally sums the count rate corresponding to a narrow range of pulse amplitude corresponding to a photon energy of 662 keV (1.06×10^{-13} joule).

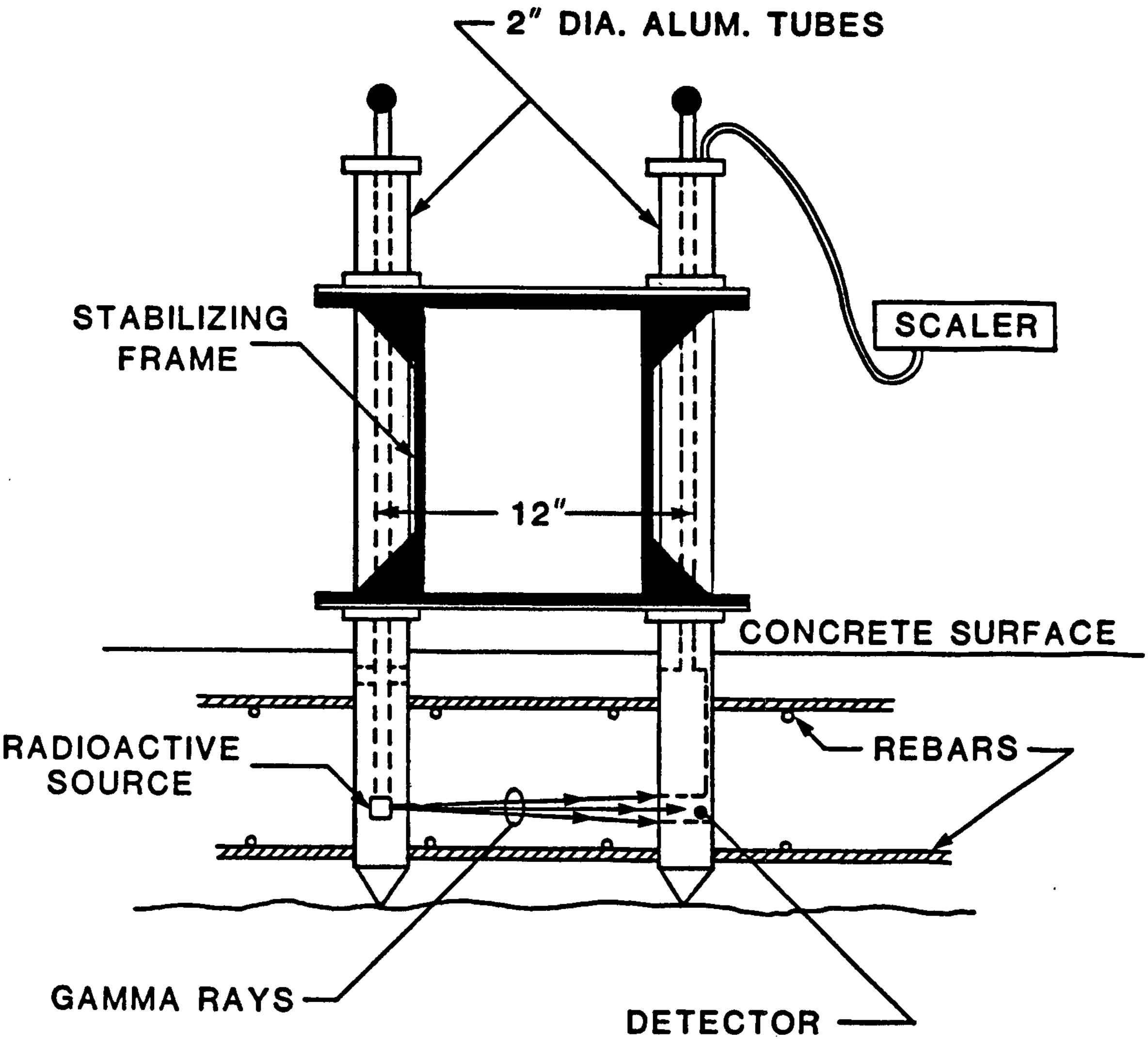

Figure 3.2. Diagram of the Troxler 2376 twin-probe nuclear density gage.

For plastic concrete, the source and detector are placed inside 2-in. (50-mm)-diameter aluminum tubes, which have been inserted into the plastic concrete. Solid aluminum points were attached to the tubes to ease insertion and prevent concrete intrusion into the tubes. A rigid steel frame attached to a plywood base assures that the tubes are inserted parallel at a nominal 12-in. (305-mm) spacing. Steel rods attached to both the source and detector were scribed to position the source and detector at equal elevations (relative to the top of the tubes). Direct transmission tests were conducted for 4-minute periods.

A small, 1-μCi (37-kBq) Cs-137 secondary source was mounted by the manufacturer to the detector tube to maintain the photopeak spectra when large changes in density were encountered when the steel dowel entered the region of measurement. In preliminary trials carried out without this secondary source, erratic and inconsistent count rates were measured when testing at steel dowel bar elevations. The orientation of source and detector was constant throughout the calibration and concrete testing procedures to minimize variability due to source and detector geometry.

Density as a function of nuclear count rate was first established by testing nine concrete standard blocks fabricated at different densities using different proportions of quartzite and expanded shale coarse aggregates. Hardened concrete densities ranged from 70 to 149 lb/ft^3 (1,120 to 2,390 kg/m^3). Standard blocks with dimensions 1 ft $\times$ 2 ft $\times$ 4 in. (305 mm $\times$ 610 mm $\times$ 102 mm) were cast with aluminum tubes for source and detector probes spaced at 12 in. (305 mm). Blocks were stored at 50% RH at 72°F (22.2°C). Companion cylinders were also made to determine hardened concrete density from weights in air and in water. Testing was initiated after approximately 90 days when cylinder densities did not significantly change with time. Density was determined at the beginning, middle, and end of the 2-month-long nuclear testing period. Average density change during the test period for the nine standard block mixes was 0.3%. Average standard block density was used in calibrating nuclear count rates. In addition to the standard concrete blocks, a polypropylene and three metallic (Mg, Al, Mg/Al alloy) standard blocks supplied with the gage were used to calibrate the nuclear count rate. Count rates are determined for material positioned between the point radiation source and the scintillation detector (diameter, 1.5 in. [38 mm]; thickness, 0.5 in. [13 mm]) because concrete is not a homogeneous material and the density is determined for a pyramidal section, the sensitive volume, between source and detector. (Figure 3.2 shows the pyramidal section in a side view, with the tip of the pyramid at the source and the base at the detector.) Differences in count rates were measured when the source and detector positions were reversed. Four-minute counts were measured three times for each calibration block in both directions.

Tests were conducted on twelve block specimens 14 $\times$ 24 $\times$ 10 in. (360 $\times$ 610 mm $\times$ 250 mm). After the specimens were cast, the aluminum tubes were positioned with a steel guide frame and driven into the plastic concrete. One specimen with aluminum guide tubes cast in place was tested in the hardened state approximately 24 hours after casting. The typical paving mix used for the block specimens was made with a crushed quartzite coarse aggregate, quartzitic fine aggregate, and 650 lb/yd^3 (386 kg/m^3) of cement. A set retarder was used in the mixes to allow sufficient time for plastic concrete testing. Three blocks were made at a significantly lower unit weight by substituting expanded shale coarse aggregate for the quartzite coarse aggregate.

Table 3.8. **Properties and characteristics of block specimens.**

Specimen Number	Unit Weight (lb/ft³)	Coarse Aggregate	Air Content (%)	Dowel Bar	Voids	Objective
1	147.2	crushed quartzite	5.0	no	none	tubes cast in place or driven
2	150.7	crushed quartzite	1.5	no	none	plain concrete density
3	150.8	crushed quartzite	2.2	no	none	plain concrete density
4	150.4	crushed quartzite	2.0	yes	none	effects of steel
5	150.0	crushed quartzite	2.1	yes	none	effects of steel
6	150.4	crushed quartzite	2.6	yes	none	effects of steel
7	146.0	crushed quartzite	6.6	no	none	effects of air content
8	135.2	quartzite, exp. shale	7.6	yes	none	effects of steel, low density
9	123.6	quartzite, exp. shale	7.6	yes	none	effects of steel, lower density
10	132.8	crushed quartzite	14.5	yes	three	effects of steel, high air, voids
11	146.0	crushed quartzite	5.8	yes	two	effects of steel, high air, voids
12	110.0 to 150.0	quartzite, exp. shale	3.4 to 17.3	no	none	hardened concrete layers

Note: 1 lb/ft^3 = 16.02 kg/m^3

Fresh concrete densities as listed in Table 3.8 were determined in general accordance with AASHTO T 121, except that the unit weight bucket was consolidated with a vibrating table in a manner similar to that used to consolidate the block specimens. Throughout the investigation, it was assumed that the unit weight of the concrete consolidated in the specimen block molds was identical to that determined by using AASHTO T 121 on samples taken from the concrete batch. Specimens were tested at elevation increments of 0.2 in. (5 mm). Because the density is determined for a pyramidal section between source and detector, differences in count rates were measured when the source and detector positions were reversed. One 4-minute count was taken for each calibration block in both directions. Eight specimen blocks had a dowel bar 1.25-in. (32-mm) in diameter across the 14-in. (360-mm) span positioned at mid-depth. Tests near dowel bars were performed at elevation increments of 0.1 in. (2.5 mm) to determine the point at which the steel dowel bar begins to influence density counts.

To simulate large voids, rigid foam blocks 1 in. wide (direction of testing) by 2 in. long by 4.4 in. high (25 × 50 × 110 mm) were positioned below the bottom of dowel bar elevation. Tests were performed with three voids positioned at the quarter points between the two guide tubes. A second specimen with two voids positioned at the third points between the guide tubes was also tested.

Test Results

Density as a function of nuclear count rate was first established by testing 9 hardened concrete blocks fabricated at different densities, three metal blocks, and one polypropylene standard block. Differences in count rates were measured when the source and detector positions were reversed. Three 4-minute counts were taken on each calibration block in both directions. The average count rate for the six trials was correlated with density as shown in Figure 3.3. The pooled replication standard deviation of count rates was 10.1 for counts ranging between 1,960 and 5,700. This corresponds to standard deviations of 0.7 and 0.9 lb/ft^3 (11.2 and 14.4 kg/m^3) at densities of 140 and 150 lb/ft^3 (2,240 and 2,400 kg/m^3), respectively.

For plastic concrete testing, one 4-minute count was taken at each elevation and in each direction. Tests away from voids and steel dowel bars indicated that differences between predicted and fresh concrete densities ranged between 0.1 and 2.5 lb/ft^3 (1.6 and 40 kg/m^3) and averaged 1.2 lb/ft^3 (19 kg/m^3). Differences in density when source and detector positions were reversed were attributed to nonhomogeneous density material and to the fact that density is determined for a pyramidal section between source and detector. There was no trend or consistent difference due to method of guide tube positioning (driven or cast), concrete density, or test elevation.

Tests were conducted to evaluate the effects on measured density of driving aluminum tubes into the plastic concrete. A test specimen was cast with aluminum tubes at the 12-in. (305-mm) spacing positioned in the form before the fresh concrete was placed. Tests were also conducted on the same specimen with a second set of tubes driven into the concrete after the concrete was placed and consolidated into the form. As shown in Table 3.9, average densities for tubes cast in place and driven in were 144.6 and 148.2 lb/ft^3 (2,316 and 2,374 kg/m^3), respectively. Density determined by weighing the filled form was 147.2 lb/ft^3 (2,358 kg/m^3). On average, the density when tubes were cast in place was 2.6 lb/ft^3 (42 kg/m^3) less than—and when driven in was 1 lb/ft^3 (16 kg/m^3) greater than—the measured plastic density of the fresh concrete.

Differences between the two procedures were significant. The predicted density was consistently higher for tubes driven, rather than cast, into the concrete at all 15 elevations tested. Differences ranged between 0.1 and 8.5 lb/ft^3 (2 and 136 kg/m^3) and averaged 3.5 lb/ft^3 (56 kg/m^3) higher for tests with guide tubes driven into the plastic concrete. When

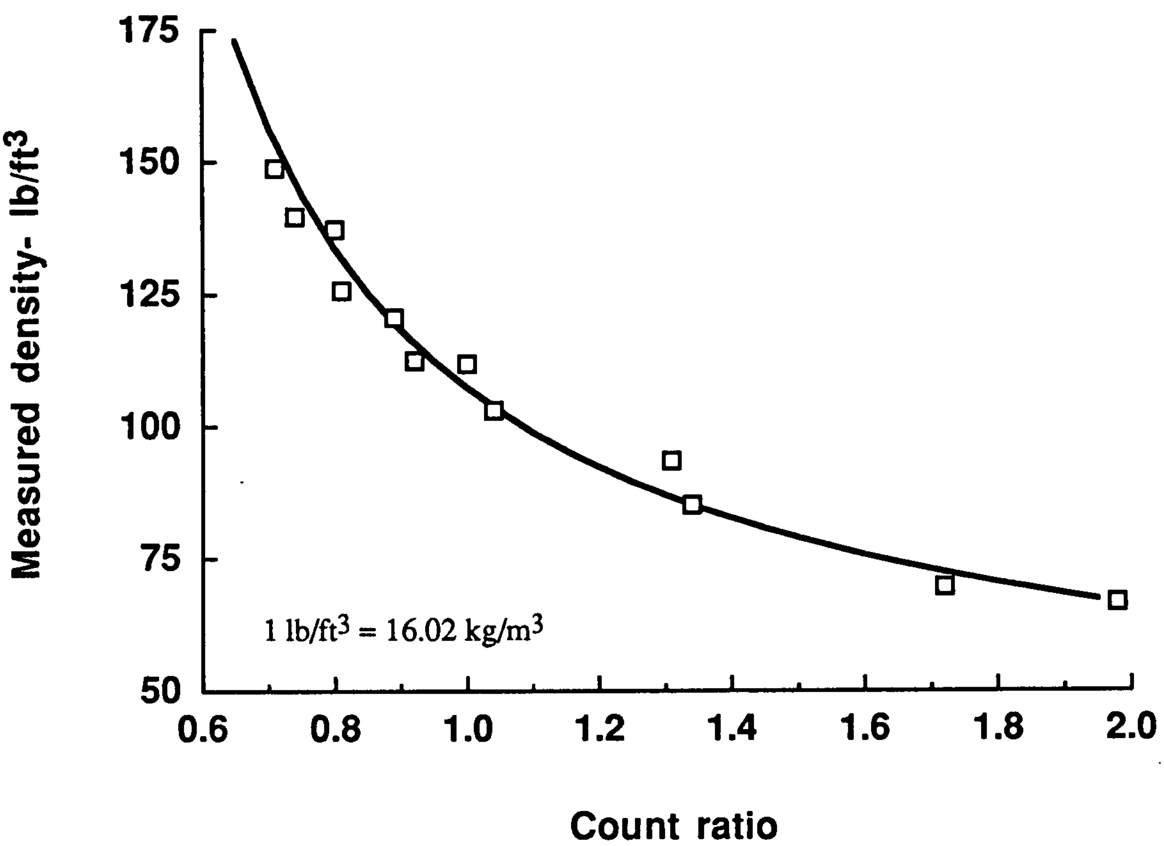

Figure 3.3. Twin-probe calibration curve.

Table 3.9. **Comparison between guide tubes cast in place and driven.**

	Tubes cast in place (lb/ft^3)	Tubes driven in (lb/ft^3)
Minimum	141.0	144.9
Maximum	147.4	151.2
Average	144.6	148.2
Measured	147.2	147.2
Standard deviation	2.1	2.0

Note: 1 lb/ft^3 = 16.02 kg/m^3

compared with unit weight determinations, however, nuclear density measurements taken from driven-in tubes were much closer to actual values. There was no significant trend of increasing density with depth.

A total of 175 tests was made on eleven specimens at elevations away from dowel bars and voids with the guide tubes driven into the plastic concrete. These tests were used to evaluate the feasibility of determining density in plain concrete at densities ranging between 123.6 and 150.8 lb/ft^3 (1,980 and 2,416 kg/m^3). As listed in Table 3.10, the average predicted densities were very close to the measured densities. For the 175 plastic concrete tests, the density prediction error (predicted minus measured) averaged -0.1 lb/ft^3 (1.6 kg/m^3) with a standard deviation of 2.3 lb/ft^3 (36.8 kg/m^3). The relative frequency error distribution was compared with the normal distribution centered at zero by using the sample standard deviation as shown in Figure 3.4. Measurement error appears to follow a normal distribution.

Table 3.10. Predicted versus measured density for plain concrete.

Specimen Number	Measured Density (lb/ft^3)	Number of Tests	Minimum Difference (lb/ft^3)	Maximum Difference (lb/ft^3)	Average Difference (lb/ft^3)	Standard Deviation (lb/ft^3)
1	147.2	15	−2.3	4.0	1.0	2.0
2	150.7	15	−2.7	0.9	−0.9	1.0
3	150.8	17	−0.9	1.4	0.0	0.7
4	150.4	6	−1.6	1.0	−0.4	1.2
5	150.0	5	−2.4	0.8	−0.8	1.2
6	150.4	11	−0.8	2.4	0.6	1.1
7	146.0	32	−5.5	1.5	−2.0	1.9
8	135.2	21	−5.3	6.9	1.2	2.9
9	123.6	21	−1.9	5.6	−0.5	2.8
10	132.8	25	−1.5	3.9	1.6	1.5
11	146.0	7	−2.9	1.0	−0.6	1.4

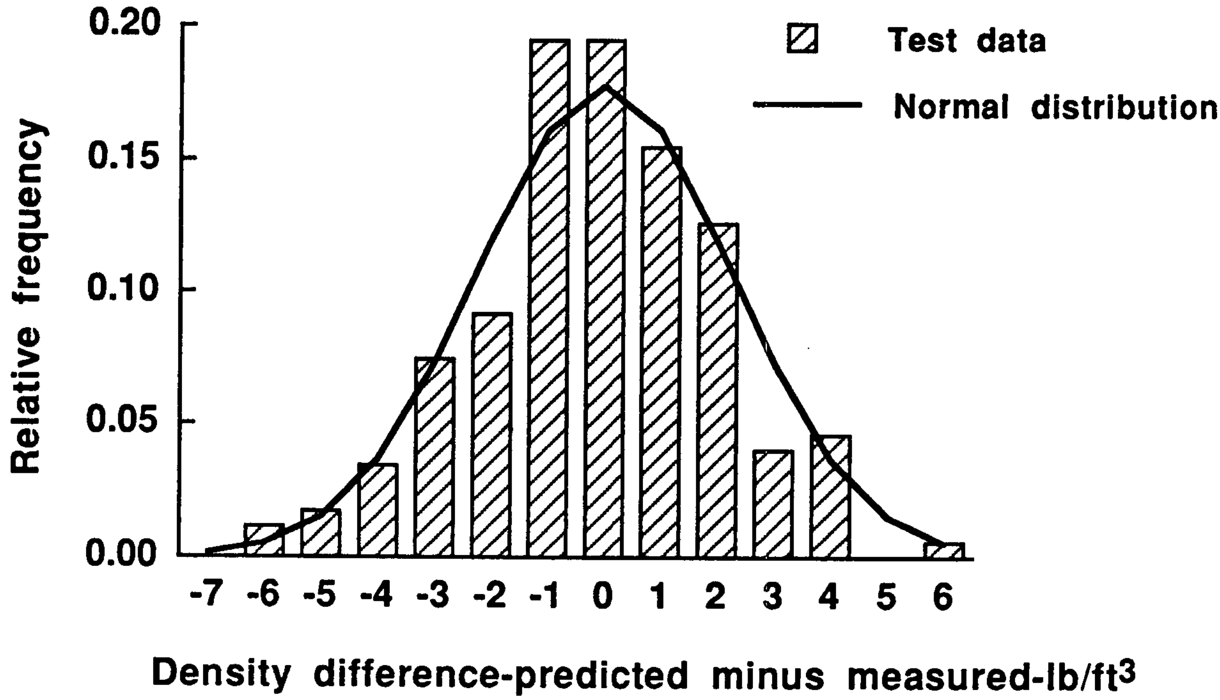

Figure 3.4. Histogram of prediction errors for twin-probe gage.

Tests near dowel bars on seven blocks were conducted at elevation increments of 0.1 in. (2.5 mm) to determine the point at which the steel dowel bar begins to influence density counts. As shown in Figure 3.5, the steel dowel influences the predicted density readings. Typical of all tests with dowels, the density profile is slightly offset from the actual dowel position, indicating that the detector position within the steel tube enclosure is positioned slightly higher than assumed. Assuming that steel symmetrically influences density readings (Figure 3.5), the total zone of influence is approximately 2.6 in. (66 mm). This corresponds to a symmetric distance of approximately 0.7 in. (18 mm) on each side of the 1.25-in. (32-mm)-diameter dowel bar. Estimated symmetrical distances for the effects of steel for the seven specimens tested are listed in Table 3.11. Average effective symmetrical distance from dowel bars where steel does not affect concrete density measurements is 0.3 in. (8 mm).

Table 3.11. Effective distance from influence of steel dowel bar.

Specimen Number	Total Zone of Influence (in.)	Symmetric Concrete Zone (in.)
4	1.4	0.1
5	2.8	0.8
6	2.6	0.7
8	1.3	0.1
9	1.3	0.1
10	1.3	0.1
11	1.8	0.3

Note: 1 in. = 25.4 mm

To simulate large voids in the plastic concrete, rigid foam blocks were positioned below the bottom of dowel bar elevation. Tests were performed with three voids positioned at the quarter points between the two guide tubes. A second specimen with two voids positioned at the third points between the guide tubes was also tested. As shown in Figure 3.6, the presence of voids significantly reduces the predicted density. For the specimen with three voids, the predicted density decreased from 134.4 lb/ft^3 (2,153 kg/m^3) above the dowel to 91.5 lb/ft^3 (1,466 kg/m^3) below the dowel (voids). Similarly, for the specimen with two voids, the predicted density decreased from 145.3 lb/ft^3 (2,327 kg/m^3) above the dowel to 117.9 lb/ft^3 (1,889 kg/m^3) below the dowel (voids).

Assuming a pyramidal section of material tested between the point source and detector, the volume percentage of concrete occupying this zone was 74% and 83% for the three- and two-void specimens, respectively. The weighted average (volume) densities calculated using measured concrete density and foam density of 1.9 lb/ft^3 (30.4 kg/m^3) were 98.8 and 121.5 lb/ft^3 (1,583 and 1,946 kg/m^3) for the three- and two-void specimens, respectively. Predicted densities based on nuclear readings (91.5 and 117.9 lb/ft^3 [1,466 and 1,889 kg/m^3])

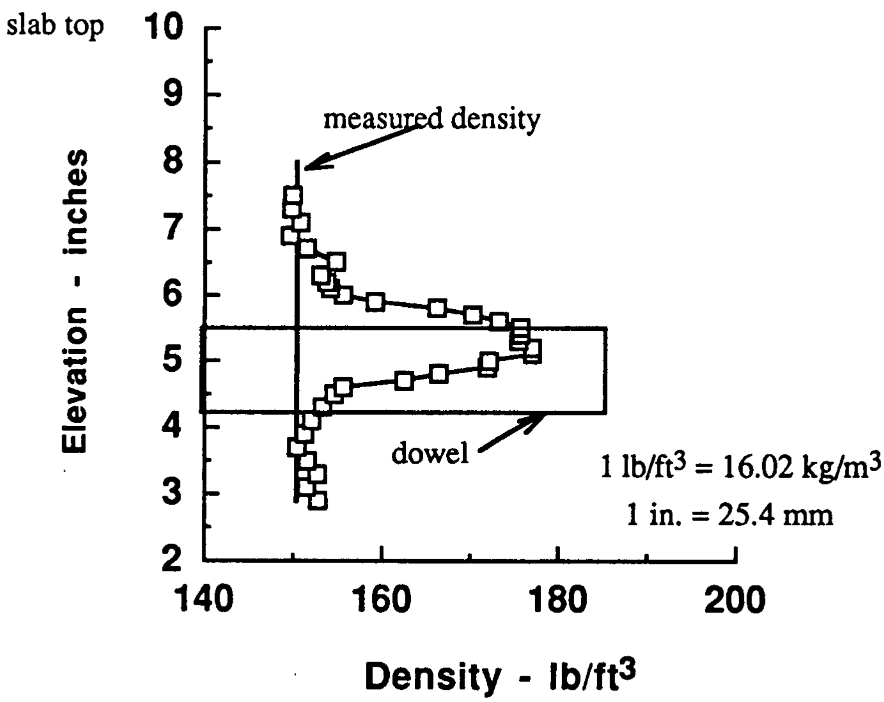

Figure 3.5. Effect of dowel bar on measured density.

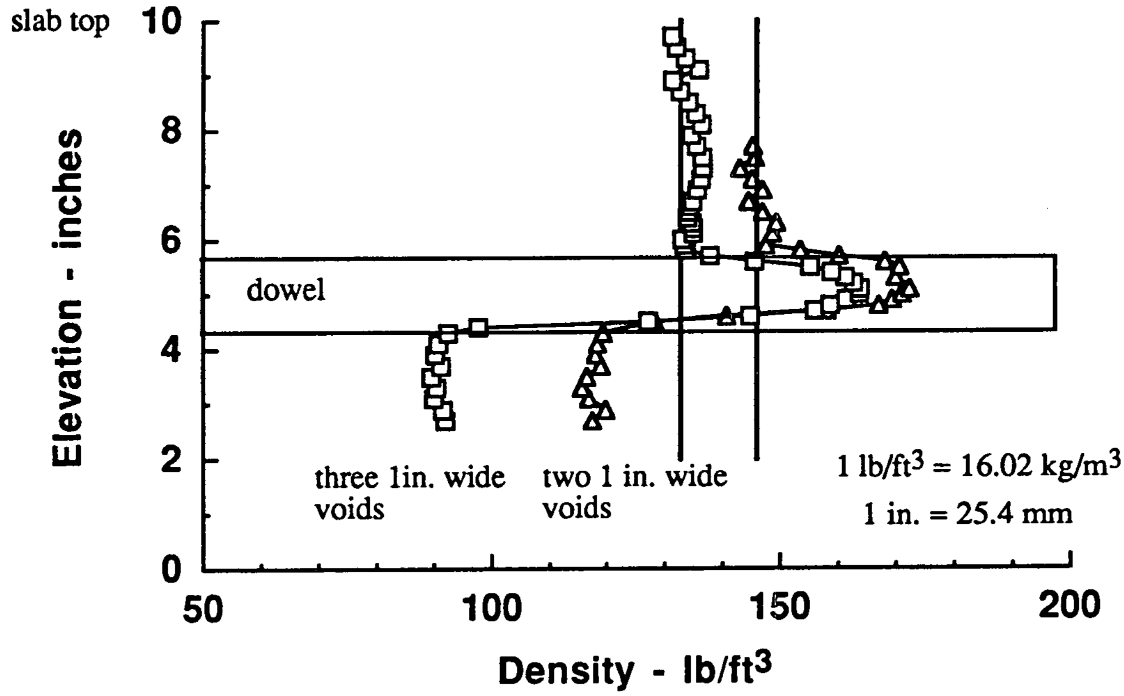

Figure 3.6. Effect of simulated voids on twin-probe density measurements.

were slightly less than the weighted average densities but were significantly lower than densities measured above the dowel bars.

Tests were also performed on one hardened concrete specimen made with three layers of concrete at different densities. As shown in Figure 3.7, results similar to those produced by plastic concrete testing were obtained. Some effects of the interface at the 7-in. (178-mm) elevation were observed.

Nuclear Density Testing Precision and Bias

Accuracy, or bias, of nuclear gage testing is difficult to assess. The volume of concrete represented in density measurements is indeterminate and may vary with the source-detector geometry of the equipment and the concrete mix. The several sources of variation affecting apparent accuracy that exist when indirectly determining density with nuclear gages include:

- inhomogeneities in the concrete sample;
- relying on measurement of plastic density in a unit weight container as representative of the true density;
- establishing the correlation between density and nuclear count rate.

Differences in density when source and detector positions were reversed were attributed to nonhomogeneous density of the concrete and to the fact that the density is determined for a pyramidal section between source and detector. Tests on plastic concrete indicated an average difference of 1.2 lb/ft^3 (19 kg/m^3) when the source and detector positions are reversed. To minimize the effect of inhomogeneities, a test result should be the average of tests in both directions. When this is done, this source of error is minimized and need not be considered in the overall variance.

Error is also introduced by assuming that the relatively small sample of concrete tested is representative of the larger 0.5-ft^3 (0.014-m^3) sample used for plastic unit weight tests. As shown in Figure 3.4, the error distribution in predicting measured unit weight from nuclear testing appears to be normally distributed around zero with a standard deviation of 2.3 lb/ft^3 (36.8 kg/m^3). This prediction error incorporates size of specimen effects and other unknown sources of experimental error.

Assuming that unit weight tests represent the true density, the second major error source is the accuracy in predicting density from the count rate correlation. As shown by the scatter of data points about the fitted line in Figure 3.3, some prediction error appears when the calibration curve is established. Standard error of estimate (nontransformed data) for calibration prediction error for data in Figure 3.3 was 0.6 lb/ft^3 (9.6 kg/m^3).

Assuming that these final two sources of variability are independent, the total estimated variance is computed as the sum of the individual variances. Estimated standard deviation for

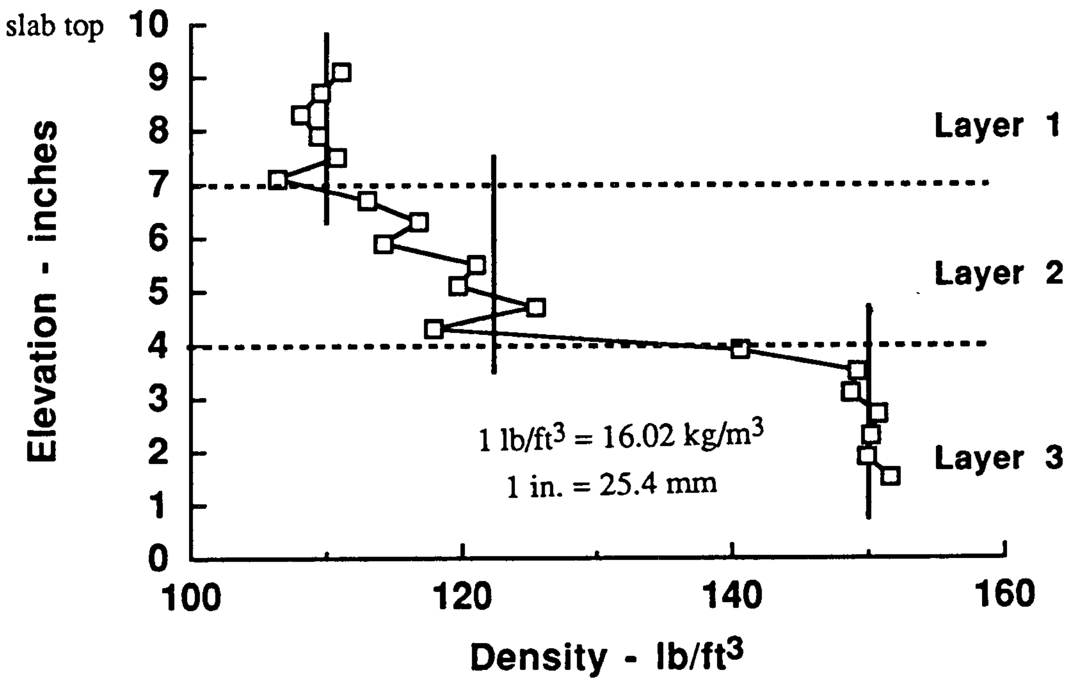

Figure 3.7. Density measurements on slab having three layers.

accuracy was 2.4 lb/ft^3 (38.4 kg/m^3). This implies that the difference between the measured and true densities will exceed approximately 6.8 lb/ft^3 (108 kg/m^3) in only 1 case in 20.

Nuclear testing is better evaluated by its precision defined as how well results can be reproduced. Two different effects should be considered in evaluating precision: the between-test and within-test variability. Between-test precision incorporates equipment and operator bias in setup and operation. Within-test precision incorporates only equipment effects because tests are immediately repeated without repositioning the source and detector.

Day-to-day equipment and operator variability is controlled by correlating the count ratio with density rather than just count rate with density. Count ratio is the count rate on material tested to the count rate on the magnesium standard block. Daily sources of variability in equipment settings, different testing environment effects, radiation source decay, and other equipment effects of variability are minimized by using a count ratio concept. Tests on calibration blocks and standards were performed by one of two trained personnel on seven different days to evaluate the between-test variability. Three count rates were averaged and used to compute count ratios for each testing period. Density was determined using the calibration curve shown in Figure 3.3. Assuming the calibration curve is free from error, the between-test variability was evaluated by computing the pooled standard deviation of the seven predicted densities for each of the nine hardened concrete and three field standard blocks. The magnesium standard block density was not used in the analysis because by definition the count ratio is 1.0. Replicate individual block standard deviations ranged from 0.6 to 3.0 lb/ft^3 (9.6 to 48 kg/m^3) and averaged 1.3 lb/ft^3 (20.8 kg/m^3). The pooled replicate between-test standard deviation was 1.6 lb/ft^3 (25.6 kg/m^3). This means that two readings by a single individual on a single sample should differ by more than 4.5 lb/ft^3 (72 kg/m^3) in only 1 case in 20.

Within-test variability was determined from data used in calibrating density with count rate. Three 4-minute counts were taken on each calibration block in both directions. Because computing density from count ratios (count divided by magnesium block count) would incorporate a between-test source of variability, counts were used in evaluating within-test precision. The pooled replication standard deviation of count rates was 10.1 for counts ranging from 1,960 to 5,700. With the assumption that the calibration curve perfectly relates density with nuclear count rate, the average magnesium block was used in evaluating within-test effects on density. The pooled replication standard deviation of count rates corresponds to standard deviations of 0.7 and 0.9 lb/ft^3 (11.2 and 14.4 kg/m^3) at a density of 140 and 150 lb/ft^3 (2,240 and 2,400 kg/m^3), respectively. This means that density calculated from repeated sets of counts on the same sample should differ by more than 2.0 and 2.5 lb/ft^3 (32 and 40 kg/m^3) at these densities in only 1 case in 20.

With the assumption that the two precision sources of variability are independent, the total estimated variance is computed as the sum of the individual variances. With the assumption of a concrete density of 150 lb/ft^3 (2,400 kg/m^3), the estimated standard deviation for precision was 1.8 lb/ft^3 (28.8 kg/m^3); i.e., repeated readings on a single sample of concrete should differ by more than 5.1 lb/ft^3 (82 kg/m^3) in only 1 case in 20. Precision of equipment operation excluding operator and equipment setup bias was 0.9 lb/ft^3 (14.4 kg/m^3) at a density of 150 lb/ft^3 (2,400 kg/m^3).

Nuclear Density Testing Summary and Recommendations

The twin-probe nuclear gage was evaluated for use in determining the density of plastic horizontal strata. The commercially available gage required the attachment of a second small radioactive source on the detector tube to maintain the photopeak spectra for Cs-137 and the fastening of solid aluminum points to the tubes inserted into the plastic concrete.

Density is indirectly determined by correlating nuclear count rates with density determined on material blocks of known densities. The standard deviation or prediction error for thirteen different density materials was 0.6 lb/ft^3 (9.6 kg/m^3). Tests indicated that differences in predicted densities averaged 1.2 lb/ft^3 (19 kg/m^3) when the source and detector positions were reversed.

A limited number of tests suggested that, on average, the density measured when guide tubes were cast into fresh concrete was 2.6 lb/ft^3 (42 kg/m^3) less than the plastic density determined from unit weight tests. When guide tubes were driven into already consolidated concrete, the measured density was 1 lb/ft^3 (16 kg/m^3) greater than the plastic density obtained from unit weight measurements. On a larger sample of data, the apparent effects of insertion tubes driven into the plastic concrete were smaller. Overall, for the 175 plastic concrete tests, the density prediction error (predicted minus measured) averaged -0.1 lb/ft^3 (1.6 kg/m^3), with a standard deviation of 2.3 lb/ft^3 (36.8 kg/m^3). Measurement error appears to follow a normal distribution. On the basis of the larger data set, it appears that disturbance effects of possible increased density adjacent to tubes driven in are minimal and do not significantly affect predicted densities.

Tests near dowel bars were performed to determine the test proximity at which the steel dowel bar influences density counts. On all tests near dowels, the density profile is slightly offset from the actual dowel position, indicating that the detector crystal within the steel tube enclosure was positioned slightly higher than assumed. With the assumption that the effects of steel symmetrically influence density readings, this symmetric zone of influence ranged from 0.1 to 0.8 in. (3 to 20 mm) beyond each side of the dowel bar. Estimated symmetrical distances for the effects of steel for the seven specimens tested are listed in Table 3.11. On the basis of laboratory tests, the recommended effective symmetrical distance from dowel bars at which steel does not affect concrete density measurements is 1 in. (25 mm)—i.e., the source and detector should be located no closer than 1.0 in. (25 mm) from the highest or lowest point on a dowel or rebar. As Figure 3.5 shows, the presence of steel in the gage's sensitive volume is usually very apparent from the readings obtained.

To simulate large voids, rigid foam blocks were positioned below the bottom of dowel bar elevation. The presence of voids significantly reduces the predicted density, demonstrating that the gage could detect larger voids under dowel bars.

Accuracy or bias of nuclear gage testing is difficult to assess. The volume of concrete represented in density measurements is indeterminate and may vary with the source-detector geometry of the equipment and the concrete mix. Sources of accuracy variability include test size effect and calibration error. Estimated standard deviation for accuracy was 2.4 lb/ft^3

(38.4 kg/m^3). This implies that the difference between the measured and true densities will exceed approximately 6.8 lb/ft^3 (108 kg/m^3) in only 1 case in 20.

Nuclear testing is better evaluated by its precision, which is defined as how well results on a single sample can be reproduced. Between-test precision incorporates equipment and operator bias in setup and operation. Within-test precision incorporates only equipment effects because tests are immediately repeated without repositioning the source and detector. By assuming a concrete density of 150 lb/ft^3 (2,400 kg/m^3), the estimated total standard deviation for precision was 1.8 lb/ft^3 (28.8 kg/m^3); i.e., repeated readings on a single sample of concrete should differ by more than 5.1 lb/ft^3 (82 kg/m^3) in only 1 case in 20.

The size of the 2-in. (51-mm)-diameter aluminum tubes made it difficult to insert them into the low-slump concrete. Driving the tubes into the concrete created high pressure and entrapped air, which tended to eject the tubes after insertion. Because calibration of the nuclear gage was performed at a set 12 in. (305 mm) between the detector and source, the tubes must be properly positioned in plastic concrete. Because source and detector elevation is relative to the top of the tubes, any differences in tube elevation effectively increases the nuclear travel path. Increasing the volume of concrete tested decreases the count ratio leading to a higher than actual predicted density. It is recommended that a provision to release air as tubes are driven into the concrete be incorporated into design of future devices. Alternatively, the size of the detector could be reduced, thereby reducing the required insertion tube diameter. Additional research would be required to evaluate the loss in accuracy or precision with a smaller detector.

Nuclear tests were done with 4-minute counting periods. The gage is provided with selectable sampling times of 15 seconds, 30 seconds, 1 minute, 2 minutes, and 4 minutes. Testing time might be reduced by reducing the time the count rate is measured. A limited number of tests in the laboratory indicates that the time might be reduced to 2 minutes with no loss in accuracy. Additional research is needed to optimize accuracy and minimize testing time.

Marking the probe rods so that the relative orientation between source and detector was constant throughout the calibration and concrete testing procedures minimized variability due to source and detector geometry. The insertion tubes, probe rods, source, and detector tube should have some markings so that, during field testing and calibration, orientation variability is eliminated.

The unit used in evaluating plastic concrete density was classified as a research or development model. The unit should be made more practical and more durable for field use by improving operating manuals, decreasing equipment size, and improving the electronics and user-friendliness.

A field testing program is highly recommended to demonstrate plastic concrete density determination. The effect of uncertainty in exact dowel location on the number of tests required to ascertain that tests are not influenced by dowel bars should be evaluated. This will establish whether the gage can quickly and easily be used to evaluate consolidation under dowel bars.

Laboratory tests indicated differences in predicted densities averaged 1.2 lb/ft^3 (19 kg/m^3) when the source and detector positions were reversed. Tests in the field would also be required to establish variability in concrete density. This would be used to establish whether switching source and detector positions are required at each elevation.

Other Newly Developed Test Procedures

As part of the SHRP Concrete and Structures Program (C-100 and C-200 series projects), a variety of test procedures that can be usefully applied to fresh and hardened concretes under both laboratory and field situations was developed. Many of these procedures have been brought to fruition under their respective contracts; they are now available as SHRP products and are described and illustrated in a SHRP products catalog (SHRP 1992). As part of C-206, a number of SHRP products were evaluated early in the program to determine which of these required further work and validation under C-206. The techniques described earlier in this chapter represent those that were selected for the more detailed evaluations. Other test methods were less well developed at the time that these preliminary evaluations were made. These methods and products were subsequently developed under each of the SHRP projects and are fully described in documents prepared by investigators on C-100 and C-200 series projects. Documents issued under these projects should be consulted for details. So that readers will be aware of the various techniques that were developed during the SHRP program in these areas, a listing and very brief description and opinion of the utility of each technique is included below. Again, the reader is referred to the documents issued by the original investigators or the aforementioned catalog (SHRP 1992) for details.

Tests for Concrete Permeability

Both field and laboratory tests for concrete permeability have been developed under the SHRP Concrete and Structures Program. The field test allows a large number of measurements to be taken in a noninvasive manner in a short period of time. The laboratory tests are designed for measurement of permeabilities of concrete specimens prepared in the laboratory or cores obtained from field structures.

Field Technique

Under contract C-101, Assessment of Physical Condition of Concrete Bridge Components, a surface air flow device that indicates permeability based on rate of air flow through a concrete surface under an applied vacuum was developed. The device can be used on horizontal surfaces such as decks or pavements and on vertical surfaces such as bridge girders. However, the test is not sufficiently quantitative to allow for prediction of actual permeabilities from the results of field tests. The test must be regarded as an indicator of

relative permeability and, if more accurate values are needed, cores should be taken and tested using more standardized techniques.
Product No. 2031
Documentation: *Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion, Volume 7: Method for Field Measurement of Concrete Permeability* (SHRP-S-329)

Laboratory Pulse-Decay Technique

Under contract C-201, *Concrete Microstructure*, a transient pressure permeability apparatus was designed and constructed to allow a rapid and accurate measurement of water transport in concrete. The sample must be completely saturated with water using a vacuum technique prior to start of the test. A differential pressure is established across a concrete cylinder or core, and both pressure rise and decay are monitored during a period of 1–2 hours. From the rate of decay of the applied pressure, the permeability may be calculated. The test is applicable to concretes with permeabilities as low as 1 μdarcy ($10^{-18} m^2$). As currently configured, specimens that may be accommodated by the test rig are restricted to cores or cylinders with maximum diameters of 3 in. (75 mm).
Product No. 2007
Documentation: *Development of Transient Permeability Theory and Apparatus for Measurements of Cementitious Materials* (SHRP-C-627)

Laboratory AC Impedance Technique

The permeability of a material is a function of its porosity, pore size distribution, and interconnectedness of pores. Likewise, the electrical impedance of a porous material is a function of the same parameters plus the impedance of the pore fluid and extent of saturation. When the pore fluid and saturation are held constant, measurement of concrete impedance (or resistance, in the DC case) indicates relative permeability. This is the basis for the well-known AASHTO T 277 test, somewhat improperly termed the rapid chloride permeability test. While the AASHTO test requires 6 hours, AC impedance can be measured in a matter of minutes, provided that the sample is saturated before being tested. The AC impedance test developed under SHRP C-205, *Mechanical Behavior of High-Performance Concretes*, can be used to obtain a rapid estimate of relative permeability for both laboratory and field core samples.
Product No. 2026
Documentation: *Mechanical Behavior of High Performance Concretes, Volume 1: Summary Report* (SHRP-C-361), and *Volume 4: High Early Strength Concrete* (SHRP-C-364)

Fluorescent Optical Microscopy of Concrete

Impregnation of a concrete surface with fluorescent epoxy can enhance the features of both the macrostructure and microstructure of concrete. Macrostructural features can include aggregate distribution and air-void size and distribution. These can be examined at magnifications ranging from 10 to 100 times using a good quality stereomicroscope. Parameters of the air-void system can be determined by a trained technician. The technique can also aid in analysis of concrete microstructure using carefully prepared thin sections. Preparation of thin sections requires considerable experience and expensive sample preparation equipment. Once the thin sections are prepared, an experienced petrographer with education in optical mineralogy can use the technique to examine composition of aggregates, quality of the cement paste (including w/c), paste density, microcracks, and products of reactivity. This may include alkali-silica or carbonate reaction products and products of reaction with sulfates or other salts. By comparison with a set of calibrated standards, more quantitative information on paste porosity and w/c can be developed.
Product No. 2008
Documentation: *Concrete Microscopy* (SHRP-C-662)

Detection of Alkali-Silica Reactivity (ASR)

First Step—Visual Identification

A handbook that can aid in visual identification of ASR has been produced by SHRP. This can be used to establish the presence of ASR in pavements and bridge structures by visually comparing the illustrations in the handbook with distresses encountered in the field. The handbook also describes the nature of ASR and its effects, gives many examples of ASR in bridges and pavements, and describes a rapid test procedure for identification of ASR (see chemical test below). The handbook can be supplemented by use of the CONPAV-D and CONSTRUC-D modules of the HWYCON expert system described in Chapter 2 of this report. The handbook also includes comparisons of cracking caused by mechanisms other than ASR so that users may better be able to distinguish causes of cracking. In some cases, more detailed testing may be required to establish cause, especially when multiple mechanisms are operative.
Product No. 2010
Documentation: *Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures* (SHRP-C-315)

Second Step—Chemical Test for ASR Detection

The presence of ASR gel reaction products in concrete is indisputable evidence that ASR has developed. These gels are not always obvious, and many times they are detectable only by a skilled petrographer using a laboratory-grade microscope. For other personnel to recognize this evidence much earlier in the life of the structure, a method for gel recognition has been developed. The method uses a solution of uranyl acetate which, when sprayed onto a fracture surface of concrete, will fluoresce bright yellow under short-wavelength ultraviolet light wherever ASR gel is present. The method does not replace detailed petrographic examination but can be viewed as a compliment to a petrographic exam; it is one of the tools that can be used in assessing the presence of ASR. The method is described in the SHRP ASR handbook, as are examples of concretes examined using the method. Although the method can be used in the field, regulatory questions concerning disposal of concrete treated with the test solution make it more appropriate as a laboratory test procedure.
Product No. 2013
Documentation: *Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures* (SHRP-C-315)

Test for Determining Aggregate Reactivity

A new, more rapid test for alkali reactivity was proposed to ASTM in 1990. This test method is ASTM Proposal P-214, *Proposed Test Method for Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction*. It has since been modified somewhat on the basis of tests carried out under SHRP C-202, Eliminating or Minimizing Alkali-Silica Reactivity. By using this test, results can be obtained in as few as 16 days, compared to many months for C-227. The method is especially useful for the slow reactor aggregates, which may take more than a year to show noticeable expansion using C-227 procedures. Aggregates are prepared for test in a manner similar to C-227, and either high-alkali or the specific job cements can be used. Mortar bars are prepared and then stored in a 1N sodium hydroxide solution at 176°F (80°C). Expansion limits established at 16 days allow an assessment of potential reactivity to be made. A limit of 0.08% has been suggested on the basis of experimentation carried out by the C-202 investigators. The method can also be used to determine the amount of fly ash or ground-granulated blast furnace slag (GGBFS) that should be used to inhibit expansion due to ASR. Recent work at the Virginia Transportation Research Council (Lane 1993) has indicated that the method may also be able to detect aggregates susceptible to alkali-carbonate reactions, although further work needs to be carried out to confirm these observations. Lane also noted that the method may not be applicable to some reactive gneisses and quartzites from the eastern United States, which fail to expand in the procedure but which have caused long-term problems in actual field installations.
Product No. 2009
Documentation: Eliminating or Minimizing Alkali-Silica Reactivity (SHRP-C-343)

Use of Pozzolanic Materials

In many cases, certain materials that will inhibit ASR can be added to the concrete batch. The materials most widely used for this purpose are pozzolans, primarily fly ashes. Fly ashes are finely divided residues of coal combustion and are in themselves reactive. Because their surface area is so high, however, the gels formed are nonexpansive and do not harm the concrete. Ground-granulated blast furnace slags function in a similar manner. Fly ashes can be effective when used in the range of 20% to 40% cement replacement. Class C fly ashes may require higher replacement levels to be effective and in some cases may not be effective at any realistic level of replacement. Slags are used in larger quantities, ranging from 50% to 75% replacement. Another material that can be used as a pozzolan is silica fume, a very highly divided form of silica. Because of its high surface area, it is used at lower replacement levels, ranging from 5% to 10% by weight of cement. Natural pozzolanic materials, such as volcanic ashes, calcined shales, and diatomaceous earths, are available in limited quantities in certain localities and can also be used to control ASR. The particular cement/pozzolan/aggregate combination must be tested carefully before being used because all pozzolans of a given class are not equally effective. The new test for ASR reactivity (see above) can be used effectively for this purpose.
Product No. 2017
Documentation: *Eliminating or Minimizing Alkali-Silica Reactivity* (SHRP-C-343)

Use of Lithium Hydroxide

Research conducted under SHRP C-202 has demonstrated that certain lithium salts are very effective inhibitors of expansive ASR. A lithium-bearing silicate reaction product that is similar to gels produced by ASR (but that is apparently nonexpansive) forms. Because all lithium salts studied convert to lithium hydroxide (LiOH) in concrete, lithium hydroxide was chosen as the most appropriate material for control of the reaction. It can be added to a concrete mix at a dosage of approximately 1.0% by weight of cement. For a particular aggregate/cement combination, the rapid reactivity test previously described can be used to determine the optimum dosage of LiOH to use. Additionally, LiOH can be used in combination with fly ash and may be especially effective in this regard in areas where deicing salts based on sodium chloride are used.
Product No. 2017
Documentation: *Eliminating or Minimizing Alkali-Silica Reactivity* (SHRP-C-343)

Identification of Aggregates Susceptible to D-Cracking

Pressures can be developed within concrete aggregates during freezing. D-cracking is a special case of aggregate-related freezing and thawing deterioration of concrete that occurs primarily with slabs-on-grade. It starts at the undersides of slabs where moisture is available and saturation of the concrete is high. Within 1 year or less, closely spaced horizontal crack planes can develop at the underside of the slab. Within a few years, these cracks will propagate to the surface to form the patterns of fine, closely spaced parallel cracks typical of d-cracking at joints, edges, and major structural crack areas. Aggregates can be identified by testing of concrete prisms using standard techniques such as AASHTO T 161, but these techniques are time consuming and labor intensive. Highway agencies have continued to search for new methods for determining durability of coarse aggregate prior to incorporation into concrete mixtures. As part of the studies carried out under SHRP C-203, *Resistance of Concrete to Freezing and Thawing*, a technique termed hydraulic fracture has been developed.

The test is based on the assumption that hydraulic pressures produced in concrete by freezing can be simulated by subjecting aggregates submerged in water to high pressures. As the applied pressure increases, the water is forced into the smaller pores of the aggregate. Under sufficient pressure, water can penetrate into pores in the size range known to be associated with freeze-thaw damage. At the same time the water is penetrating the pores, air in the pores is being compressed. When the external pressure is released, this compressed air will expand and push the water back out of the aggregate, creating a hydraulic pressure very similar to that which occurs when water freezes into ice and exerts pressure on the remaining unfrozen water. Fracture of the aggregate occurs when the water cannot be released quickly enough from the pore system. This is the basis for this test.

A total of five sets of pressurization cycles are performed, making for a total of fifty separate pressurization cycles. Hydraulic fracture index (HFI) is calculated. HFI represents the number of cycles needed to obtain 10% fracture in the aggregate. In general, HFI greater than 75 will indicate a durable aggregate, while an index less than 75 will indicate a possible problem regarding freeze-thaw durability. Although no false positives have been encountered using this procedure (i.e., the test accurately identifies all aggregates that are not susceptible to d-cracking), false negatives have been encountered, indicating that the test may reject sound aggregates. For this reason, when an aggregate fails the test, additional tests using standard freeze-thaw procedures should be performed.
Product No. 2002
Documentation: *Resistance of Concrete to Freezing and Thawing: Frost Resistance of Concrete Made with Durable Aggregates, with Frost-Susceptible Aggregates, and Field Testing* (Janssen forthcoming)

Modified AASHTO T 161 Freeze-Thaw Test

While testing of aggregates alone can prove useful as a screening procedure, information on the freeze-thaw performance of the actual concrete mixtures to be used on a particular job is often needed. Where combinations of aggregates are used, it may be more realistic to test the combination in concrete rather than test each individual aggregate by itself in an unconfined test. AASHTO Test Method T 161 Procedure A subjects concrete test specimens to a series of rapid freeze-thaw cycles while totally submerged in water. This is a very severe test procedure because the concrete is not allowed to dry at any point in the cycle. In reality, it is likely that concrete will undergo periods of drying during the course of a typical winter. Although AASHTO T 161 Procedure B can be used to freeze specimens in air during the test, the drying that occurs during this process may result in a lower degree of saturation than is actually present in field concrete.

Because of the drying that occurs in conventional T 161 Procedure B when specimens are frozen while directly exposed to air, a modification was developed by the SHRP C-203 contractor. This modification involves wrapping each specimen with a soft, absorbent terry cloth towel before immersing it in the freeze-thaw chamber. After the specimen is wrapped with the towel, the normal steps in the testing for Procedure B are followed. The specimens remain wrapped at all times except when they are being measured for length, weight, and frequency. Comparison testing indicates this new procedure may be at least as severe as Procedure A and in some cases even more so. Round-robin testing needs to be carried out on the technique to evaluate its reproducibility.
Product No. 2018
Documentation: *Resistance of Concrete to Freezing and Thawing: Frost Resistance of Concrete Made with Durable Aggregates, with Frost-Susceptible Aggregates, and Field Testing* (Janssen forthcoming)

Measurement of Damping Constant of Concrete

Measurement of mechanical damping or internal friction (Q^{-1}) has been an accepted technique for studying internal structure and environmental response of materials such as polymers and composites for many decades. It has largely been ignored to date in the concrete field. Damping may be measured by the sharpness of the resonance peak of a specimen in the frequency spectrum, indicating how much energy is dissipated through such processes as phase transitions and cracking. An increase in damping (or a decrease in its inverse quality [Q] factor) implies a degradation in the specimen and usually indicates a rapid decrease in elastic properties. In SHRP C-203, the Q factor was studied by measuring damping after excitation of the freeze-thaw specimens, using an impulse method. Changes in damping were an early indicator of freeze-thaw damage. Although the measurement of damping alone will probably not substitute for measurement of resonant frequency and length change as indicators of distress in test specimens, it is a useful adjunct to durability testing and offers the possibility of new insight into concrete behavior. Additional studies of its applicability should be performed.
Product No. 2019
Documentation: *Durability Testing of Concrete and Aggregates—Users' Manual* (forthcoming)

Impact-Echo Technique for Flaw Detection

A research program started at the National Institutes of Standards and Technology (NIST) and continued under SHRP C-204, Non-Destructive Testing for Quality Control/Condition Analysis of Concrete, dealt with the development of the impact-echo technique for detecting flaws in concrete structures. This could include such flaws as incompletely consolidated concrete at joints and internal cracking as well as delaminations caused by rebar corrosion or debonded overlays.

The basic principle of the impact-echo technique is the use of simple mechanical impacts to generate stress waves instead of using bulky transmitting transducers. A transit stress pulse is introduced into the concrete by mechanical impact on the surface. The stress pulse propagates into the concrete along spherical wavefronts, which are reflected by internal interfaces or external boundaries. The arrival of these reflected waves (or echoes) at the surface where the impact was generated produces displacements, which are measured by a receiving transducer and recorded on a digital oscilloscope.

New developments in C-204 included the automated signal interpretation needed for recording hundreds of impact-echo test points as, for example, would be recorded during an inspection of a bridge deck. An artificial intelligence technique called a neural network was developed to automate signal interpretation. The results obtained from the network give the probability of a defect occurring within a given thickness of the structure. Instrumentation used for the impact-echo system consists of an impact source transducer and waveform analyzer or a portable computer with a data-acquisition card. The equipment is now commercially available. The impact-echo system appears very promising as a means of detecting flaws in concrete. The work carried out under C-204 should be applicable to pavements and bridge overlays. Its application to heavily reinforced and geometrically complex structures is apparently still being investigated and is beyond the scope of the inexperienced user at this time.
Product No. 2012
Documentation: Sansalone and Carino 1989; Carino and Sansalone 1990

High-Performance Concrete Mixes

High-performance concrete is defined with reference to the performance requirements of its intended use. It would represent a concrete that is superior to conventional mixes currently being used for a particular application in terms of its ultimate strength, rate of strength gain, durability, or other particular properties necessary for superior performance. A variety of concrete mixes that could be classified as high performance were developed under SHRP C-205. These included mixes that could be used for rapid opening of pavements or pavement repairs, very high strength concretes for increasing span lengths of bridge beams and other structural applications, and fiber-reinforced concretes capable of improving ductility and wear resistance of riding surfaces. The following mixes were developed:

- Very Early Strength (VES) concrete. This mix is capable of achieving strengths of 2,000 psi (13.8 MPa) within 4 hours of being batched. The early strength is achieved through use of a high cement content, a high-range water reducer (melamine based), and a nonchloride accelerator. The mix may be made with local materials, but trial batches must be prepared to determine whether local materials will meet the desired rate of strength gain. Under hot placement conditions, the concrete may be cured using standard pigmented curing compounds; but, under more temperate conditions, curing blankets may be needed. An alternative mix developed under C-205 uses Pyrament cement to effect rapid rate of strength gain. Field experiments under C-206 indicated that concretes produced using Rapid-Set cement were an alternative to the use of Pyrament and showed the most rapid rate of strength gain of all mixes studied, reaching more than 3,000 psi (21 Mpa) in less than 3 hours. As part of the trial batching process for VES mixes, durability testing should be carried out because not all VES mixes were found to be equally durable in both projects.

- High Early Strength Concrete (HES). This mix was developed by the C-205 investigators to be capable of reaching a strength of 5,000 psi (34 MPa) within 24 hours. The mix ingredients and proportions are similar to VES concrete in that a high cement content (Type III), a high-range water reducer (naphthalene-based), and a nonchloride accelerator are used. Because the required rate of strength gain is less than for the VES mix, less accelerator may be used. Although the specification for this mix calls for 5,000 psi (34 MPa) at 24 hours, work conducted under C-206 indicated that the mix reached compressive strengths and modulus of rupture, which should be suitable for purposes of early opening 6–8 hours after placement. While the mix appears less sensitive to changes in materials and proportions than VES, trial batches and durability testing should be carried out with local materials before the mix is used on any given job.

- Very High Strength Concrete (VHS). These concretes are defined as mixes capable of reaching strengths of 10,000 psi (69 MPa) within 28 days. Such concretes have their greatest potential for application in the highway field for structures in which their high strengths can be used to reduce dead load, increase span length, or reduce the number of support elements. When used in elements directly exposed to both traffic and deicing salts, the high strengths coupled with low permeability should help to extend the service life of these structures. By using combinations of Type I cement, fly ash, silica fume, and high-range water reducers, mixes that meet these criteria were developed under C-205. It should also be noted that such mixes are becoming commercially available in many locations (Burg and Ost 1992); although they are being used for the most part for high-rise buildings, with some modification (primarily the inclusion of air entrainment) the commercial mixes should be readily adaptable to highway construction.

- High Early Strength Fiber-Reinforced Concrete (HESFRC). There are several advantages to reinforcing concrete with uniformly dispersed and randomly oriented fibers. These advantages include improvement in (1) ductility, (2) impact resistance, (3) tensile and flexural strength, (4) fatigue life, and (5) durability and abrasion resistance. The most significant result of incorporating steel fibers in concrete is to delay and control the tensile cracking of concrete. This crack-controlling property of the fiber reinforcement, in turn, delays the onset of flexural and shear cracking, imparts improved postcracking behavior, and significantly enhances the ductility and energy absorption properties of the concrete. These properties make FRC a promising material for pavement construction. Work under C-205 has extended the application of FRC to high early strength mixes so that the enhancement of mechanical properties afforded by fibers can be obtained at earlier ages.

Product No. 2014

Documentation: Mechanical Behavior of High-Performance Concrete: Volume 1 Summary Report (SHRP-C-361); *Volume 2 Production of High-Performance Concrete* (SHRP-C-362); *Volume 3 Very Early Strength Concrete* (SHRP-C-363); *Volume 4 High Early Strength Concrete* (SHRP-C-364); *Volume 5 Very High Strength Concrete* (SHRP-C-365); *Volume 6 High Early Strength Fiber-Reinforced Concrete* (SHRP-C-366)

Mechanical Testing of High-Performance Concrete

Testing for Compressive Strength

Current procedures for determining the compressive strength of cylindrical concrete specimens were developed for the testing of concretes with conventional strength levels, generally not more than 8,000 psi (55 MPa). Concretes with higher strengths are more sensitive to small changes in test procedures and require considerable extra care and some modification in procedures to produce accurate results. The capping of specimens to obtain planar end conditions is one area in which modifications to existing procedures have yielded more reproducible results. While sulfur-based capping compounds can be used for strengths up to 15,000 psi (100 MPa), the particular compound used appears to affect results. At strengths higher than 15,000 psi (100 MPa), the surface grinding of specimens is a more appropriate procedure. As an alternative to surface grinding, polymeric end caps restrained in steel rings can be used. AASHTO T-22 currently allows such caps. Neoprene end caps with a durometer hardness of 70 can be used for strengths up to 12,000 psi (80 MPa). Above 12,000 psi (80 MPa), polyurethane caps with a durometer hardness of 90 are recommended.

Product No. 2024

Documentation: *Mechanical Behavior of High-Performance Concrete: Volume 1 Summary Report* (SHRP-C-361); *Volume 4 High Early Strength Concrete* (SHRP-C-364)

Testing for Flexural Strength

As is the case for compressive strength, current procedures for determining flexural strength (modulus of rupture) were developed for conventional concretes. A modification to the current test procedure (AASHTO T 97) allows for obtaining additional information on behavior of concrete beyond the point at which sudden failure will occur in the standard test on plain (unreinforced) beams. The modification involves inclusion of a reinforcing bar within the specimen so that stress is transferred to the steel to prevent failure of the

specimen. In this manner, flexural strains can be measured with the use of differential transducers mounted on the specimen, and data on flexural strain capacity may be obtained. This test is not viewed as a replacement for standard testing on plain beams but rather as a procedure that can be used in developmental studies on new materials.
Product No. 2023
Documentation: *Mechanical Behavior of High-Performance Concrete: Volume 1 Summary Report* (SHRP-C-361); *Volume 4 High Early Strength Concrete* (SHRP-C-364)

Testing for Interfacial Bond Strength

As part of SHRP C-205, a procedure for determining interfacial bond strength and slip between existing and newly placed concrete was developed. The test uses an inverted L-shaped reinforced concrete section bonded to an upright L-shaped concrete section. The sections are relatively massive and are more typical of specimens that would be used for research studies on structural elements than of those sizes more commonly associated with concrete materials testing. Procedures for determining shear strengths between overlays and concrete substrates have been discussed in detail by Knab et al. (1989). These procedures utilize either cores or cylinders up to 4 in. (100 mm) in diameter; they are also more appropriate for screening materials and testing replicate samples, activities important in the typical agency or independent materials test lab. The procedure developed under C-205, therefore, must be viewed as a useful research tool for those interested in developing information on the structural behavior of concrete/overlay interfaces, but not as a technique for readily determining bond strengths between any two given materials.
Product No. 2025
Documentation: *Mechanical Behavior of High-Performance Concrete: Volume 1 Summary Report* (SHRP-C-361); *Volume 4 High Early Strength Concrete* (SHRP-C-364)

4

Field Studies of New Test Procedures, Materials, and Criteria for Concrete Rehabilitation

Objectives

Project C-206 included a field validation task. The objectives of these field studies were threefold: first, to evaluate materials and concrete mixtures developed under the SHRP program and elsewhere and obtain a complete set of data on their utility and physical properties for the various rehabilitation applications chosen. Second, new test methods that can aid in improved quality control of concrete and that can be used to determine in-place strength by nondestructive methods were to be evaluated using the field experiments as a platform for this evaluation. These methods included determination of batch water content using a microwave oven procedure, field measurement of concrete strength obtained from test cylinders cured by means of a temperature-matched curing procedure, prediction of in-situ strength using the maturity approach (ASTM C 1074), and prediction of in-situ strength using pulse velocity techniques (ASTM C 597). Finally, to develop criteria for early opening of full-depth concrete pavement repairs, both strength gain with time and traffic loadings on the repaired areas were monitored and used to conduct a fatigue analysis that identified the minimum strength necessary to allow traffic to move safely on the repairs. From a knowledge of strength gain with time for any set of materials, the material that meets the time requirement for early opening of a particular application could then be chosen.

Selection of Applications for Field Studies

From information developed during the preparation of the synthesis (Whiting et al. 1993) and evaluation of the various application areas in the field of concrete highway construction, two areas readily emerged as the most important and needed technologies:

- durable (penetration-resistant) bridge deck overlays for concrete structures;
- early opening of rehabilitated concrete pavements to traffic.

Given the alarming rate at which the nation's bridges are deteriorating and the enormous annual expenditures on bridge maintenance, durable bridge deck overlays easily ranked as one of the most important technologies to be developed. Approximately one-half the cost of bridge deterioration is related to concrete bridge decks, with much of the deterioration the result of the penetration of concrete by deicing salts and the corrosion of the reinforcing steel. A recent report (Transportation Research Board 1991) indicates that within the next 10 years, between 2.5 and 5 million ft^2 (230 and 460 thousand m^2) of bridge deck surface area will require rehabilitation each year. At repair costs ranging from \$20 to \$40/ft^2 (\$215 to \$430/m^2) this would result in an annual repair cost of \$50 million to \$250 million for the nation's bridge decks. Despite significant increases in bridge rehabilitation and replacement efforts by the states, there has been essentially no reduction in the backlog of deficient bridges; durable, penetration-resistant bridge deck overlays are therefore of particular interest to highway agencies in areas of the country where deicing salts are used regularly.

In recent years, early opening of concrete pavements to traffic has also been given much emphasis. As documented in the synthesis, many of the recent developments in materials and processes for concrete paving focus on early opening. Fast-Track technology (Pearson 1988) allows concrete pavements to be constructed or repaired and then opened to traffic in 4–24 hours. This is considered a most important and needed technology that would allow more use of concrete for such repair applications.

Recognizing the importance of early opening technology, FHWA initiated Demonstration Project SP 201, Accelerated Rigid Paving Techniques, in 1988 to promote the development of Fast-Track techniques. Several pilot projects have been completed under this program with the assistance of the FHWA. The problems that remain to be solved before the Fast-Track technology can become more widely used include the following:

- determining slab strength accurately and quickly;
- establishing minimum strength of concrete at opening for various applications.

While several highway agencies specify minimum strength for opening to traffic, the effects of early opening on performance of concrete pavements has never been formally investigated. To ensure adequate service life of early opening projects, the minimum required strength should be established on the basis of reliable performance data. Improved methods of determining the in-place strength of concrete are also needed.

On the basis of information presented above, two field experiments were conducted in conjunction with SHRP C-206 to obtain more information in these important application areas:

- A field experiment to evaluate materials and processes for constructing durable and penetration-resistant bridge deck overlays.
- A field test to evaluate materials and processes for early opening of concrete pavement repair and rehabilitation projects.

In the concrete pavement area, the various types of repair applications were reviewed and the most important application was selected to serve as the testing platform. By ranking the applications according to the frequency with which these procedures are used, the following order was obtained:

- full-depth repair/slab replacement;
- partial-depth repair;
- reconstruction/unbonded concrete overlays;
- bonded concrete overlays.

On the basis of this ranking, full-depth repair was selected as the testing platform for evaluating early opening materials and processes under C-206. While it was recognized that much effort also goes into partial-depth repairs, this topic was being addressed in detail by SHRP H-105 and H-106, and the consensus of the research team was that further work was not needed at this time.

Bridge Deck Overlays

Overlay Technology

The continued deterioration of reinforced concrete bridge decks brought about by chloride-induced corrosion of the reinforcing steel has led to increased use of rigid concrete bridge deck overlays. Besides periodic painting of steel support beams, the placement of deck overlays is now perhaps the most common maintenance procedure performed on concrete bridges. By placement of a concrete overlay, the riding surface is restored and further penetration of chloride salts into the deck is reduced. To meet these objectives, the overlays must be strong, must be well bonded to the underlying concrete, and must have low permeability. Special materials used consist of latex modifiers (primarily styrene-butadiene type), silica fume, and high-range water reducers. Bonding grouts may be separately applied, although the most common practice is to broom out some of the mortar fraction of the concrete onto the surface just ahead of the paver.

Latex-modified concrete (LMC) overlays are one of the most widely used methods and have generally exhibited good performance in reducing chloride penetration. Mix designs used for LMC overlays typically call for 658 lb/yd^3 (387 kg/m^3) of cement, relatively low water content (w/c, 0.39 or less), and 15% (by cement weight) of a 46% emulsion of styrene-butadiene latex. In a study of 132 bridge decks, Bishara (1980) reported much lower chloride contents in decks overlaid with LMC than in unprotected decks. Studies by Indiana, Kentucky, and Michigan (Babaei and Hawkins 1987) show equally good performance for LMC. Distress in LMC overlays in Ohio, however, was reported by Abdulshafi, Kvammen, and Kaloush (1990), who attributed problems to ongoing corrosion in the original deck as well as deficiencies in construction practices. LMC mixtures have been modified to obtain overlays that can be opened to traffic within 24 hours. This has been accomplished through the use of Type III cement, a lower w/c, and a higher cement content than are normally employed in LMC mixes to produce early opening latex-modified concretes (LMC-III). Published data on the effects of these changes to conventional LMC mixes, however, are limited (Howard 1988; Sprinkel 1988). Deck preparation and curing techniques should follow those used for other LMC jobs. As with other early opening applications, although post-construction verification testing would be beneficial, it may not be possible to allow for additional closure time in many such instances. Additional study of such early opening overlays would be very beneficial, as traffic could be allowed back onto bridges in a much shorter time after repairs had been carried out.

Silica fume concrete (SFC) overlays utilize a relatively high cement content of 600–700 lb/yd^3 (360–420 kg/m^3), low w/c (in most cases, less than 0.40), and sufficient high-range water reducer to obtain a workable concrete mixture. Additions of silica fume have ranged from 5% to 15%, based on cement weight. Laboratory data (Marusin 1986) indicate that reduction in permeability to chloride ions is a function of silica fume addition, the greatest reductions occurring at additions greater than 10% by weight of cement. SFC is very resistant to penetration of chlorides, with laboratory ponding tests carried out during 2½ years (Ozyildirim 1993) indicating that chloride contents at 1.75 in. (44 mm) remained significantly below threshold corrosion levels of 1.3 lb/yd^3 (0.8 kg/m^3). Short-term field data (Whiting and Dziedzic 1989) are promising, but long-term field performance data are lacking.

Research Plan

Two states were chosen for evaluation of their LMC-III and SFC overlay systems. Within each state, two test sites were selected. Each site consisted of twin bridges so that LMC-III could be placed on one bridge and SFC on the other bridge. Four bridge sites were thus included in the experiment, with a total of eight test sections. At each site, aggregate materials were identical for the LMC-III and SFC mixes.

Sites selected for the overlay placements were the following:

- Site 1—This site consists of twin bridges located on US 52 over Twelve Mile Creek just west of New Richmond, Ohio. These are three-span bridges supported over continuous structural steel I-beams and crossframes. The deck in the westbound lanes was overlaid with SFC; that in the eastbound lanes, with LMC-III.
- Site 2—This site consists of twin bridges located on I-270 over Raymond Run, Columbus, Ohio. These are very short single-span bridges simply supported at the abutments. The northbound lanes of the test structure were overlaid with SFC; the southbound lanes, with LMC-III.
- Site 3—This site consists of twin bridges located on I-265 over Kentucky State Highway 22, northeast of Louisville. Both structures are three-spanned bridges supported on steel I-beams. The northbound lanes of the test structure were overlaid with LMC-III; the southbound lanes, with SFC.
- Site 4—This site consists of twin bridges located on US 41 over Kentucky State Highway 351 on the Henderson city bypass route. Both structures are three-span bridges of

reinforced concrete composite deck/girder construction. The northbound lanes of the test structure were overlaid with LMC-III; the southbound lanes, with SFC.

More detailed site descriptions as well as specific materials and mixes used at each site are included in Appendix A. Results of field testing and laboratory testing of specimens cast from the field mixes are presented after the site descriptions.

Placements and Test Procedures

Most aspects of the placements went quite well, with contractors closely following state specifications on placement of the overlays. There were a few exceptions, however, that may affect the ultimate performance of the overlays. At Site 2, higher than specified amounts of water were discharged from the mobile concrete mixers for at least part of the placement, resulting in a w/c ratio of 0.48, as opposed to the specified w/c of 0.39. At Site 3, water used to wet the deck prior to overlay was excessive, and some of the water in variable-depth repair areas was not fully removed prior to placement of the concrete. Finally, at Site 4, bonding grout was applied nonuniformly.

A variety of test procedures was carried out at the overlay sites. These included both field test procedures and testing of specimens prepared in the field and transferred back to the laboratory. A synopsis of each procedure follows, with further details provided in Appendix A.

- Determination of water content. A microwave oven drying technique was used to determine the water content of concrete at Sites 1 and 3. Development of the technique is fully described in Chapter 3 of this report.
- Strength gain estimation using the maturity method. Relationships between time/temperature and compressive strength were determined using procedures described in ASTM C 1074, *Standard Practice for Estimating Concrete Strength by the Maturity Method*. These procedures were applied only to the LMC-III overlays because strength development during the first 24 hours was critical for the targeted 1-day opening time.
- Tensile bond testing. Development of bond strength is one of the most important overlay characteristics, especially for those scheduled for early opening to traffic. The bond of overlay to substrate was determined by using a tensile pull-off technique similar to the test for surface adhesion described in ACI 503R-89. Tests were carried out at Sites 1, 3, and 4. Additionally, cores were taken at all sites through the substrate concrete and used to determine shear bond strength.
- Compressive and split tensile strength. Duplicate test cylinders were given standard field cure, transported to the laboratory and tested at 3, 7, 28, and 90 days. SFC concretes were moist cured; LMC-III concretes were moist cured for 1 day, then air cured.
- Modulus of elasticity. Static elastic modulus was determined at 7 and 28 days before compressive strength tests were performed. ASTM C 469 procedures were used.

- Durability testing. Testing included freeze-thaw testing (AASHTO T 161—Procedure B [modified]) of beams cast on site, permeability (AASHTO T 277) on both site-cast cylinders and cores removed from the hardened overlays, and deicer scaling resistance (ASTM C 672) performed on cores removed from the overlays.

Test Results

Determination of Water Content via Microwave Drying

Results of water content determinations at Sites 1 and 3 are shown in Table 4.1. Tests were not carried out at Sites 2 and 4 because the testing equipment was being used for other purposes at the time these placements were made. As shown in Table 4.1, measured water contents were generally very close to values reported from batch tickets. At Site 3, however, there were significant deviances, mainly for the latex mixes produced using mobile concrete mixers. Mobile concrete production is subject to greater calibration errors than weigh batching, which may help to explain some of the discrepancies. In addition, during production of latex overlays, contractors frequently adjust the settings on the mobile mixer to improve the workability of the concrete.

Table 4.1. Field water contents determined using microwave drying.

Site No.	Mix Type	Reported Total Water (lb/yd^3)	Measured Total Water (lb/yd^3)	Difference (% of reported)
1	SFC	321	316	−1.55
	LMC-III	285	273	−4.20
3	LMC-III	219	233	+6.39
	LMC-III	218	246	+12.84
	SFC	263	253	−3.80
	SFC	263	265	+0.80

Note: 1 yd^3 = 0.7645 m^3

Strength Gain Estimation Using Maturity Method

To estimate the in-place strength of concrete in the overlays based on maturity concepts, the concrete temperature was continuously monitored and the in-place maturity was computed using procedures described in ASTM C 1074 (see Appendix A). The temperatures of the overlays were monitored throughout the first 24 hours after placement. In the early spring placements at Sites 1 and 2, overlay temperatures did not exceed 75°F (24°C) during the first 24 hours after placement. At Sites 3 and 4, overlays were placed in the summer, and temperatures in the overlays approached 90°F (32°C).

Field cylinders were also monitored for temperature, were tested in the field at various times, and were used to establish the maturity/strength relationship for each overlay. Time/strength curves over the full 24-hour curing period were generated by using these relationships. Strengths predicted at 24 hours using this technique are shown in Table 4.2 and compared with strengths measured at 24 hours on cylinders placed underneath the burlap used to cure the overlay.

The low strengths experienced at Site 2 can be attributed to the increased amount of water in the mix as well as to temperatures that fell to near 40°F (4°C) the night after placement. Predicted strengths are within 20% of values measured on the field cylinders. It is likely that some of this difference is the result of the larger thickness of the test cylinder as opposed to the overlay and the insulating effects of the mold, which led to higher temperatures in the cylinder than in the overlays at each site.

Table 4.2. Strength prediction using maturity method for LMC-III overlays.

Site No.	Predicted Strength at 24 Hr (psi)	Cylinder Strength at 24 Hr (psi)	Difference (%)
1	2,300	2,650	−15
2	620	700	−13
3	4,900	4,900	0
4	3,320	3,920	−18

Note: 1 psi = 0.0069 MPa

Bond Testing

Bond pull-off was measured for the LMC-III overlays at Sites 1 (US 52), 3 (I-265), and 4 (US 41). Tests were not carried out at Site 2 because the overlay failed to reach its intended 24-hour strength. This was attributed to the high water content in the mix. Results of tensile bond tests are summarized in Table 4.3. Although no criteria have been established for this particular test, previous experience has shown that a minimum bond pull-off strength of 100 psi (0.7 MPa) after 24 hours of field cure indicates a concrete with the potential to achieve excellent long-term bond strength. Cores obtained from the overlay and tested in the lab using a direct shear technique (Table 4.4) support this conclusion: laboratory work (Knab, Sprinkel, and Lane 1989) carried out on bond strength of latex-modified concretes to concrete substrates indicates a minimum strength of 200 psi (1.4 MPa) using a direct shear bond test method is a reasonable criterion.

Table 4.3. Tensile bond strength at 24 hours for LMC-III overlays.

	Test No. (psi):			
Site No.	1	2	3	Average (psi)
1	15	100	130	108
3	105	95	*	100
3	160	150	*	155
4	85	165	155	135

Note: 1 psi = 0.0069 MPa
*Only two tests carried out at this location.

Table 4.4. Direct shear strengths of cores taken from bridge overlays.

		Shear Strength (psi) of:		
Site No.	Mix	Specimen 1	Specimen 2	Average
1	LMC-III	525	500	515
	SFC	485	565	525
2	LMC-III	435	320	380
	SFC	445	320	385
3	LMC-III	265	300	285
	SFC	350	345	350
4	LMC-III	195	230	215
	SFC	295	295	295

Note: 1 psi = 0.0069 MPa

Strength and Elastic Modulus Testing

Complete compressive strength test data through 90 days are summarized in Appendix A. All strengths appeared more than adequate, inasmuch as compressive strength is not a critical determinant of overlay performance. More important are bond of the overlay to the substrate and permeability of the overlay to chloride ions. The difference between LMC-III and SFC strengths are illustrated by the comparison data in Table 4.5. for Sites 2 and 4. The behavior illustrated for Site 2 was typical also for Sites 1 and 3, that is, much higher strengths were recorded for the SFC mixes at any given age.

For Site 4, however, differences between LMC-III and SFC were not as great. In spite of the fact that strengths for SFC were lower at Site 4, they were still more than adequate for the intended application. At all sites, strengths of cylinders cast from replicate batches fell within

5% of those of cylinders cast from initial batches at 28 days, indicating adequate batch-to-batch control.

Table 4.5. Comparison of compressive strengths of overlays.

Site No.	Age (days)	Compressive Strength (psi) of: LMC-III	SFC
2	1	3,120	*
	7	4,720	8,750
	28	5,840	10,670
	90	6,050	11,740
4	1	3,780	*
	7	5,050	5,810
	28	6,040	7,100
	90	7,100	7,160

Note: 1 psi = 0.0069 MPa
*Not determined

Split tensile strengths showed similar trends, although there were fewer differences between LMC-III and SFC mixes. After 7 days, all strengths fell between 600 and 800 psi (4 and 5.5 MPa). The highest LMC-III tensile strengths were obtained with the Site 3 mix, and consistently lower strengths were obtained with the Site 2 mix. In the case of SFC, the highest strengths were obtained from the Site 3 and Site 2 mixes, and the lowest from the Site 4 mix.

Static elastic modulus of elasticity was measured at 7 and 28 days. For any given location and age, moduli of SFC concretes were consistently higher than those of LMC-III concrete, which is to be expected given the difference in strengths. Moduli followed the same basic trend as for compressive strengths. For LMC-III mixes, lowest moduli and strengths were obtained at Site 2, highest at Site 3. For SFC, lowest moduli and strengths were recorded at Site 4, highest at Site 3.

Durability Testing

Test specimens were prepared for evaluation of freeze-thaw resistance testing using modified procedures developed under SHRP C-203. The modified procedures consist essentially of an AASHTO T 161 B method (freeze in air/thaw in water) with the specimens wrapped in towels during the air freeze to reduce drying from the surface during the freeze cycle. Janssen (forthcoming) has found this technique to be no less, and perhaps somewhat more, severe than Procedure A. Results (average of three specimens) after 300 cycles of testing are presented in Table 4.6. All mixes show acceptable behavior through 300 cycles. Air-void analyses carried out on the SFC mixes (LMC-III mixes being non-air entrained) showed acceptable air void systems for all the concretes. Deicer scaling resistance tests carried out on cores taken from the overlays showed acceptable behavior for all sites except the LMC-III overlays at Sites 1 and 3, where severe scaling was observed. Because field experience does not indicate scaling to be a widespread problem with LMC overlays, long-term observations of these test sections will help resolve this apparent discrepancy.

Table 4.6. Resistance to freezing and thawing of overlay concretes after 300 cycles of testing (modified AASHTO T 161B).

Site No.	Type	Durability Factor	Expansion (%)	Mass Change (%)
1	LMC-III	102	0.01	+0.40
	SFC	93	0.01	+0.07
2	LMC-III	96	0.03	+0.54
	SFC	88	0.05	+0.14
3	LMC-III	100	0.02	+0.14
	SFC	98	0.01	+0.12
4	LMC-III	100	0.02	+0.27
	SFC	99	0.00	-0.05

Permeability is widely accepted as a measure of the durability of the concretes used in bridge structures: the time needed for chloride ions to reach the reinforcing steel is inversely proportional to the permeability of the concrete. An indication of relative permeability of the overlay concretes was obtained using the rapid determination of the chloride permeability of concrete procedure (AASHTO T 277). Tests were carried out at 28 days. Test results are summarized in Table 4.7 as the average of two specimens. Results are in the ranges of what normally is expected with LMC and SFC mixes. Typically, LMC exhibits charge passed in the range of 500 to 1,500 coulombs; SFC permeabilities are significantly less, always less than 1,000 coulombs.

Table 4.7. AASHTO T 277 relative permeability results.

Site No.	Type	Charge Passed (coulombs)
1	LMC-III	1,364
	SFC	460
2	LMC-III	123
	SFC	349
3	LMC-III	680
	SFC	308
4	LMC-III	915
	SFC	555

Follow-Up Surveys of Bridge Deck Overlays

Follow-up surveys were carried out in the fall of 1992 after all overlays had been placed. At each site, the overlay surface was visually inspected for signs of distress such as cracking, spalling, or punchouts. The overlays were also sounded with a chain drag and steel rod to acoustically detect subsurface delaminations that could indicate unbonded areas beneath the overlay.

All overlays appeared to be in relatively good condition. Some overlays exhibited minor narrow cracking, and some showed surface blemishes attributable to finishing technique. The only major deficiencies were those observed at Site 4 (US 41), where poor bond was evidenced at the longitudinal construction joint between adjacent overlay sections; this resulted in hollow areas along the joint.

The large amount of baseline information developed for these test sections will make them ideal candidates for continued observation and testing. It is expected that future monitoring of the sections by state forces or as part of FHWA research programs will develop long-term data that will reflect material durability and the ability of the overlays to carry traffic over long periods of time. State forces or other agencies that will assume these functions for SHRP should make annual surveys of the bridge deck overlay test sections. These surveys should include visual observations, delamination detection, and chloride sampling in the overlay material. Data for traffic across the bridges, if available, would also help interpret overlay performance.

Conclusions

The series of placements and subsequent testing carried out under this phase of the project demonstrated that bridge deck overlays offer a good solution to the problem of bridge deck deterioration. With use of special mixes, the overlays achieved sufficient strength in 24 hours to be opened to traffic. Their low permeability and good bond strengths should assure long-term performance, provided that proper care is taken in the placement and curing of the overlays. The following conclusions were based on the results of testing carried out in the field and laboratory as well as on initial post-construction surveys. It should be noted that these conclusions are strictly valid only for the particular materials and mixes used in this research program. Extrapolation of these conclusions to other materials or circumstances should be done with caution only after consideration of the particular conditions that may be encountered.

- Bridge deck overlays that utilize latex-modified concretes can be opened to traffic within 24 hours after placement if Type III (High Early Strength) cement is substituted for Type I cement in the mix. Strengths at 24 hours should exceed 2,000 psi (14 MPa) using this approach, provided that w/c is maintained below 0.40 and temperatures during the first 24 hours after placement do not fall below 50°F (10°C).

- Durable concrete overlays having very low permeabilities can be produced using silica fume concrete (SFC). Compressive strengths of SFC mixes will generally be significantly higher than latex-modified concretes (LMC) used for overlays, while tensile strengths of the two systems are roughly equivalent. For the particular materials tested at the four bridge sites in this program, permeability to chloride ions—as measured by the AASHTO T 277 test procedure—was generally lower for SFC than for LMC mixes.

- Provided that surfaces are properly prepared, good bond of overlays to substrate can be obtained using either SFC or LMC overlays. Shear bond strength in excess of 200 psi (1.4 MPa) at 28 days can easily be achieved with either system.

- The key to successful performance of concrete overlays is careful surface preparation followed by installation by experienced and conscientious personnel. Mix quantities must be verified for each batch of concrete delivered; if mobile mixers are used, they should be calibrated and checked prior to use. Operations should proceed without delay, and curing must be initiated immediately after final texturing of the surface. Whenever possible, soaker hoses should be used to keep burlap curing mats continuously wetted throughout the period of moist curing.

Full-Depth Pavement Repairs

Repair Technology

The purpose of full-depth repair of concrete pavements is to restore structural integrity and improve the rideability of concrete pavements having certain types of distresses that cannot be corrected by using partial-depth repairs. Working cracks and badly deteriorated joints are the most common problems that require full-depth repairs or slab replacements (Darter, Barenberg, and Yrjanson 1985; Snyder, Reiter, and Hall 1989).

Full-depth repair and slab replacement are used to address several types of distresses that occur at or near transverse cracks and joints. These include spalling, d-cracking, failure of joint load transfer devices, slab breakup (corner breaks or diagonal cracks near the joint), and breakup of the slab into several pieces (Darter, Barenberg, and Yrjanson 1985; Snyder, Reiter, and Hall 1989). A detailed description of these distresses is given by Darter, Barenberg, and Yrjanson (1985) and the Transportation Research Board (1979). The severity of the distresses is the main criterion by which the decision to repair is made and repair size is determined. In general, low-severity distresses do not require full-depth repair within the first 2 years of onset.

The performance of full-depth repairs on in-service pavements has been inconsistent (Snyder, Reiter, and Hall 1989). Although many repairs have provided satisfactory performance, many others have failed within 1 year of construction. The major causes of these premature failures have been faulty design (particularly poor load transfer design), poor installation conditions, and poor construction quality (Darter, Barenberg, and Yrjanson 1985; Snyder, Reiter, and Hall 1989; Hall et al. 1989). A 1987 field survey by FHWA has shown that, if properly designed and constructed, full-depth repairs can provide good performance for 10 or more years. Numerous factors affect the performance of full-depth repairs, including repair dimensions, concrete removal method, drainage condition, load transfer design, repair materials, traffic, and condition and quality of construction (Snyder, Reiter, and Hall 1989).

The materials used in full-depth repairs to date have predominantly been conventional concrete mixes with high cement content. Typical patch mixes contain between 658 and 940 lb/yd^3 (390 and 558 kg/m^3) of cement. Depending on the required opening time and availability, Types I, II, or III portland cement are used. Opening times can range from 6 to 48 hours after placement, unless an accelerating admixture is used (Darter, Barenberg, and Yrjanson 1985). Calcium chloride or other accelerating admixtures are used if early opening of the repair to traffic is desired.

Many states now allow early opening when Fast-Track mixes are used (Grove et al. 1990). The high early strength is typically obtained by using a high cement content, low w/c, and accelerating admixtures. A rich, low-water-content mix containing 1%–2% calcium chloride will produce adequate strength and abrasion resistance for opening to traffic in 4–5 hours at temperatures above 50°F (10°C) (Transportation Research Board 1977). The accelerators are added at either the ready-mix plant or the job site, depending on the temperature and distance to the plant. Other admixtures commonly used in repair mixes include air-entraining agents, water reducers, and superplasticizers.

It is now possible to perform full-depth repair of concrete pavements and open the pavement to traffic in 4 hours or less (Munn 1989; Klemens 1990). This involves not only the use of Fast-Track mixes but also rapid techniques for concrete removal and dowel placement. A combination of innovative techniques has been used in New York to make full-depth repairs of concrete pavements overnight on the Long Island Expressway (Klemens 1990). Slab removal by liftout, the use of gang drills, and a Fast-Track mix were the key elements that made this possible. The maturity concept was used in the Long Island Expressway project to determine strength for opening as well as on projects in Iowa (Grove et al. 1990). Similar techniques have been used in Utah to make full-depth repairs on Interstate I-15 (Munn 1989). Utah used regulated-set portland cement for opening 5 hours after concrete placement. Other proprietary products, such as Pyrament cement, a blended cement using alkali-activated alumino-silicates (Malone et al. 1985), and Rapid-Set cement, a sulfo-aluminate-based cement, have the potential for allowing opening of repairs in less than 4 hours. High-performance concrete mixes developed under SHRP C-205 also have the potential for use when early opening of repairs is required. Evaluation of early opening of repairs made with conventional and proprietary materials was a major focus of this part of the research project.

Research Plan

Sites in two states were chosen for evaluation of concrete mixes to be used for early opening (2–24 hours) of full-depth pavement repairs. Within each state, a pavement test site was chosen. The sites were selected so that repairs could be staged over a number of hours to obtain a spectrum of opening times for each material. The repairs for any one material were then opened at the same time so that the last repair made would see the minimum opening time scheduled for the particular concrete mix under evaluation. Three categories of opening times were chosen: 2–4 hours, 4–6 hours, and 12–24 hours. Sites selected for the repair placements were the following:

- Site 5—This site was located on eastbound I-20 west of Augusta, Georgia. The test sections were added to a contract that was already in progress for full-depth repairs of working cracks in pavement slabs. A total of sixty repair sections were selected, with twenty repairs assigned to each of three concrete mixes chosen for evaluation at this site. Repair lengths ranged from 6 to 15 ft (1.8 to 4.6 m).
- Site 6—This site was located on the eastbound and westbound driving lanes of State Route 2 near Vermilion, Ohio. The test sections originally built on this project were used to evaluate various factors affecting d-cracking of concrete slabs. The test sections were added to a contract for full-depth repair and installation of new drains and shoulders that was just beginning at this site. A total of eighty repair sections were selected, with ten repairs assigned to each of eight mixes chosen for evaluation at this site. All repair sections were 6 ft (1.8 m) in length.

A total of nine separate early opening concrete mixes was evaluated in this field study. Mixes VES, Fast-Track, and GADOT were used at Site 5. Mixes PC1, RSC1, PC2, RSC2, VES, HES, Fast-Track, and ODOT were used at Site 6. Although the original intent of the research was to compare identical mixes at the two locations, this could not be achieved for two reasons: only three mixes could be utilized at Site 5 due to restrictions on placements of special concretes, and the small-sized aggregate required at Site 6 to avoid future d-cracking in the sections resulted in higher water demands for the concretes than originally intended. While this was offset to some degree by increases in cement content, most w/c had to be adjusted upwards from initial designs to afford proper workability for placement of the concretes. All mixes except PC1 and PC2 were air entrained. Aggregates at Site 5 were siliceous in nature. At Site 6, a limestone coarse aggregate and mixed siliceous/calcareous sand were used. A brief description of each of the mixes is given below. More detailed information on constituent materials and mix proportions and properties is given in Appendix A.

- PC1—This mix used 900 lb/yd^3(530 kg/m^3) of Pyrament PCB-XT cement at a w/c of 0.27. It was designed for a strength gain of 2,000 psi (13.8 MPa) in 4 hours. This mix was placed in the 2- to 4-hour opening category.
- RSC1—This mix used 750 lb/yd^3(445 kg/m^3) of Rapid-Set cement at a w/c of 0.40 and with citric acid as a set retarder. It was designed for a strength gain of 2,000 psi (13.8 MPa) in 4 hours, but in practice gained strength much more rapidly (see discussion below). The mix was placed in the 2- to 4-hour opening category.
- VES—This mix was developed under SHRP C-205 and was designed to achieve a strength of 2,000 psi (13.8 MPa) in 4 hours. As designed by C-205, it uses 870 lb/yd^3(520 kg/m^3) of Type III cement, a melamine-based high-range water reducer, and a chloride-free accelerator (calcium nitrite) at a w/c of 0.38. While these proportions were used at Site 5,

local materials at Site 6 required the use of 915 lb/yd^3 (540 kg/m^3) of cement to reach the desired strength. The mix was placed in the 4- to 6-hour opening category.

- PC2—This mix used 850 lb/yd^3(500 kg/m^3) of Pyrament PCB-XT cement at a w/c of 0.29. It was designed for a strength gain of 2,000 psi (13.8 MPa) in 6 hours. This mix was placed in the 4- to 6-hour opening category.
- RSC2—This mix used 650 lb/yd^3(380 kg/m^3) of Rapid-Set cement at a w/c of 0.50 and with citric acid as a set retarder. It was designed for a strength gain of 2,000 psi (13.8 MPa) in 6 hours, but in practice gained strength much more rapidly (see discussion below). The mix was placed in the 4- to 6-hour opening category.
- GADOT—This was a mix being used by the Georgia Department of Transportation on the I-20 repair project and is designated GADOT for the purposes of this report. This mix contained 750 lb/yd^3(445 kg/m^3) of Type I cement and 1% calcium chloride at a w/c of 0.38. It was designed to reach a strength of 1,000 psi (6.9 MPa) in 4 hours and was placed in the 4- to 6-hour opening category.
- ODOT—This mix was developed by the Ohio Department of Transportation and follows specifications for Class FS concrete for accelerated setting and strength development (Ohio Department of Transportation 1992). It uses 900 lb/yd^3 (530 kg/m^3) of Type III cement, 2% calcium chloride accelerator, and a Type B set-retarding admixture. The w/c was 0.40. The mix was designed to reach a modulus of rupture (MOR) of 400 psi (2.8 MPa) in 4 hours and was placed in the 4- to 6-hour opening category.
- Fast-Track—This mix was adapted from designs used by Iowa DOT for early opening applications (Grove et al. 1990). It contains 750 lb/yd^3 (445 kg/m^3) of Type III cement, as well as a Type A water reducer, and the mix is prepared at a w/c of 0.35. At Site 6, more water was needed to reach the needed workability, and the w/c was 0.49. This mix was placed in the 12- to 24-hour opening category.
- HES—This was the second mix developed by C-205 to be used in the field studies. It contained 870 lb/yd^3 (520 kg/m^3) of Type III cement, a chloride-free accelerator, and a naphthalene-based high-range water reducer. The w/c at Site 6 for this mix was 0.41. The mix was placed in the 12- to 24-hour opening category, although work at Site 6 later showed that rate of strength gain was such that repairs using this mix could be opened 6–8 hours after placement.

The research plan for these pavement sites also included further evaluation of test procedures used at the bridge deck overlay locations (microwave drying and maturity monitoring) as well as evaluation of additional techniques that could be used to determine strength gain in the repair areas. These procedures are listed in the following section. Cores were removed from repair areas upon reaching the desired strength as indicated by the in-place methods.

Placements and Test Procedures

Preparation for full-depth repairs was similar at both Sites 5 and 6. Both projects involved making full-depth repairs on existing 9-in. (230-mm) jointed plain concrete pavement (JPCP).

Work involved full-depth saw cutting at the joints, removal of the slab by lift-out, drilling of dowel holes by automatic ganged dowel bar drills, and dowel insertion and epoxy grouting. Plastic grout-retention disks were used. Work at Site 5 was carried out at night, work at Site 6 during daylight hours. Temperatures during placement ranged between 60° and 80°F (15° and 25°C).

The major difference between the operations at the two sites was the problem in control of concrete workability at Site 6 as opposed to Site 5. As previously noted, the work at Site 5 was ongoing, and contractors had learned that to work with early opening mixes, the concrete must be batched quickly, rapidly transported to the site, and discharged and finished in the most expedient manner. At Site 5, a well-controlled, permanent ready-mix yard was located approximately 10 minutes from the job site. At Site 6, the concrete batch plant was even closer to the site, but it was a temporary plant set up for supplying concrete to the job on SR 2. Control problems were evident, and many attempts were made to adjust water content and other proportions at the plant prior to transporting concrete to the job, resulting in delays and loss of workability of the mixes. In addition, the special cements used were obtained by the concrete supplier in bag form, resulting in having to add the cement bag-by-bag to the hopper of each ready-mix truck, causing further delays. After these problems were resolved, operations at Site 6 ran more smoothly, indicating that experience with these types of mixes is needed before they can be reliably utilized for fast-paced operations.

As for the bridge deck overlay sites, a variety of test procedures were carried out at the pavement repair sites. These included both field test procedures and the testing of specimens prepared in the field and transferred back to the laboratory. A synopsis of each procedure follows. Further details are provided in Appendix A.

- Determination of water content. The microwave drying procedure was used.
- Temperature development in test slabs. The first and last slab for each mix were instrumented with thermocouples placed 0.5, 5 (mid-depth), and 9 in. (13, 130, and 230 mm) below the surface of the pavement. These were used to monitor slab temperatures throughout the placement and to calculate maturity from which to obtain an estimate of strength development in the slabs.
- Strength gain prediction using pulse velocity. Expanded polystyrene blocks were attached to the subbase to allow for placement of pulse velocity transducers in the holes left by the blockouts after removal subsequent to set of the concrete. Pulse velocity was determined using equipment described in ASTM C 597 *Test Method for Pulse Velocity through Concrete*. Strength was estimated from relationships between pulse velocity, and strength was obtained from testing of specimens cast from laboratory trial batches prepared with the job materials.
- Temperature-matched curing (TMC). Test cylinders were maintained at the same temperature as the in-place concrete throughout the cure period by automatically controlling the temperature of heated molds with temperature sensors embedded in the placed concrete. Cylinders were tested by using a portable field compression tester.
- Compressive and split tensile strength. Duplicate test cylinders were given standard field cure, transported to the laboratory, and tested at 3, 7, 28, and 90 days. Additional test

cylinders were placed in an insulated curing box immediately after being cast on site to simulate strength gain in the thick pavement section.

- Modulus of elasticity. Static elastic modulus was determined at 7 and 28 days prior to carrying out compressive strength tests. ASTM C 469 procedures were used.
- Durability testing. Testing included freeze-thaw testing (AASHTO T 161—Procedure B [modified]) of beams cast on site and of cores removed from the repair areas just prior to opening. Cores were tested using Procedure A.

Test Results

A large body of data was generated at the two pavement repair sites. The full set of results is reported in Appendix A. In this chapter, only typical results will be presented for purposes of comparison and to demonstrate the properties of the various early opening mixes. For more complete test results, consult Appendix A.

Determination of Water Content via Microwave Drying

Water content of selected mixes was determined using the same microwave oven drying technique employed at the earlier bridge deck overlay placements. Examples of results are shown in Table 4.8. VES water contents are compared for Sites 5 and 6, along with the Fast-Track mix from Site 6. For most of the mixes measured, water contents fell within 4% of reported. In some exceptions, as can be seen from the VES replicate mix shown in the table below, water was apparently added to the mix to restore workability. The method appears promising as a quality control tool, although further evaluation on a wider variety of materials and locations needs to be done.

Table 4.8. Water contents determined at pavement repair sites.

Site No.	Mix Type	Total Water (lb/yd^3)		Difference (% of reported)
		Reported	Measured	
5	VES	357	358	+0.2
	VES (rep.)	359	355	−1.1
6	VES	424	449	+5.9
	VES (rep.)	408	444	+8.8
	Fast-Track	399	382	−4.3
	Fast-Track (rep.)	380	364	−4.2

Note: 1 yd^3 = 0.7645 m^3

Temperature Development in Test Slabs

To estimate strength gain from maturity data, a record of the temperature of the test sections throughout the period of cure was required. Temperature was measured at 30-minute intervals using thermocouples embedded at the depths previously noted. Typical profiles are shown in Figures 4.1 and 4.2 for the ODOT mix at Site 6 and the Fast-Track mix at Site 5. The use of high cement content and chemical accelerators resulted in a large temperature increase in the ODOT section. In addition, the temperature differential across the ODOT slab is much greater than that across the Fast-Track section. This can be attributed to the cooler air temperatures at the Ohio site (which would tend to cool the surface more quickly) and the higher starting temperature near the base of the slabs at the Georgia site. Highest peak temperatures recorded at Site 5 were close to 140°F (60°C) for the VES and GADOT mixes. In comparison, peak temperatures at Site 6 were much higher, ranging from 160° to 165°F (71° to 74°C) for the VES and ODOT mixes. Pyrament cement, used at Site 6 in mixes PC1 and PC2, generated the least heat of all the mixes studied, with peak temperatures lower than 110°F (43°C). These field data demonstrate that many early opening mixes have the potential to generate high temperatures during the first few hours after casting; such temperatures present a risk of slab cracking, but it must be kept in mind that it is the temperature differential (among other factors) that will lead to development of stresses, not the temperature itself. To minimize differentials, insulation mats can be used and kept in place until temperatures fall back to ambient levels and the concrete has gained sufficient tensile strength.

Early Strength Gain Predictions

Strength gain of in-place concrete was monitored using pulse velocity (PV), maturity, temperature-matched curing (TMC), and cylinders in an insulated curing box (I.C.B.). More complete descriptions of each of these methods and plots of strength gain versus time are included in Appendix A. Table 4.9 shows strengths predicted at opening using each of these techniques as compared to strengths obtained on cores taken from each slab at the times indicated. Results indicate that no one technique is consistently superior in predicting in-place strength. Maturity has the advantage that most of the predictions tend to be conservative; therefore, if maturity is used as a determinant of the opening time of the section, the slab will most probably have a greater strength than that predicted. However, accurate prediction using the maturity technique requires that job materials be obtained and maturity/strength relationships be derived in the laboratory prior to field operations, a somewhat laborious procedure. The more rapid COMA meter devices were evaluated, but found to be generally inaccurate in assessing equivalent age of the slabs (see Appendix A). The other techniques are mostly on the optimistic side in terms of prediction. There appears to be little benefit in using the more expensive thermal-matched curing system as opposed to storage of test cylinders in a simple insulated curing box. Given that a portable field compression tester is available, the insulated box approach may be the most direct as well as the simplest method to apply in early opening applications.

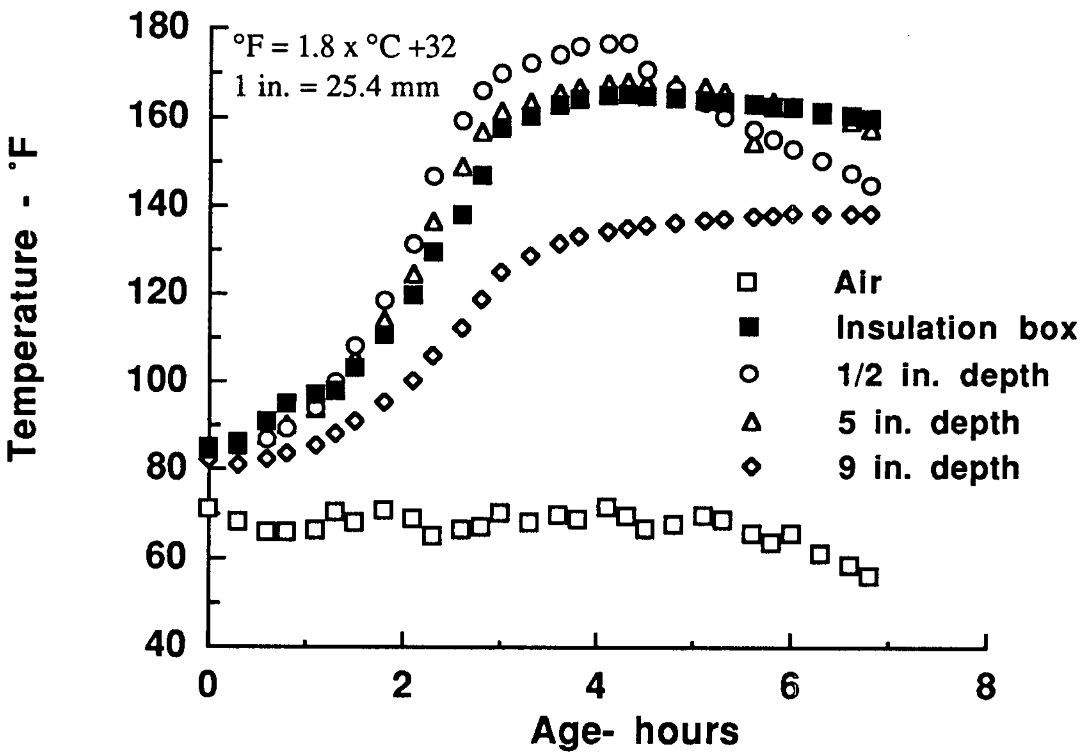

Figure 4.1. Temperature profiles for ODOT mix used at Site 6.

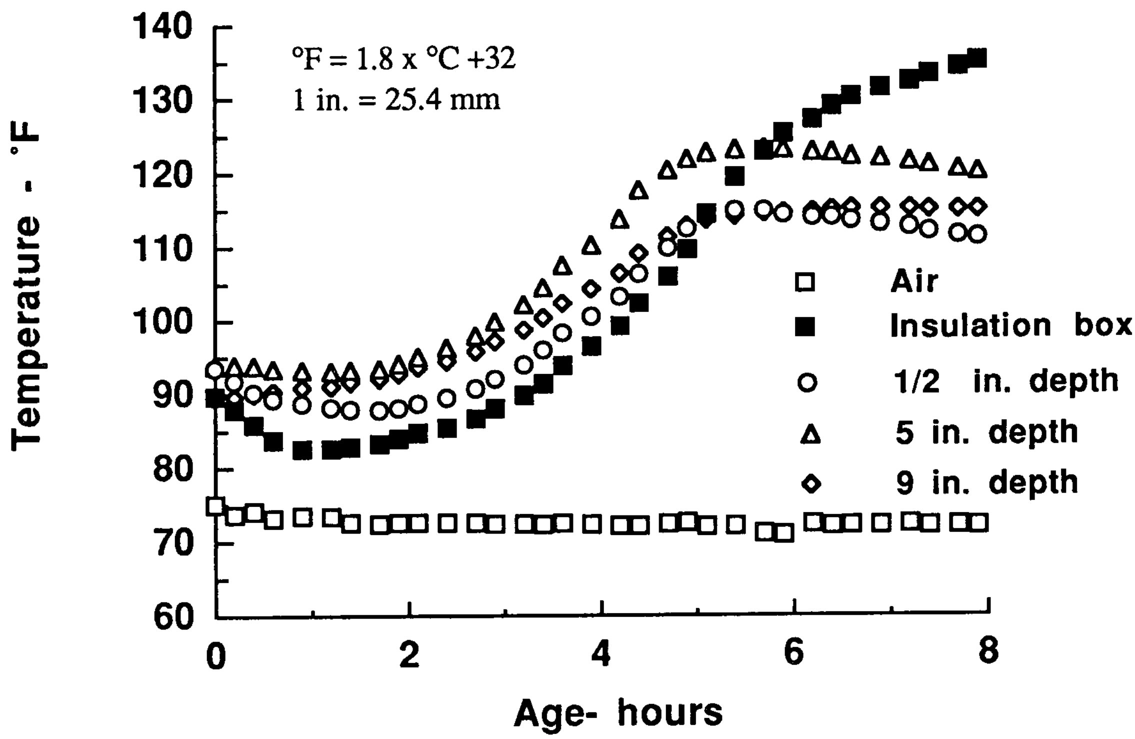

Figure 4.2. Temperature profiles for Fast-Track mix used at Site 5.

Table 4.9. Strength prediction compared for various techniques.

Mix (Site)	Time (hours)	Percent Difference (Core vs. Predicted) of: Maturity	PV	TMC	ICB	Core (psi)
VES (5)	5.8	+6	+20	+34	+18	2,450
Fast-Track (5)	7.8	−4	+26	+96	+74	1,150
PC1 (6)	6.2	+6	+9	+12	+15	1,650
RSC1 (6)	2.4	0	+5	+6	−2	3,250
PC2 (6)	2.9	−40	0	+5	−5	1,000
RSC2 (6)	4.7	−18	+30	+16	+21	3,050
VES (6)	4.4	−26	0	+22	+7	2,050
ODOT (6)	4.5	−31	+36	+10	+10	4,200
HES (6)	6.7	−16	−13	+2	0	3,000
Average		−16	+12	+22	+15	

Note: 1 psi = 0.0069 MPa

While not directly measured at early ages, development of MOR for the early opening mixes was determined by PV and maturity techniques using relationships developed on specimens cast from trial batches in the laboratory prior to field testing. Mix RSC1 gained strength most rapidly, exceeding a value of 300 psi (2 MPa) within 2 hours of placement. The companion PC1 mixture reached the same value after 4 hours. For the mixes designed for 4- to 6-hour opening (PC2 and RSC2), both exceeded 300 psi (2 MPa) within 5 hours. Both VES and ODOT mixes reached 300 psi (2 MPa) within 4 hours. This is a more rapid gain of flexural strength than seen for the companion mixes (PC2 and RSC2) prepared with more expensive proprietary rapid-setting products. HES, a nominal 12- to 24-hour opening mix, gained MOR very rapidly, reaching more than 400 psi (2.8 MPa) in only 6 hours. The Fast-Track mix gained strength more slowly than HES, reaching 300 psi (2 MPa) in about 10 hours at both sites.

Long-Term Strength Gain and Elastic Moduli

Compressive strength was measured on standard laboratory-cured cylinders at ages up to 90 days. Mixtures prepared with Pyrament cement (PC1 and PC2) rapidly gained strength during the period, indicating that significant hydration potential remains in the cement after its initial early rapid strength gain. Ultimate strengths for PC1 and PC2 mixes exceeded 10,000 psi (70 MPa). The concretes produced with Rapid-Set cement, however, appeared to have little potential for increased strength gain after the initial rapid strength rise during the first few hours after placement. Final strengths fell in the range of 5,000 to 6,000 psi (34 to 41 MPa) for these mixes. Other mixes showed a continuing increase in strength during the time period. Highest strengths were obtained for the ODOT mix, which reached 9,700 psi (66.9 MPa) at 90 days. The remaining three mixes (VES, HES, and Fast-Track) were similar in their strength profile over time and reached final 90-day strengths between 6,000 and

7,000 psi (40 to 50 MPa). Tensile strength showed similar behavior during the period. The highest split tensile strengths, nearly 1,000 psi (7 MPa), were exhibited by the PC1 mix. Tensile strength development for the Rapid-Set mixes was essentially flat across the time period. The remaining mixes showed modest gains in split tensile strengths during the period monitored, reaching 600–700 psi (4–5 MPa) at 90 days.

Elastic moduli were measured at 7 and 28 days. For a given strength level, moduli were higher at Site 5 than at Site 6, reflecting the differences in aggregate at the two sites. At 7 days, elastic moduli ranged from a low of 2.10×10^6 psi (14.5 GPa) for Fast-Track at Site 6 to a high of 4.40×10^6 (30.3 GPa) for VES at Site 5. At 28 days, elastic moduli ranged from a low of 3.10×10^6 psi (21.4 GPa) for HES at Site 6 to a high of 4.90×10^6 psi (30.7 GPa) for VES at Site 5. There was a general association between compressive strengths and elastic moduli for most of the mixes at both sites, with the mixes exhibiting highest strengths also showing higher moduli values.

Durability Testing

Beams for freeze-thaw testing were cast in the field, transported back to the laboratory, and maintained under saturated limewater for 14 days. Results using the modified AASHTO T 161 B procedure are shown in Table 4.10. At Site 5, both VES and Fast-Track mixes were highly durable. The GADOT mix, however, showed deterioration with the passage of time. The low air content of 3.4% and the use of calcium chloride as an accelerator may explain this poor behavior. At Site 6, with the exception of HES and RSC2 mixes, the tested concretes performed rather poorly. Durability factors were very low, and expansions for many of the concretes exceeded 0.1% at 300 cycles. The lack of air entrainment in mixes PC1 and PC2 would appear to explain their poor behavior. The mixes that performed well (HES and RSC2) had more than adequate air content and very low spacing factors. Mix RSC1 had a low air content and high spacing factor, which would help explain its poor performance. The ODOT mix, while having an acceptable air-void system, had a somewhat low air content of 4.3% and contained calcium chloride—which, in common with the GADOT mix used at Site 5, appears to degrade freeze-thaw performance.

The two mixes whose performance cannot be explained by the air void systems are Fast-Track and VES. Both performed extremely well at Site 5. At Site 6, however, while air contents for these mixes were higher than at Site 5, they did not pass the freeze-thaw test. While linear traverse analyses were being performed for air content, varying degrees of microcracking were observed in most of the companion specimens (with the exception of RSC1, RSC2 and HES). This observation may also help explain the poor freeze-thaw resistance of some of these mixes because the freezing of water within the thermally induced microcracks may exert additional stresses on the specimens during the freeze-thaw cycling.

Performance of the core specimens was better than for the corresponding beams cast from the concretes and tested using the modified AASHTO T 161B procedure. As in the beam testing, mixes RSC2 and HES performed the best, although mass loss was greater for the core specimens. Although the remainder of the cores did not perform as well as these, accrued damage during the test was nowhere near as great as experienced in the beam specimens. As noted, the increased severity of the modified procedure B may have contributed to the differences observed between beam and core testing.

Table 4.10. Resistance to freezing and thawing of repair concretes after 300 cycles of testing (modified AASHTO T 161B).

Mix (Site)	Durability Factor	Expansion (%)	Mass Change (%)
VES (5)	100	0.00	+0.82
Fast-Track (5)	100	0.01	−3.27
GADOT (5)	75	0.12	−1.78
PC1 (6)	15	0.19	+0.82
RSC1 (6)	64	0.26	−3.27
PC2 (6)	14	0.15	−1.78
RSC2 (6)	101	−0.02	−0.04
VES (6)	15	0.13	−1.77
ODOT (6)	13	0.09	+0.86
HES (6)	80	0.07	−0.65
Fast-Track (6)	25	0.09	+0.46

Follow-Up Surveys of Full-Depth Pavement Repairs

Follow-up surveys were carried out in the fall of 1992, approximately 2 months after the patches had been placed. At each site, the overlay surface was visually inspected for signs of cracking and cored to determine compressive, split tensile strength and elastic modulus. Falling-weight deflectometer (FWD) tests were also conducted to measure deflection transfer efficiency and to back-calculate the dynamic modulus of elasticity and modulus of subgrade reaction.

For the Georgia sections, compressive strengths ranged from 4,540 to 5,790 psi (31.3 to 39.9 MPa). Joint deflection transfer efficiency averaged approximately 75%. No visible cracking or other distresses were noted.

For the Ohio sections, compressive strengths ranged from 5,600 to 8,040 psi (38.6 to 855.4 MPa). Joint deflection transfer efficiency averaged approximately 75%. A significant number of longitudinal cracks were noted during the visual survey. All of the cracks occurred at mid-slab, apparently within a few weeks of opening to traffic. Longitudinal cracking was not anticipated because load-induced stresses are considerably lower than those expected to cause transverse cracking. A general trend was noted between higher cement contents, peak temperatures, and longitudinal cracking. Higher percentages of longitudinal cracking were noted for mixes with greater cement contents. Cracking could be attributed to transverse restraint provided by dowel bars. With a higher peak heat of hydration and potentially larger contraction upon cooling, the restraint stresses may have exceeded concrete tensile strengths.

The large amount of baseline information developed for these test sections will make them ideal candidates for continued observation and testing. It is expected that future monitoring of the sections by state forces or as part of FHWA research programs will develop long-term data that will reflect material durability and the ability of the patches to carry traffic over long periods of time. State forces or other agencies that will assume these functions for SHRP should make annual distress surveys of the patch test sections. Records of traffic data would also help interpret material performance.

Conclusions

Results of this phase of the investigation indicate that full-depth pavement repairs can be rapidly carried out using a variety of concrete mixtures. Opening of repaired areas to traffic is possible in as few as 2–4 hours after placement of the repair if special, rapid-strength-gain cements are used. Opening can be accomplished in 4–6 hours using mixes containing large amounts of Type III cement plus various chemical accelerators and high-range water reducers. More conventional mixes, such as Fast-Track designs, may also be used; however, these may take up to 12 hours to develop strengths suitable for opening to traffic. All of the mixes designed for early opening exhibit higher rates of heat generation and more rapid losses of slump than conventional concretes. These materials must be produced and placed expediently, and close cooperation between concrete suppliers and contractors is required for successful application. The following conclusions were based on the results of testing carried out in the field and laboratory. As was the case for the bridge deck overlay placements, these conclusions are strictly valid only for the particular materials and mixes used in this research program. Extrapolation of these conclusions to other materials or circumstances should be done with caution only after consideration of the particular conditions that may be encountered.

- Concretes used for early opening applications having a high rate of early strength gain develop considerable heat during the first few hours after placement. Temperature rise is greatest for mixes using large amounts (more than 800 lb/yd^3 [470 kg/m^3]) of Type III cements and chemical accelerators. Temperatures as high as 165°F (74°C) at mid-slab may be expected for these mixes. Proprietary rapid-strength gain cements, such as Pyrament and Rapid-Set cements, also exhibit rapid rate of strength gain but release less heat, resulting in lower peak temperatures within the repair slabs.
- A variety of methods can be used to predict in-place strength gain of concretes designed for early opening applications. Techniques such as maturity, pulse velocity, temperature-matched curing, and use of well-insulated test cylinders were all evaluated. All techniques could estimate strength to within an average of ±20% of strengths determined on cores obtained from the sections. Although maturity techniques appeared to offer the most conservative predictions, they require considerable precalibration work in the laboratory. The same holds true for pulse velocity methods, though calibration for these is less arduous. Temperature-matched curing and use of insulated cylinders yield similar results; the latter is simpler and less expensive, although an on-site portable compression machine is required for both techniques.

- Most of the early opening mixes continued to gain strength after placement up to at least 90 days (the longest test period included in this study). Strengths as high as 10,000 psi (70 MPa) were reached with mixes prepared with Pyrament cement. Although exhibiting more rapid rates of gain of early strength (2–4 hours), Rapid-Set cement showed only a moderate increase in strength at 1 day.
- Questions still remain regarding durability of concretes designed for early opening applications. Proper air content and adequate air-void systems are necessary but not sufficient conditions for obtaining the desired freeze-thaw durability. There are indications that microcracking in the concretes, which may have been thermally induced, may account for some of the poor performance in freeze-thaw testing. The use of calcium chloride should be avoided because it contributes to reduced freeze-thaw resistance. Although the freeze-thaw test used for evaluation of durability may be very severe, additional work with regard to durability of early opening mixes should be performed before their use in northern areas with harsh winter conditions can be recommended unequivocally.

Criteria for Early Opening of Full-Depth Repairs

Objective

Data collected from the field trials were analyzed to determine the amount of fatigue damage attributable to early opening and the minimum flexural strength required for opening concrete repairs to traffic. The effects of concrete compressive strength and dowel bearing stresses were also examined to study how early opening affected faulting of the repairs.

Information on project descriptions, constituent materials, mix proportions, testing methods, and results were previously summarized and are presented in detail in Appendix A. Traffic data and presentation of analytical procedures needed for development of early opening criteria are given in Appendix B.

Fatigue Analysis

A fatigue analysis was performed to evaluate the effects of early age fatigue. Stresses used in the analysis were computed for the combination of load and curling (thermal) stresses. Load-induced flexural stresses were computed using regression equations developed by Salsilli (1991). Stresses were computed at various distances from the longitudinal edge. Traffic data collected during field installations were used to compute equivalent combined pass-coverage ratios (p/c) to account for the effects of loads placed near the point at which fatigue damage is to be calculated. Curling stresses were also computed. Predicted temperature gradients used in the analysis were generated from the Climatic-Materials-Structural (CMS) model developed for FHWA (Dempsey, Herlacher, and Patel 1983). The model was used to estimate hourly temperature and moisture gradients through the pavement, given material properties and climatic data obtained from the National Climatic Data Center.

Flexural strength (modulus of rupture) development was estimated from early age cylinder tests. Relationships between modulus of rupture and compressive strength were established from strength test data on mixes made in the laboratory using job materials prior to field testing. Cylinder strengths correlated well with core and nondestructive testing strengths.

The allowable number of 18-kip (80-kN) equivalent single-axle load (ESAL) repetitions necessary to cause cracking of 25% of a slab was determined by input of the combined thermal and load stresses into a fatigue model. Fatigue damage was determined using Miner's theory by summing the ratio of the number of applied loadings to the number of allowable loadings. With the use of traffic survey, strength development, and computed (load and thermal) stress data, fatigue damage was calculated on an hourly basis for the first 3 days and then on a daily basis for the next 11 days. Early age fatigue damage was minimal for both the Georgia and Ohio patches. Early age fatigue averaged less than 2% for Georgia and less than 1% for Ohio repairs.

The allowable early age fatigue was determined by considering both temperature (curling) and load-induced stresses. Annual frequency histograms of predicted slab temperature differentials were used to predict thermal curling stresses. The number of ESAL applications for each temperature gradient range was determined by multiplying total annual ESALs for the design lane by the frequency of occurrence within the temperature gradient range. The allowable early age fatigue was calculated by subtracting the product of design service life and fatigue damage per year from 100%. For a 15-year design life, the allowable early opening fatigue damage was significantly greater than the actual early opening fatigue consumed, suggesting that further fatigue damage during early opening could have been tolerated without reduction in expected service life. This assumes that the fatigue damage function and concept developed for mature concrete can be applied to early age fatigue.

Bearing Stress Analysis

For doweled concrete repairs subjected to early opening, the maximum dowel bearing stress can govern the opening time—especially for shorter repairs, where allowable early age fatigue is relatively high. Maximum bearing stresses were computed using the modified Friberg analysis (Heinrichs et al. 1989). Bearing stresses were compared to compressive strengths of concrete in the patches. The compressive strength required to prevent excessive dowel bearing stress is approximately 1,500–2,000 psi (10.3–13.8 MPa), depending on the dowel diameter.

Opening Strengths

The opening strength for concrete repairs depends on traffic density, patch design life, temperature gradients, and repair length. For a given temperature gradient, design life, and traffic distribution, as the repair size increases, load-induced stresses slightly increase and curling stresses significantly increase. For very short repairs—approximately 6 ft (1.8 m)—dowel-bearing stresses are more critical than the modulus of rupture at early ages. For repairs up to 12 ft (3.7 m) in length, the opening criteria used by many states—third-point modulus of rupture of 300 psi (2.1 MPa) or a compressive strength of 2,000 psi (13.8 MPa)—are reasonable.

Conclusions

The fatigue and bearing stress analyses indicated that patches can be designed for rapid openings to reduce traffic delays. When special mixes are used, the overlays achieve sufficient strength to be opened to traffic in from 2–24 hours. The following conclusions were based on the data obtained in the study.

- Patches can be opened to traffic, depending on the concrete mix design, 2–24 hours after placement. High cement factors are required, ranging from 650 to 900 lb/yd^3 (385 to 530 kg/m^3). The use of proprietary mixes was necessary to achieve the earlier opening times. Type III cement and chemical accelerators are required for opening at 4–6 hours.

- Opening strength depends on the repair length, concrete temperature gradient, traffic distribution, and required service life. Generally, more rapid-setting mixes can be opened to traffic at a lower strength than mixes having more moderate strength gains, and longer repairs require higher opening strengths.

- For very short repairs—approximately 6 ft (1.8 m)—dowel bearing stresses are more critical than the modulus of rupture at early ages. For repairs up to 12 ft (3.7 m) in length, the opening criteria used by many states—third-point modulus of rupture of 300 psi (2.1 MPa) or a compressive strength of 2,000 psi (13.8 MPa)—are reasonable.

- Field data indicate that many of the early opening mixes generate very high temperatures during the first few hours after casting. Follow-up surveys in Ohio indicated a significant number of longitudinally cracked patches. Cracks may have developed from a combination of temperature gradient (curling restraint) and contraction (dowel restraint) stresses.

- The early opening fatigue analysis used mature concrete fatigue functions and methodology. Extrapolation of mature concrete fatigue analysis methods to early ages has not been verified. Long-term performance surveys on these early opening experimental pavement repair sections are recommended to verify that fatigue assumptions used in the analysis can be applied in establishing early opening criteria.

5

Highway Concrete Expert System

Introduction

Advances in computer hardware and software development tools make it feasible to develop expert systems that are effective decision-making tools for highway staff. Activities such as diagnosing distresses, designing concrete structures, and making decisions related to the selection of repair and rehabilitation procedures and materials are conducive to expert systems application. Specific examples of applications and architectures were presented in a synthesis report developed as part of this project and published as a SHRP report (Kaetzel and Clifton 1991). The report describes the benefits of expert systems to the highway field.

Computerized systems can be developed that integrate different forms of knowledge normally used by highway staff. The knowledge may be represented in the form of pictures, drawings, databases, guides, and specifications. With the addition of reasoning from high-level experts, (represented as rules on how to use the knowledge), a coherent system has been developed for use by highway inspectors, engineers, concrete specifiers, and decision makers. This system for highway concrete, called HWYCON, was developed in Task 3 of SHRP C-206, Optimization of Highway Concrete Technology. The system is intended to help improve concrete quality and durability and help achieve the desired specifications for concrete pavements and structures. Specific examples of how expert systems can be used for highway concrete are shown in Table 5.1. HWYCON includes knowledge in all three areas identified in Table 5.1 and is designed to address materials-related activities.

This chapter presents information on the design and development of HWYCON as well as recommendations on modifications and future enhancements. The chapter is intended to present an overview of HWYCON system. A detailed report covering the development and implementation of the system is presented in the documentation provided with the computerized system.

Table 5.1. Examples of the use of expert systems for highway concrete.

Highway Activity	Use of Knowledge
Diagnosing distresses	Distress identification and cause
	Visual display of distress characteristics (e.g., pictures, drawings) for better interpretation
	How-to descriptions of laboratory and field tests to confirm distress cause(s)
Selecting materials	Recommendations on the design of concrete for alkali-aggregate, freeze-thaw, corrosion, and sulfate durability
	Examples of concrete mixture proportioning to achieve early opening times and desired compressive strength
	Recommendations on the use of materials with known problems and limitations
Developing repair and rehabilitation plans	Recommendations on the selection of materials that are suitable for specific repair approaches (e.g., full-depth repair, bonded overlays)

Development Approach and Architecture of the Highway Concrete (HWYCON) Expert System

The HWYCON expert system contains three subsystems and is designed to assist the staff in state highway departments in diagnosing distresses, designing and specifying concrete, and making repairs and rehabilitating concrete pavements and structures (e.g., bridge decks and parapet walls). More specifically, the diagnostics component of HWYCON, CONcrete PAVement Diagnostics (CONPAV-D), is designed to help identify materials-related distresses in concrete pavements, to give conclusions about the cause of the distress(es), and to make recommendations on tests and procedures to perform to confirm potentially conflicting or unknown causes. Another component of the diagnostics subsystem is CONcrete STRUCture Diagnostics (CONSTRUC-D). This component of the subsystem identifies distresses, drawing conclusions and making recommendations on their cause for bridge decks and substructures. The CONcrete MATerials (CONMAT) selection subsystem of HWYCON is designed to assist concrete specifiers in designing concrete for durability areas and recommends which materials related to permeable bases, recycling concrete, and Fast-Track concrete should be selected. The third subsystem of HWYCON, CONcrete PAVement Repair and Rehabilitation (CONPAV-R), is designed to recommend procedures and materials on full-depth repair, partial-depth repair, bonded and unbonded overlays, and diamond grinding and milling. Figure 5.1 illustrates the structure of the HWYCON expert system. Although there is knowledge in HWYCON that address effects of traffic and loads on highway concrete structures, the system primarily addresses materials related distresses, selection of materials for concrete, and repair of highway pavements and structures.

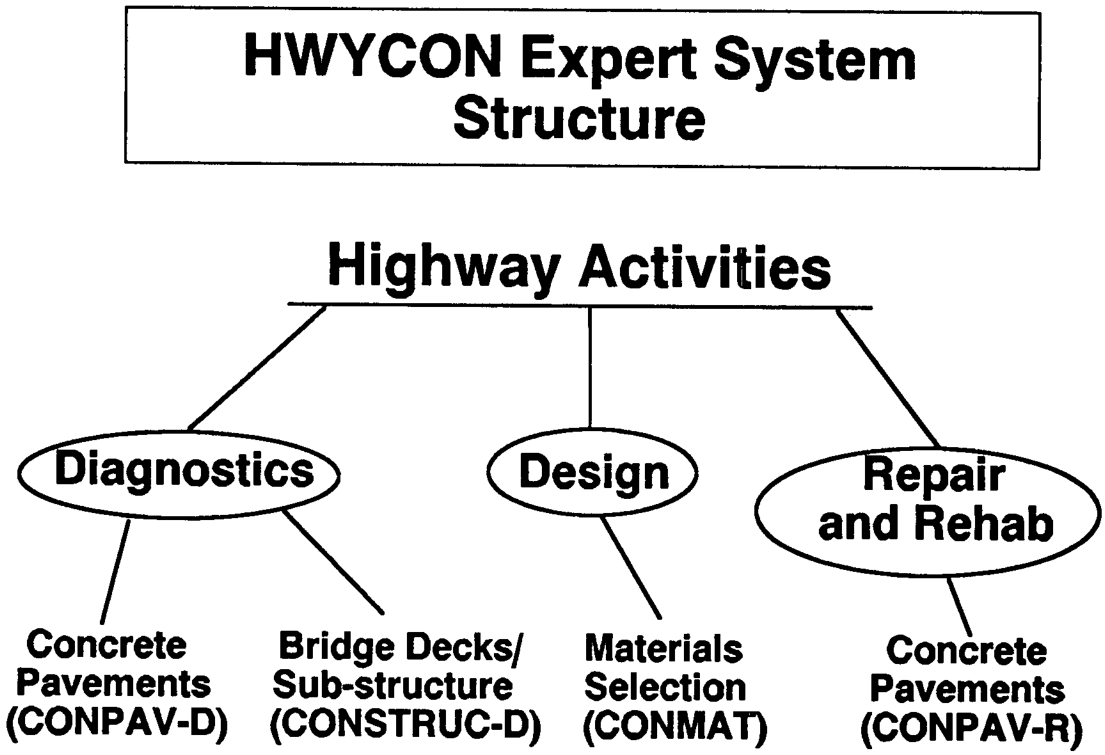

Figure 5.1. Illustration of the HWYCON expert system structure.

Design of an Expert System

Several important factors can contribute to the success of deploying an expert system. The factors considered in the development of HWYCON include the following.

- Initially limiting the scope of the knowledge domain, then allowing it to grow.

- Obtaining feedback from users through prototype development and distribution.

- Selecting a development tool that provides a platform for further enhancements and the addition of new knowledge.

- Selecting a development tool that allows flexibility for incorporating different knowledge forms and provides a high level of programmer productivity.

When designing expert systems, developers will obtain the best available human knowledge sources to design the knowledge base, to critique and review the computerized system, and to referee the review comments received from the prototype systems. This is perhaps the only area in expert system development that has not changed dramatically. There will undoubtedly be disagreement on what knowledge to include, on the bases of credibility, preferences, and other factors. Accepted guidelines, practices, and test methods may sometimes offer solutions to a disagreement on whether to include a piece of knowledge or how to use it, but they may be outdated.

To clarify the workings of the HWYCON system, the following example is given. Although not essential for describing the system, the example will be helpful to develop a perspective on how the system operates. Perhaps the most significant change in expert systems architecture is the representation of the knowledge within the computer and its interrelationships. Most knowledge is represented in the form of rules that tell the inference engine how to use the knowledge. Rules have the basic form of "IF condition THEN action." For example, "If a crack develops before the concrete has hardened, and if the crack pattern is random, then the crack may be a plastic shrinkage crack."

Development tools today make use of the rule form of knowledge representation in combination with other techniques, such as those found in the network architecture. This type of hybrid system is often called an object-oriented system architecture. Unlike their predecessors, object-oriented systems use multiple inference (logic) procedures (as, e.g., backward chaining, whereby the system attempts to reach a goal when given information that leads to that goal—as is the case in recommending the amount of concrete cover to use when designing concrete for a corrosive environment). The forward chaining inference is also used to activate other procedures (computer modules, algorithms, or functions). The term forward chaining implies that knowledge is needed to satisfy a goal of the expert system. The forward chaining knowledge may include calculations, external programs, and the display of information. Other powerful features associated with object-oriented systems allow the knowledge engineer to draw relationships between knowledge components and define how the system is to be used (e.g., search order, type). An example taken from the Highway

Concrete (HWYCON) expert system is illustrated in Figure 5.2. The information contained in the ellipses shows the path that the system would follow to reach the goal corrosion of reinforcement. In this figure, the object attribute crack pattern and direction is defined, and its relationships are established relative to the conclusion or goal, corrosion of reinforcement. Figure 5.3 illustrates the object attribute associativity for the object crack pattern and direction. The question/answer display shown in the figure provides the initial interface to the user when the object crack pattern and direction is needed by the expert systems inference engine. This may be performed in a rule-based backward chaining inference procedure or may be activated in a forward chaining inference procedure, such as selecting a pushbutton from a previous display. Objects are connected in one of two ways: through a rule(s) contained in the knowledge base or through associating (connecting) a display with another display or object.

Development Process

One of the initial steps in the development of HWYCON involved establishing criteria for determining the scope of the knowledge domain, the sources of knowledge, performance of the computerized system, user review and feedback, and system implementation. Table 5.2 shows the criteria established for the development and implementation of the expert system.

Table 5.2. Criteria for the development and implementation of HWYCON.

Criterion	Goal
Target audience (staff)	*Diagnostics:* inspectors and engineers *Materials Selection:* concrete specifiers *Repair/Rehabilitation:* engineers, decision makers
Target audience (level of existing knowledge)	Aimed at mid-level and below
Modifications and enhancements to knowledge base	Select a development tool to allow knowledge to be segmented and modified easily
Interpretation of knowledge	Allow the use of different forms of knowledge (rules, pictures, drawings, databases, explanatory information)
Implementation computer platform	Design for desktop and portable personal computers
Feedback and review	Develop prototype systems to allow quick startup and easy user interface

To accomplish this task, numerous groups and individuals were consulted. Because one of the main objectives of the system was to capture and transfer the knowledge being produced in other SHRP projects, guidance and assistance from the SHRP C-206 Expert Task Group and other SHRP project leaders was sought in defining the scope and source of knowledge as well as for reviewing and providing feedback on the computerized system. In addition to high-level experts at NIST and C-206 team members, other experts in the field of concrete were chosen to assist in various activities. State DOTs were asked to provide information

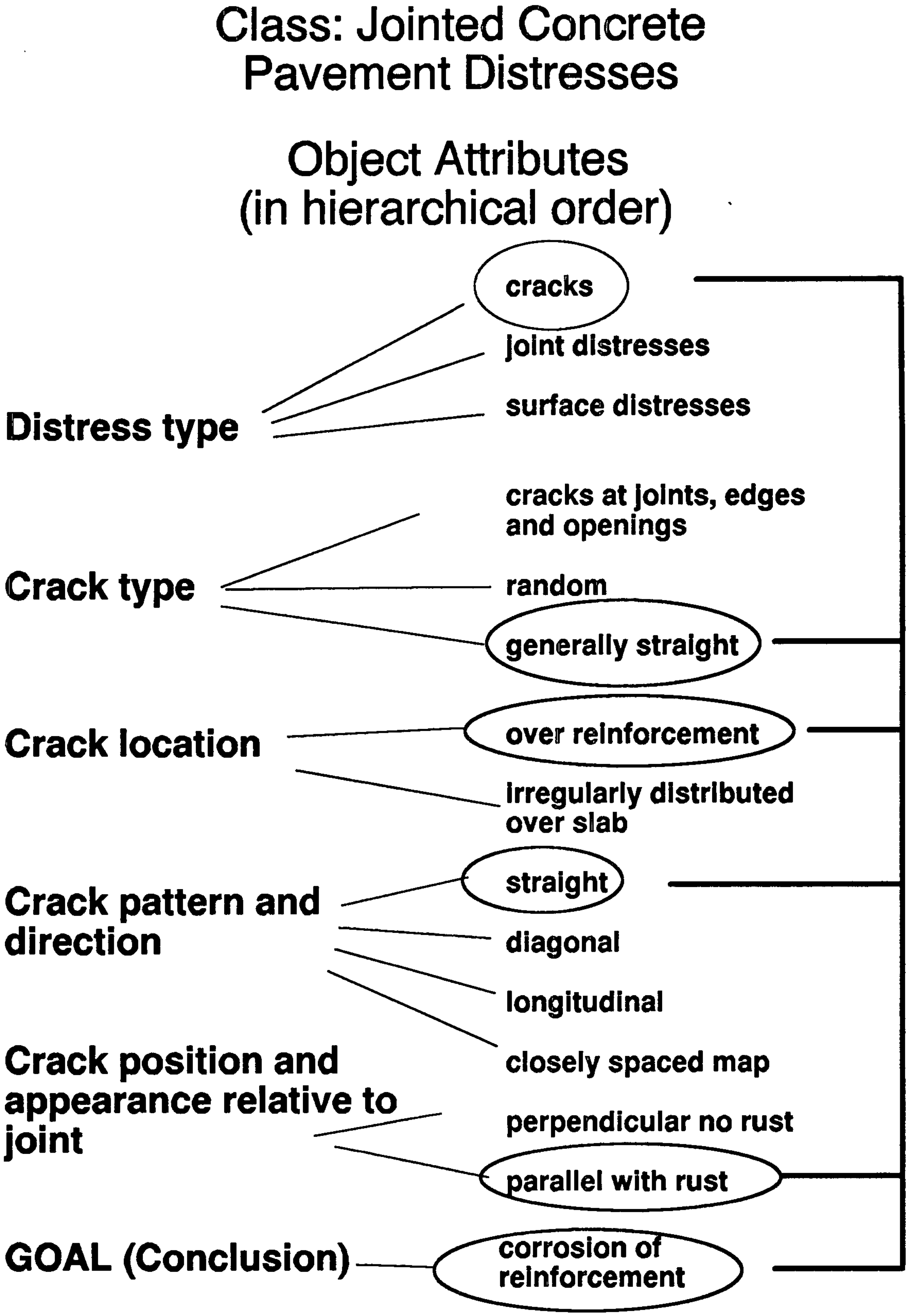

Figure 5.2. Illustration of an object-oriented knowledge structure.

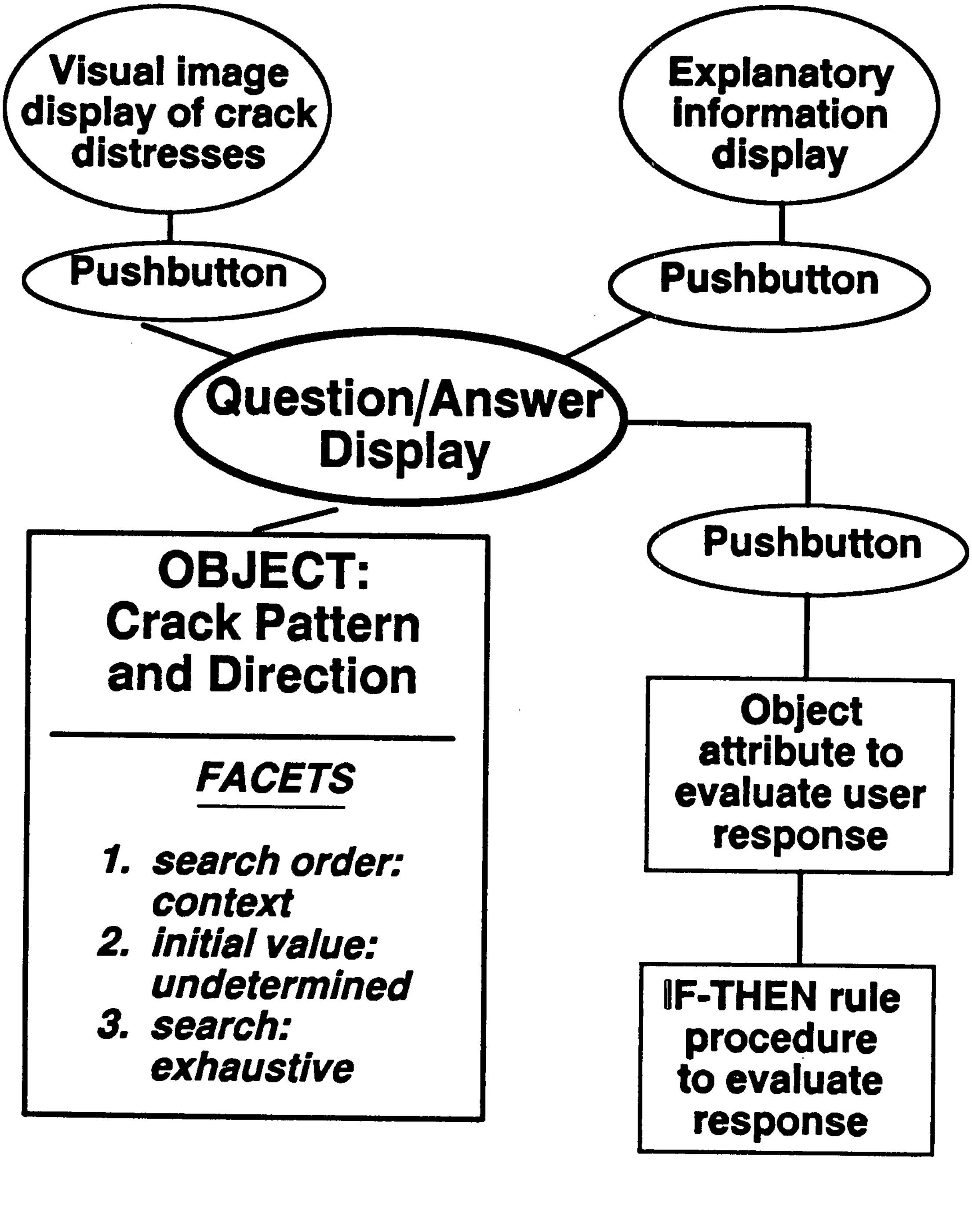

Figure 5.3. Example of a HWYCON object connectivity.

related to their areas of expertise. Literature searches, guides, specifications, and technical notes were obtained by the development team for evaluation, review, and incorporation into the knowledge base. Federal Highway Administration (FHWA), Transportation Research Board (TRB), and American Association of State Highway Transportation Officials (AASHTO) staff were interviewed. In addition, staff members from associations, institutes, academia, and other groups working in the knowledge domain were interviewed.

Table 5.3 identifies the principal groups involved in the development and implementation of HWYCON as well as their areas of contribution.

A library containing the most current and state-of-the-art developments in cement and concrete for highway activities was established and is maintained at NIST.

To produce an operational expert system, many steps are involved. These steps can be divided into four phases: conceptual design, knowledge acquisition and prototype development, review and modification, and product delivery. The specific steps taken for each of the categories were as follows.

Conceptual Design Phase

- Literature review
- Development of state-of-the-art report
- Evaluation of computer hardware and software development tools
- Interviews with domain experts, state DOTs, highway industry

Knowledge Acquisition and Prototype Development Phase

- Development of methods for integrating knowledge
- Review and evaluation of guides, specs, and manuals
- Interviews with domain experts and users
- Development of knowledge trees
- Review of knowledge trees by expert team
- Acquisition and enhancement photographs/drawings
- Development of computerized prototype

Prototype Review and Revision Phase

- Review of prototype system by development team
- Modification of prototype
- Distribution of prototype to users for review and demonstration at transportation shows and conferences
- Evaluation of review comments and revision of prototype as needed

Development of Operational System Phase

- Development of installation and quick reference manual
- Development of user and developer reference document
- Distribution of operational system to SHRP

Table 5.3. Principal groups involved in the development and implementation of HWYCON.

Principal Group	Area of Involvement				Area of Knowledge			
					Diagnostics of:			
	System Design	Knowledge Acquisition	Computer-ization	Review/ Comment	Pavement	Structure	Materials Selection	Pavement Repair/ Rehab
Academia								
Iowa State University				◆	◆	◆	◆	
North Carolina State University	◆	◆		◆			◆	
Purdue University	◆	◆		◆	◆	◆	◆	◆
University of New Hampshire	◆	◆		◆	◆		◆	◆
American Association of State Highway and Transportation Officials	◆	◆		◆	◆	◆	◆	◆
American Concrete Pavement Association	◆	◆		◆				◆
American Concrete Institute		◆					◆	
Federal Highway Administration	◆	◆		◆	◆		◆	◆
SHRP C-206 staff								
CTL contractor	◆	◆		◆	◆	◆	◆	◆
NIST team	◆	◆	◆	◆	◆	◆	◆	◆
ERES team	◆	◆		◆	◆			◆
SHRP staff and ETG	◆	◆		◆	◆	◆	◆	◆
State DOTs[1]	◆	◆		◆	◆	◆	◆	◆

[1]States represented are Florida, Illinois, Iowa, Kansas, Maryland, North Dakota, Virginia, Washington, and Wyoming.

Because many expert systems never reach an operational status due to the absence of user input and because a stigma exists concerning expert systems, it was felt that user feedback to the development process was essential. User feedback became an important part of the development of the system and allowed review comments to be evaluated during the course of the project and was instrumental in determining what changes were necessary in the design and operation of future revisions. Comments were received from approximately 60% of those people reviewing the prototype systems. This is an unusually high number of responses, particularly for expert systems.

Each subsystem was initially developed and distributed independently, in the following sequence:

- Concrete Pavement Diagnostics: CONPAV-R, CONSTRUC-D
- Selection of Materials: CONMAT
- Concrete Pavement Repair and Rehabilitation: CONPAV-R

The process of acquiring the knowledge and developing the prototype involved determining the scope of the knowledge to be contained in the subsystem, evaluating the literature, interviewing experts, developing the knowledge tree, and developing the computerized version using the expert system development shell program. The knowledge tree served as the vehicle for communication between the experts who interpreted and organized the knowledge in a hierarchical structure and the knowledge engineer who developed the computerized version of the knowledge. A computer text file was used to initially record the knowledge in a question-and-answer form along with a network diagram. Using this format, the knowledge engineer developed the knowledge tree. An example of a portion of the knowledge tree from CONPAV-D for crack distresses is shown in Figure 5.4. The knowledge tree was useful for two purposes: it provided the source document for converting the knowledge to questions and answers as well as explanatory displays and for developing the systems rules and procedures (computer-coded algorithms); and it was useful as a synopsis for experts and reviewers because it provided a road map of the knowledge contained in the system.

Review comments were evaluated by the development team, and changes were made in operational features and the knowledge base. Most operational changes were made as the result of reviews of the first prototype of the CONPAV-D subsystem. These included:

- Operational control of user sessions (e.g., pushbuttons)
- Multiple choice selections from question-and-answer screens
- Improvements in the quality and use of visual information

Changes to the knowledge base were also made to improve user understanding and to correct errors found involving incorrect paths, conclusions, and recommendations. Again, the knowledge trees were instrumental in identifying the portion of the knowledge base or operational components that required changing. Also, the use of the object-oriented system architecture allowed changes to be made more easily because user displays, rules, and object attributes could be identified quickly.

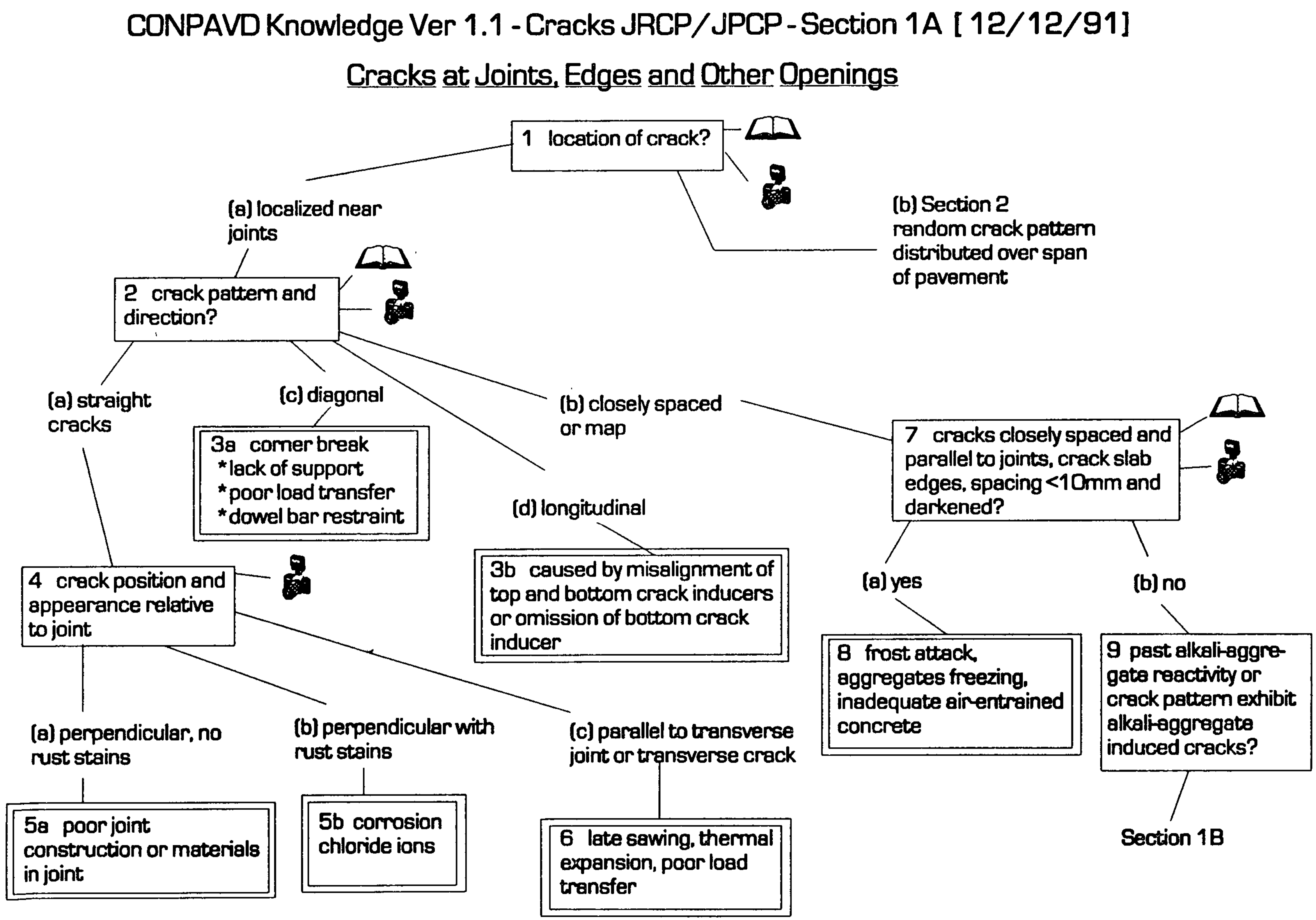

Figure 5.4. Sample of a portion of the knowledge tree from CONPAV-D cracking distresses

User Reference

A user reference was written during the final phase of the project. The document includes detailed descriptions of the design and development of HWYCON. It was written to provide users and developers with information on the architecture and use of the system and how to make modifications. The document is included with the computerized system that is available from SHRP. It includes the following topics in the order presented below.

- Background on expert systems
- Conceptual design of the system
- Knowledge sources and contents
- Computer resource requirements
- How to install and use the system
- Logic (inference) used
- Enhancing and modifying the system
- Bibliographic references
- Knowledge trees
- Field inspection checklists

Description of the HWYCON Subsystems

Concrete Pavement Diagnostics (CONPAV-D)

The first of three HWYCON subsystems to be examined is CONPAV-D. CONPAV-D is designed to allow highway inspectors and engineers to identify materials-related distresses and draw conclusions as to their causes. The system contains knowledge on jointed reinforced concrete pavements (JRCP), jointed plain concrete pavements (JPCP), and continuously reinforced concrete pavements (CRCP). Distress categories considered are cracking, joint-related, and surface distresses. Figure 5.5 shows the distress categories and distress types contained in CONPAV-D. The user of the system is asked to identify first the pavement type and then the distress category. Additional questions are then asked to solicit information to draw a conclusion. These questions may include the exposure conditions and pattern and location of the distress. Each session involves a different sequence of questions in which the expert system queries the user about observations and history of the highway pavement.

Each set of questions contained in the system is associated with a conclusion or goal. Some goals may be inconclusive; in these cases, the user is given recommendations on tests to resolve suggested probabilities involving a combination of causes or an unknown reason for the distress. Examples of some of the causes of distresses included in CONPAV-D are

- Corrosion of reinforcing steel
- Frost attack

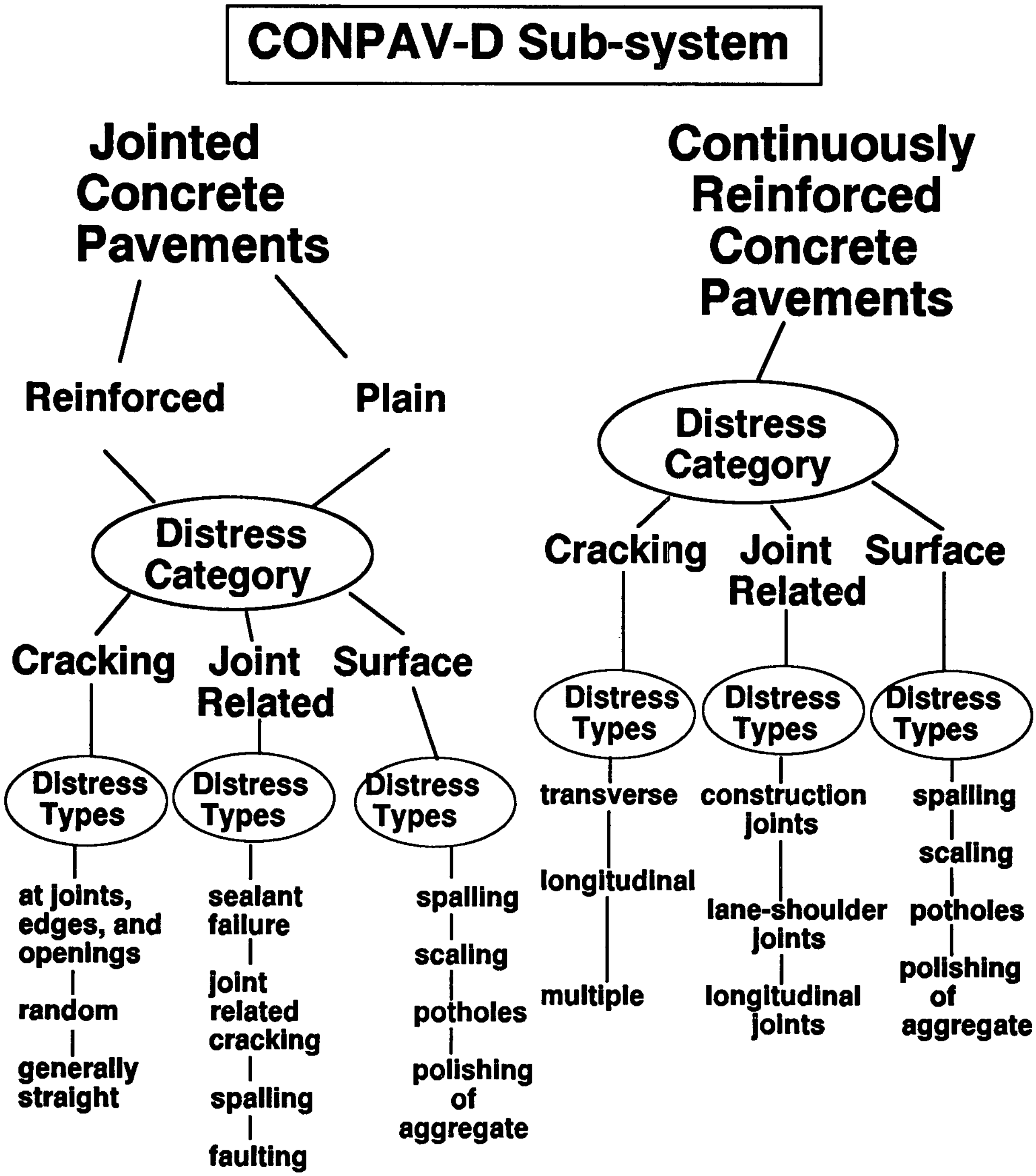

Figure 5.5. Diagram of the CONPAV-D subsystem component for concrete pavements.

- Alkali-aggregate reactivity
- Shrinkage cracking
- Sealant failure
- Spalling caused by weak concrete
- Faulting caused by pumping

This list does not include all of the distresses addressed in CONPAV-D but is presented to give the reader an idea of the range of distresses that are covered.

Knowledge about distresses is represented in four different forms: rules that simulate how a high-level expert would diagnose distresses, visual information contained in pictures and drawings that show examples of distresses, explanatory information about test procedures and why questions are being asked, and bibliographic information to help the user in further analysis or investigation.

Concrete Structures Diagnostics (CONSTRUC-D)

The first HWYCON subsystem also includes CONSTRUC-D, which is divided into two components. The first involves diagnosing distresses in bridge decks; the second involves substructure components to include parapet walls, piers, columns, and other horizontal and vertical surfaces. The knowledge contained in CONSTRUC-D for bridge decks includes cracking, spalling and popouts, scaling, and polishing of aggregate. CONSTRUC-D for structures includes all of the above plus disintegration. Figures 5.6 and 5.7 show the organization of CONSTRUC-D and the distress categories and types included in each system. CONSTRUC-D sessions are conducted like CONPAV-D sessions, in which the user responds to questions and conclusions are then given. CONSTRUC-D for bridge decks asks the user to identify the construction of the bridge—either concrete or concrete and steel—whether the reinforcing steel is epoxy coated, and the exposure conditions. The bridge deck component considers bridges that are exposed to freezing temperatures, deicing salts, and sea water. Most of the knowledge contained in CONSTRUC-D for bridge decks involves the diagnosis of cracks. In addition to cracking, spalling and popouts, scaling, and polishing of aggregate are included. Like CONPAV-D, CONSTRUC-D includes knowledge in several forms.

Selection of Concrete Materials (CONMAT)

The second HWYCON subsystem, CONMAT, was developed to assist concrete specifiers in designing durable concrete and in selecting materials for the construction and reconstruction of highway structures involving three new procedures. The durability areas include recommendations on corrosion of reinforcing steel, alkali-aggregate reactions, sulfate attack, and freezing and thawing durability. The new procedures covered include the recycling of concrete, permeable bases, and Fast-Track concrete. Figure 5.8 shows the knowledge topics covered by CONMAT. The recommendations given in CONMAT would be useful to highway staff who must specify concrete for the above-referenced durability environments

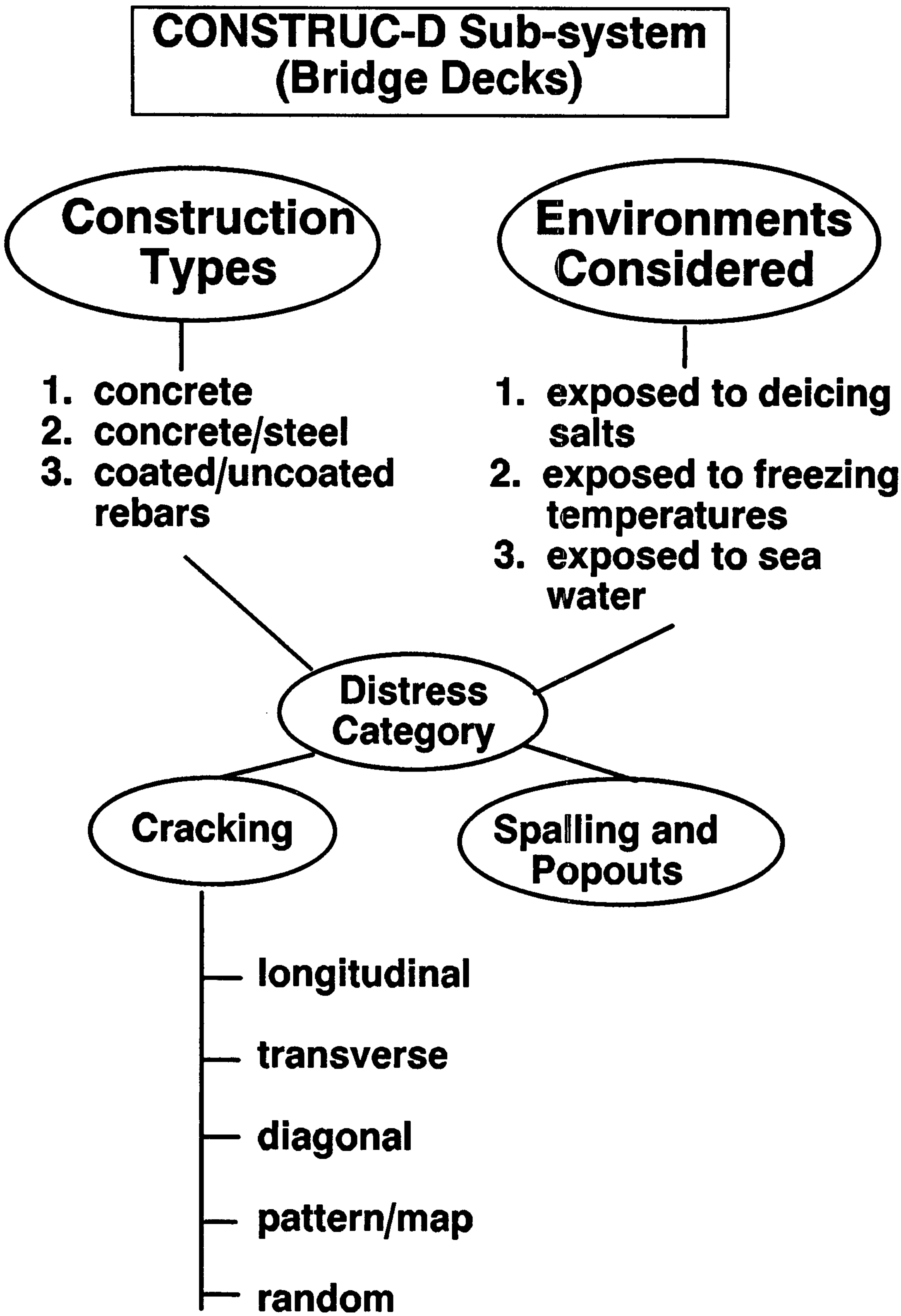

Figure 5.6. Diagram of the CONSTRUC-D component for bridge decks.

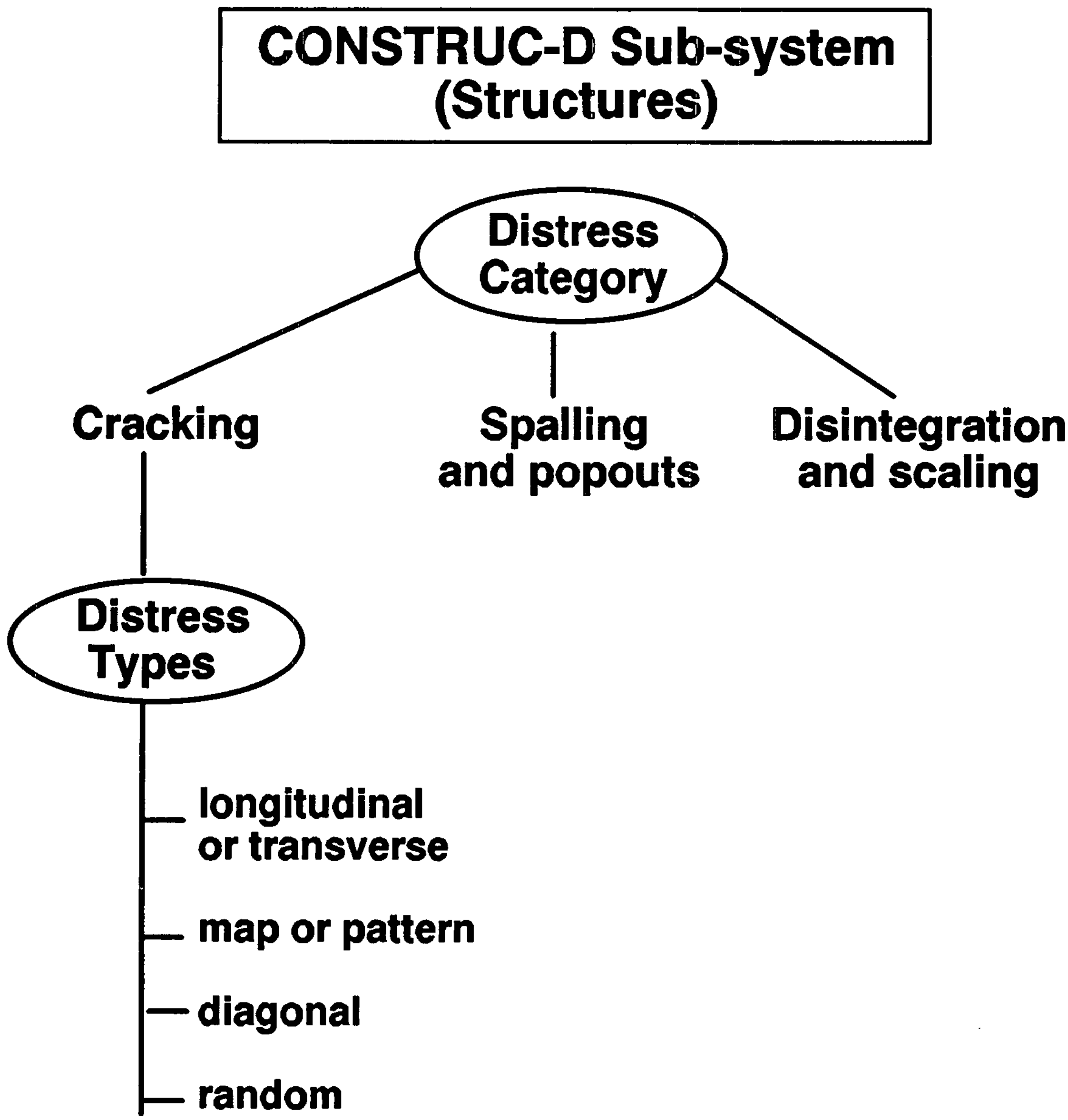

Figure 5.7. Diagram of the CONSTRUC-D component for structures.

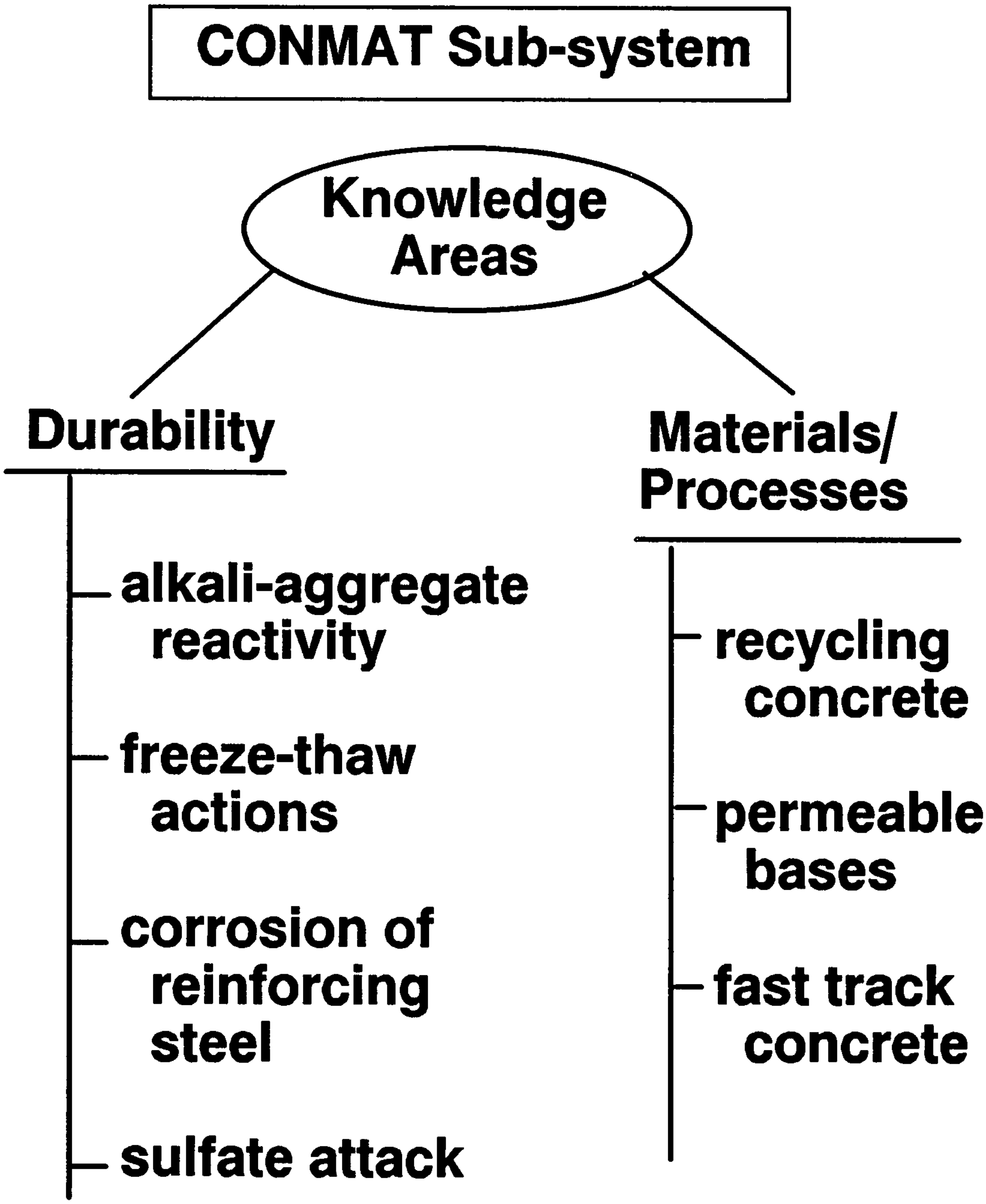

Figure 5.8. Diagram of the CONMAT subsystem showing its knowledge topics.

and procedures. The use of CONMAT may involve answering questions related to the performance of materials and their conformance to ASTM or AASHTO test methods. Much of the knowledge represented in CONMAT involving the durability areas was obtained from the American Concrete Institute's *2R-92 Guide to Durable Concrete*. SHRP-sponsored projects provided knowledge on testing of aggregates for reactivity and frost resistance and on concrete for Fast-Track construction. Additional knowledge includes recent activities in the development of procedures, tests, and published literature related to FastTrack, permeable bases, recycling of concrete, and concrete durability. Table 5.4 illustrates the areas of knowledge contained in CONMAT. Examples of recommendations given in CONMAT include w/c, depth of cover, selection of cements, selection of blended cements, selection of admixtures, mitigation of alkali-aggregate reactions, recommended chloride levels in soil and water, air content, and compressive strength. CONMAT also recommends concrete mixture design for Fast-Track pavements for different opening times and compressive strength. These include opening times of 4, 12, and 24 hours. The recycling concrete module makes recommendations on the use of aggregate and the use of the constituents of concrete if durability problems exist, depending on the type of jointed pavement selected. The permeable base module recommends stabilized and unstabilized permeable bases and includes information on separator layers and edge-drainage systems.

Table 5.4. CONMAT durability areas and the knowledge areas.

Durability Area	Knowledge Area
Alkali-aggregate reaction	Measures for determining aggregate reactivity potential, recommendations on use of reactive aggregate
Corrosion of reinforcing steel	Selection of materials (chloride content, cementitious materials, aggregate, mix water, reinforcing steel, admixtures); recommendations on the design of concrete (concrete cover, exposure to water, deicing chemicals, soil subbase materials, chlorides)
Freeze-thaw action	Prevention of scaling, severity of freezing and thawing (moderate, severe), conventional highway concrete, high-performance concrete, selection of aggregate, selection of admixtures and cementitious materials
Sulfate attack	Sulfates in soil and water for mild, moderate, severe, and very severe environment; selection of admixtures and cementitious materials

Concrete Pavement Repair and Rehabilitation (CONPAV-R)

The third HWYCON subsystem, CONPAV-R, is designed to recommend various repair and rehabilitation procedures. Highway staff involved in decision-making activities for highway pavement would benefit from this subsystem. The system expects that, before it is used, a repair or rehabilitation procedure was selected. Recommendations are given for partial-depth repair, full-depth repair, bonded concrete overlays, unbonded concrete overlays, and diamond grinding and milling. Information is contained in the system on optimum procedures and materials for each topic. Information on procedures identifies the required steps and how they are to be performed. Information on materials relates to the proper selection of materials for specific opening times. A section is included under each procedure that describes new developments. Figure 5.9 shows the structure of CONPAV-R and the procedures that are covered. CONPAV-R also includes drawings from accepted concrete paving documents that show examples of diagrams of pavement repair areas to help the user understand methods, dimensions, etc. The knowledge base also includes early opening guidelines for full-depth and partial-repair mixtures that recommend slab thickness, temperature, and mixture characteristics (w/c, cement type, and admixtures). When repairs are made or a pavement is rehabilitated, the causes of existing distresses should be identified and measures taken to avoid them. Therefore, CONPAV-R also allows the user to select CONMAT alkali-aggregate durability, freeze-thaw durability, sulfate durability, and the bonded and unbonded overlay topics. The Fast-Track concrete module can also be selected.

Implementation of HWYCON

The HWYCON expert system is available from SHRP as product number 2029. The following items are included in the operational package.

- Installation reference and overview document
- User and developer reference document
- Level 5 Run-Only system diskette
- HWYCON executable knowledge diskettes

A desktop or portable 386 or 486 computer system is required to operate the system. Computer software requirements include the Disk Operating System (DOS), version 3.0 and later; and Microsoft Windows, version 3.0. The documentation provided in the operational package includes information on specific hardware requirements as well as recommendations on achieving maximum performance in using the system.

Use of HWYCON Knowledge

Although each subsystem is designed to be used separately (except in the case of durability and Fast-Track knowledge that can be accessed from the repair and rehabilitation subsystem), it is expected that information obtained from one subsystem would be used in another. For example, identifying distresses and their causes can assist in the design of concrete for an existing durability problem and selecting materials to be used for a rehabilitation procedure. Such would be the case when alkali-aggregate reactivity was present (determined by CONPAV-D) and where it is necessary to reconstruct the pavement (using the CONMAT alkali-aggregate reactivity knowledge to select the materials). In this case, the full-depth repair procedure (using CONPAV-R, which gives materials-related guidance on the procedure) could be used to perform the pavement rehabilitation.

HWYCON's conclusions and recommendations are meant as decision-making tools. The final responsibility for the decision still lies with the user. Although the system contains high-level

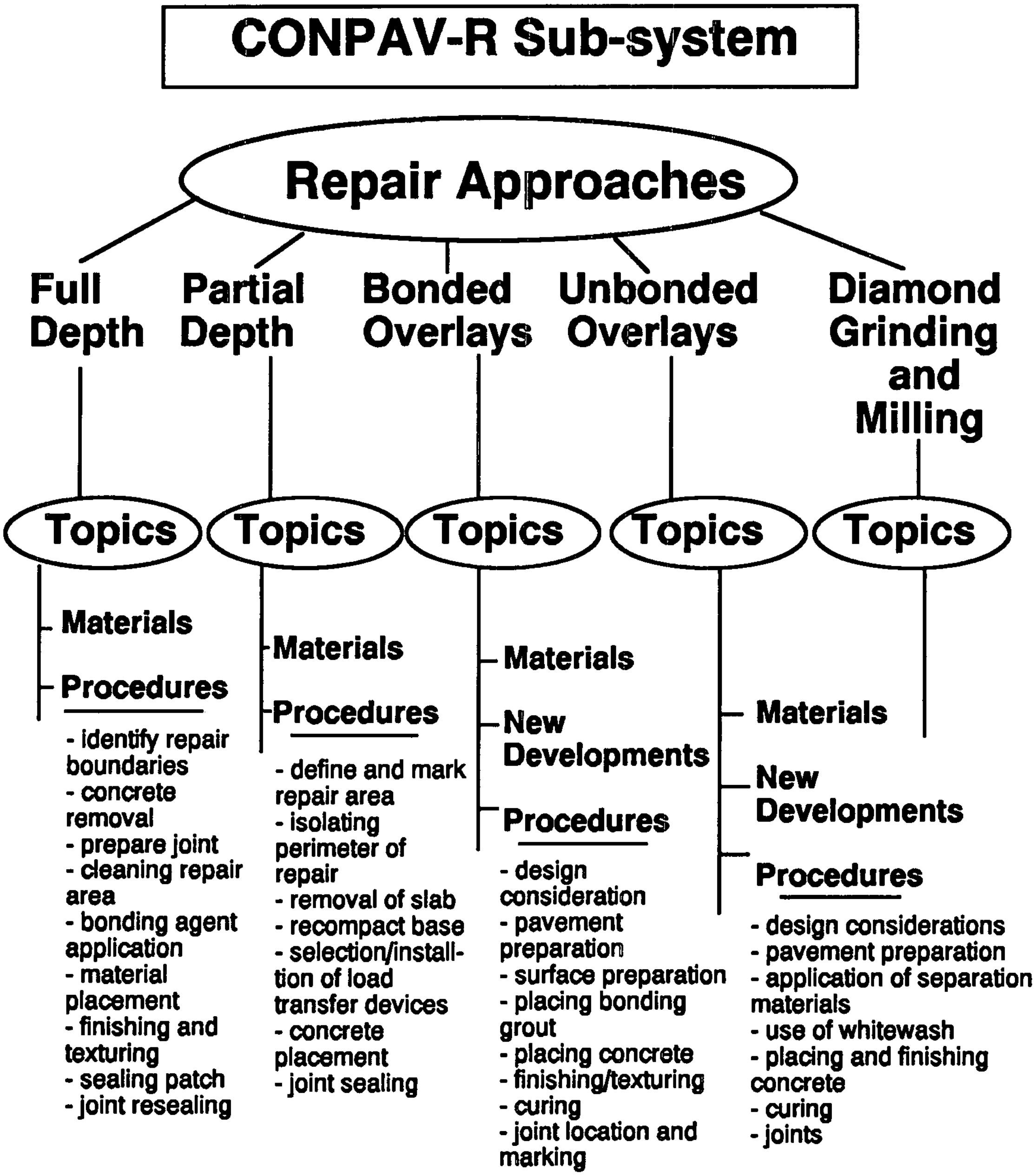

Figure 5.9. Diagram of the CONPAV-R subsystem showing the procedures covered in the system.

information, it is important to understand that variations can occur in the perception of the structure's performance and condition and that the misstatement of the observer or absence of information may invalidate the recommendation. Users are encouraged to conduct the tests and procedures recommended by the system. Standard test methods should also be used in predicting and measuring the performance of materials. If a result is inconclusive after all tests and procedures have been exhausted, an expert familiar with the problem should be consulted. HWYCON makes recommendations on many of the common problems that exist as well as on the most recent information on materials selection and procedures. As new knowledge is developed, it should be included in future versions of HWYCON, and users should acquire new versions of the HWYCON system as they become available.

Modifications to HWYCON

As stated earlier, the focus of the HWYCON system is on delivering a system that could be used by state DOTs. It is expected that an independent organization or group will be responsible for making further changes to the system and keeping it current. Therefore, it is not recommended that the HWYCON source knowledge diskettes be distributed to the states because few, if any, states possess staff familiar with developing expert system applications. Furthermore, purchasing the Level5 Object Development System tool would be required to change the system's knowledge or logic. It is important to maintain this policy to retain integrity in the system. Several scenarios are provided in a later section that describe maintenance and enhancement of the system.

At this point, a brief overview of the level of effort and staff requirements for developing the HWYCON system is appropriate. First, it is the NIST experience that developing expert systems should involve a team approach. The team should consist of in-house expert(s) active in the knowledge domain, which is important for knowledge interpretation, organization, and acquisition as well as for the evaluation of user feedback. This person can best be characterized as the "domain expert" and should be able to describe the system scope and approach. It is also important to have a team member(s) who are familiar with computer technology (e.g., hardware, development languages, and tools) and who can communicate effectively with the expert team members. This second part of the team, often referred to as the knowledge engineers, should also be familiar with the concepts of the knowledge domain. This is the approach used in developing HWYCON. To make future changes to HWYCON, the knowledge engineers must become familiar with the Level5 expert system tool. The learning curve to become proficient in using the tool could take 1 year, depending on the complexity and nature of the changes. Once a team is proficient, changes to the knowledge base structure and the system logic can be made very rapidly.

Another important consideration in using and modifying HWYCON is the availability of adequate computer facilities (computer hardware and software). This consideration is important so that optimum performance can be achieved when using the HWYCON system. Early in the project, an analysis of the computing resources available in state DOTs revealed that the most common computer platform was the IBM or compatible personal computer running the DOS software operating system. Also, because the use of visual information was important in the use of the system, a graphical user interface was selected. As a result, the Microsoft Windows environment was selected. The computer's central processor and disk speed were the key factors considered. Currently, 386- and 486-CPU-based desktop or portable computers are the only models recommended. Table 5.5 shows the recommended features for the system.

Table 5.5. Recommended computer resource requirements for the HWYCON expert system.

Component	Recommendation
Central processing unit	386 or 486 based
Display monitor	Color, 1024 × 768 pixels
Disk speed (average access time)	Between 12 and 23 milliseconds
Main memory capacity	4 megabytes
Disk storage requirements	15 megabytes for HWYCON
Performance enhancements	Disk caching
Operating system	DOS
User interface	Microsoft Windows 3.0 or 3.1

Future Enhancements to HWYCON

For expert systems to be successful, they must be nurtured and kept current. This involves mechanisms for making modifications as knowledge and needs change and for including new knowledge. Under SHRP C-206, guidelines were established for the scope of knowledge and the criteria for implementing the system. HWYCON can be modified to address other problems, and the scope can be enlarged. This would make the system more comprehensive and useful to state DOTs. In the three areas that HWYCON addresses, specific recommendations for future improvements have been identified.

For example, causes of distresses often involve factors other than materials. Other distress types could be added to include those resulting from interactions between concrete and soil as well as structurally induced cracks in concrete pavements. Other SHRP projects will also be completed concurrently with C-206, and it will be important to include this newly developed knowledge in HWYCON. An example of this new knowledge comes from SHRP C-104, which addresses the corrosion of reinforcing steel associated with bridge decks.

Additional knowledge being developed in the SHRP C-205 project on high-performance concrete could be included in the CONMAT selection of materials module. Intrinsic materials-related problems could also be added to CONMAT. These would include areas such as plastic shrinkage and thermal cracking.

As stated earlier, the concrete repair and rehabilitation subsystem (CONPAV-R) assumes that the user has already selected the appropriate procedure. This subsystem could be enhanced to recommend a repair method and to consider the cause of the distress; this would be a very powerful feature. Other enhancements of CONPAV-R could include incorporation of new knowledge on new repair or maintenance materials, such as the results on repair and rehabilitation from SHRP C-104.

New operational features could also be included; several were identified in the review and feedback process from the prototype systems but were not included due to resource limitations.

It was stated earlier in this report that continued maintenance of the HWYCON expert is important if the system is to be useful. Several organizations could provide this function. These are named below, along with a brief statement giving an example of how each organization might conduct the activity.

Federal Highway Administration (FHWA): FHWA maintains an Office of Technology Applications and has sponsored projects relating to the development of expert systems for transportation. This office could provide the environment for enhancement and continued development of HWYCON within FHWA through contracts to private industry or other government agencies.

National Cooperative Highway Research Program (NCHRP): NCHRP, through its pool fund program, could fund further development of HWYCON. This would involve obtaining support from several state DOTs that indicated an interest in the further development of the computerized system. It would also provide the opportunity for states to customize the system where common interests were shown.

American Association of State Highway Transportation Officials (AASHTO): On the basis of interest from the AASHTO constituency, its members could provide funding for the continued maintenance and enhancement of HWYCON. This activity could involve a region, several regions, or the entire nation. Involving a single region (or several regions individually) would allow a separate system to be refined to meet the need of states that have similar problems. Examples include the use of materials for construction and reconstruction as well as the occurrence of similar distresses and their causes based on exposure conditions. This alternative would also result in a more effective means of continued maintenance for the system through AASHTO. Future versions would be developed from a consensus among AASHTO members.

6

Implementation Packages for Highway Personnel

Needs and Approach

The implementation packages described in this chapter represent the culmination of efforts in this study and, to a large extent, the culmination of efforts in SHRP's concrete research program. Implementation is also the key to the success of SHRP as a whole. The SHRP program was funded to not only find solutions to many of the problems being encountered in highway engineering and maintenance, but also to get these solutions to the practitioners in state and local highway agencies quickly and in forms that will be readily understood and applied. There are an almost infinite variety of problems in the highway field and probably as many solutions, not all of which can be addressed in a single project or series of projects. It was the goal of the research team to select some of the most important problems as well as appropriate solutions and to present these to the highway engineering community in a series of implementation packages that are practical and understandable.

These implementation packages are directed toward a variety of personnel with differing needs. Policy-level personnel need to be advised of new developments on which to base their decisions regarding construction and maintenance budgets. Supervisors need to know details concerning the materials, equipment, work procedures, and labor requirements for their areas of responsibility. Construction and maintenance technicians need similar information plus any special requirements for testing and quality control. It is especially important that the information be useful and understandable by entry-level technicians and engineering personnel who must spend much of their time in hands-on activities and cannot invest large amounts of time in reading detailed texts or manuals. Educational videotape presentations are especially useful in this regard and were heavily relied upon in preparation of the implementation packages.

A total of four implementation packages were produced. Each package consists of the following materials: (1) educational videotapes utilizing professional narration and produced at actual construction sites or laboratory facilities (whenever laboratory procedures were being demonstrated)—a total of twelve tapes in all; (2) for each tape, a parallel set of 35-mm slides with a moderator's outline for use when video facilities might not be available or for large audiences to whom video monitors would not be readily visible; (3) illustrated users' manuals that detail the equipment, materials, and procedures covered by the tapes (the manuals are intended as reference material that would include details not appropriate to the video presentations); (4) instructors' guides for each session so the packages can be used later by personnel not immediately involved in the initial development of the materials; and (5) where appropriate, pocket guides that list the materials and equipment, testing requirements, and construction methods necessary to carry out the particular applications and testing being discussed and that are suitable for use in the field.

All of the above-mentioned audiovisual and text materials are utilized in each of the four implementation packages. The packages are outlined and described in some detail below, and the individual videotapes are noted. The packages are excellent training aids for both laboratory and field personnel interested in increasing their expertise in various areas of concrete highway technology. All four packages are not required to obtain benefits from these products. It is likely that laboratory personnel will be primarily interested in Package 1, which deals with laboratory testing for concrete and aggregate durability. Both laboratory and field test personnel may share interest in Package 2, which describes various conventional and novel field test methods. Packages 3 and 4 will have the most utility for state construction inspectors and for contractor personnel who perform the activities depicted in these field tapes. Again, as was the case for Chapters 1 and 2 of this report, this chapter serves only as a broad overview of what is contained in the packages. Individual agencies should obtain the packages to carry out appropriate training.

Description of Packages

Package 1—Durability Testing of Concrete and Aggregates

- Alkali-silica reactivity (ASR) testing (Tape 1)
  - — Seriousness and extent of problem
  - — Available tests and their limitations (ASTM C 289, C 227)
  - — Newly developed tests (SHRP hot NaOH soak)
  - — Detection of ASR (gel-staining technique)
- Freeze-thaw testing (Tape 2)
  - — Importance of freeze-thaw testing and illustrations of damage
  - — Testing of concretes versus testing of aggregates
  - — Conventional testing (AASHTO T 104, T 161)
  - — Newly developed tests for concrete (damping test, cloth wraps)
  - — Newly developed test for aggregate (hydraulic fracture)

Package 1 illustrates the various new methods for evaluating the durability of concretes and aggregates, therefore allowing more choice in materials by state agencies. While concentrating on alkali-silica and freeze-thaw testing (the primary foci of the SHRP concrete program), the users' manual contains additional information on other areas of concrete durability such as corrosion, testing for chloride content, and sulfate attack.

In the section relating to alkali-silica reactivity, the videotape starts with an introduction to the problem, history, some description of the mechanism, and examples of distress. The currently used tests, such as ASTM C 289 (the quick chemical test) and ASTM C 227 (the mortar bar test) are illustrated, and their deficiencies are discussed. The newly developed tests—primarily those developed under SHRP C-202, *Eliminating or Minimizing Alkali-Silica Reactivity*—are then introduced, and their applications are demonstrated and defined. These tests include the hot alkali expansion methods, as applied to mortars and concretes based on ASTM proposed test procedure P 214. Detection of ASR gel products using the uranyl acetate staining procedure (Stark 1991) is also included in the package.

The section on freeze-thaw testing begins with an introduction to the problem, illustrating how concretes are damaged by repeated cycles of freezing and what damage can result. The differences between freeze-thaw damage due to failure of the matrix (because of an improper air-void system or the lack of it, high w/c, and exposure to early freezing) and freeze-thaw damage due to frost-susceptible aggregates are clearly defined. Conventional methods for testing both concretes and aggregates are illustrated. These include both T 161 freeze-thaw testing and T 104 sulfate soundness testing. Although many other test methods exist in this area, only the most commonly accepted methods were chosen for illustration in this manual. Research methods and methods used by a limited number of agencies are therefore not included. The freeze-thaw part of the package primarily focuses on new developments generated in SHRP C-203, *Resistance of Concrete to Freezing and Thawing* (Janssen forthcoming). This includes the Washington pressure test for d-cracking (Janssen and Almond 1991), which uses hydraulic fracture as a criterion for judging the susceptibility of an aggregate source to freeze-thaw deterioration; the use of damping (Q value) to gain an early indication of damage during freeze-thaw testing; and modifications made to T 161 in which specimens are wrapped in wet cloths to prevent drying as they are frozen in air during each cycle.

Package 2—Quality Control of Concrete On-Site

- Conventional control practices
  - As-delivered concrete (Tape 3—sampling, temperature, slump)
  - As-delivered concrete (Tape 4—air content, unit weight, test cylinders and beams)
- New control practices (Tape 5)
  - The w/c (water content determined using microwave oven)
  - Concrete thermal control tables
  - Maturity monitoring

- New control practices (Tape 6)
  - Temperature-matched curing
  - Pulse velocity testing
  - Consolidation (using twin-probe nuclear density meter)

Package 2 can be used to illustrate proper control of concrete on-site. Both traditional methods of control (such as proper sampling and use of temperature, slump, and air content test procedures) as well as new methods developed by SHRP and others are included in the package. Tapes 3 and 4 consist of a synopsis of conventional control tests, which include sampling (AASHTO T 141), slump (AASHTO T 119), air content by the pressure method (AASHTO T 152), unit weight (AASHTO T 121), and temperature (ASTM C 1064). Instructions are also given on proper preparation of field test cylinders for compressive strength and field test beams for modulus of rupture. These follow AASHTO T 23 procedures.

Tapes 5 and 6 illustrate relatively newer control procedures that have been field tested during SHRP C-206. One of the most critical characteristics of as-delivered concrete is the actual w/c of the concrete delivered to the site. Additions of water are often not accurately monitored, so the actual w/c of the concrete becomes open to question. Because cement is normally batched fairly accurately and not normally added after the truck has left the plant, a knowledge of the true water content should account for most of the variance in w/c. Rapid drying of concrete using a microwave oven has proven a quick and relatively accurate technique for determining water content; this is included in Tape 5.

SHRP has published a set of tables (Andersen, Andersen, and Whiting 1992) that can be used to determine whether concrete will be susceptible to a number of thermally related problems. These include such concerns as early freezing of concrete, thermally induced cracking due to thermal gradients, and excessive internal temperature during curing. Proper measurement of concrete and surface temperatures as well as use of the tables is illustrated in this tape.

It is now recognized that the strength of concrete is a function of its maturity, a time/temperature function (Carino, Lew, and Volz 1983). The relationship between maturity and strength is a function of the particular materials and mixes employed. After strength is precalibrated against maturity, determining strength at any time on an actual field job is then simply a matter of monitoring the internal temperature of the as-placed concrete. A simple explanation of the background of the method, a description of precalibration procedures, and recommendations for placement and monitoring of thermocouples for temperature measurement in the field are documented in this tape.

One area in which quality control practices are currently lacking is the area of as-placed concrete. After slump and air content have been verified and test specimens prepared, little (if any) testing is carried out on the finished pavement. Tape 6 illustrates some new field tests evaluated under SHRP C-206. The first procedure described is a direct measure of compressive strength using standard cylinders that can be obtained through the use of temperature-matched curing (TMC). Only limited use has been made of this technique in the highway field (Sprinkel and McGhee 1989), and further evaluation was warranted. The equipment consists of insulated cylinder molds (4 × 8 in. [100 × 200 mm]) that contain heaters activated by a controller that senses the temperature in the concrete slab through a thermocouple placed at mid-depth. The heaters keep the temperature in the cylinders the same as the temperature at mid-depth in the slab. The tape demonstrates the use of the system and the testing of cylinders in the field using a portable compression tester.

The next segment deals with the use of pulse velocity testing to estimate concrete strength (In-place methods 1988). Pulses of compressional waves can be generated in concrete by an electro-acoustical transducer held in contact with one surface of the structure. The time required for the pulses to travel through the concrete from the transmitting transducer to the receiving transducer being held in contact with the pavement is measured electronically. Pulse velocity is calculated by dividing travel distance by transit time. If specimens cast from trial mixes have been used to establish a pulse velocity-strength relationship, then the relationship can be used to estimate strength in the field. The tape illustrates the use of the equipment on a full-depth pavement slab replacement project.

The final segment in Tape 6 describes the use of the twin-probe nuclear density gage. In contrast to conventional backscatter or transmission gages (Whiting and Tayabji 1988), which either measure surface density only or an average density throughout the depth of the concrete, the twin-probe nuclear density gage can determine density of concrete across a 12-inch (305-mm) path at any single depth in the pavement. Reference to a completely consolidated sample can help determine the degree of consolidation.

Package 3—Rehabilitation of Highway Concrete

- Introduction (Tape 7)
  - Topics to be covered
  - Short examples of rehabilitation methods (full- and partial-depth repairs, overlays)
  - Materials developments (faster strength gain, lower permeability)
  - Pavement repairs
- Full-depth repairs (Tape 8)
  - Requirements for full-depth repair
  - Load transfer
  - Repair materials options
  - Isolation of removal area
  - Concrete removal
  - Dowel installation
  - Concrete placement
  - Curing and opening

- Partial-depth repairs (Tape 9)
  - When to use
  - Selection of materials
  - Boundary repair
  - Concrete removal
  - Surface preparation
  - Joint preparation
  - Bonding agent application
  - Material placement
  - Curing
- Pavement overlays (Tape 10)
  - Types (bonded and unbonded)
  - Preoverlay repairs for bonded overlays
  - Surface preparation and bonding agent
  - Concrete placement and curing
  - Joint sawing
  - Separation layers for unbonded overlays
  - Concrete placement and curing
  - Joint location and sawing
- Bridge deck overlays (Tape 11)
  - Need for deck overlays
  - Examination of deck condition
  - SHRP decision model
  - Materials and concrete mixes
  - Concrete removal
  - Surface preparation
  - Placement of equipment and setup
  - Placement of overlay
  - Finishing and curing

In this package, an introductory tape offers an overview of what is to be covered in more detail later in the package. Such topics as partial-depth spall repairs, full-depth joint repairs and full slab replacements, and concrete pavement and bridge deck overlays are introduced. The availability of rapid-setting concrete materials that make concrete a cost-competitive material for these types of repairs is emphasized. The tape includes short action sequences of all the repair techniques covered later in the bulk of the package.

Full-depth repairs of jointed pavement are covered in the next tape. The necessity for full-depth repair is tied to the severity of damage. Transverse joint design and the proper use of dowel bars and tied joints are discussed. The increasing use of fast-setting mixes for full-depth repairs is presented, including the use of special cements to achieve very early opening of repaired sections. The details of repair steps are then treated. This includes sawing to isolate the repair area; lift-out of the slabs; dowel hole drilling and dowel insertion (including the use of grout retention disks); and placement, consolidation, and finishing of concrete. Texturing and curing are recommended as preferred practices. Finally, recommendations on joint sawing and criteria for opening are presented.

Tape 9 deals with partial-depth pavement repairs, beginning with a discussion of the appropriate circumstances under which partial-depth repairs may be successfully used. Materials that can be used to achieve rapid-setting repairs are discussed next, including high early strength cements and proprietary rapid-setting products. The procedures are then illustrated. Considerable emphasis is given to establishment of repair boundaries and removal of concrete. Sawing, chipping, cold-milling, and water blasting are all illustrated, and the advantages and disadvantages of each are discussed. Surface preparation after concrete removal is also addressed. The use of joint inserts is treated next, followed by a discussion of the application of bonding agents. The tape concludes with a discussion of placement of material into the repair areas, consolidation, finishing, and curing.

Tape 10 illustrates procedures for the placement of both bonded and unbonded pavement overlays. It opens with a discussion of which type of overlay is appropriate to which pavements and distress conditions. Bonded overlays are then discussed, beginning with what preoverlay repairs are needed on slabs and joints and including such practices as cross-stitching longitudinal cracks and sealing deteriorated joints. The surface preparation methods of shotblasting, cold milling, sandblasting, and waterblasting are then shown and discussed. The placement of bonding grouts after preparation is included. The section on bonded overlays concludes with a discussion of concrete placement and curing, followed by a discussion of requirements for joint sawing and sealing.

The section on unbonded overlays begins with a discussion of characteristics and thickness of the separation layer related to the types and degrees of distress in pavement. The placement of the separation layers using hot-mix asphalt (the preferred system) is then illustrated. Concrete placement and curing are the same as for bonded overlays. The technique of sawing joints to mismatch those in the original placement is noted. The tape concludes with discussions of proper joint spacing and the use of tied shoulders.

Although most of the units in Package 3 emphasize the use of concrete for highway pavements, the need for repair of highway bridges damaged by the corrosion of reinforcing steel was also recognized. Although most of the efforts in this area of the SHRP program have been addressed in other projects, a module describing the proper construction of bridge deck overlays was needed because this is a very popular technique for repairing corrosion-damaged bridge decks. The tape begins with a discussion of the need for bridge deck overlays and methods used for examination of deck condition. These include such techniques as sounding for hollow or delaminated areas, detection of delaminations using penetrating radar, chloride sampling, and measurement of half-cell potentials. The SHRP decision model (Weyers 1993), which can be used to select the most cost-effective method of bridge repair, is referenced. The tape then discusses materials such as latex-modified and silica fume concretes, which are widely used to obtain low permeability overlays. Mix designs and requirements are included in the discussion. Techniques for concrete removal, including hydrodemolition and rotomilling, are shown. Final surface preparation and repairs of deteriorated reinforcement are then addressed. Placement and setup of equipment is discussed, as are monitoring of environmental conditions and limitations on placement temperatures. The placement process for latex-modified overlays is then shown. Both latex and silica fume overlays are seen to be placed almost identically, the major difference being the use of ready-mix trucks for silica fume delivery and mobile concrete mixers for latex-modified concrete delivery. The tape concludes by emphasizing proper curing, using moist curing techniques, and skid texturing of the surface.

Package 4—Early Opening of Full-Depth Concrete Repairs (Tape 12)

- Why early opening
- Factors to be considered
- Determining production rates for construction activities
- Concrete removal
- Dowel insertion
- Contingencies
- Scheduling
- Opening of repairs and criteria

Package 4 addresses a topic that has garnered wide interest in recent years: early opening of concrete pavement repairs. Early opening technology allows the design of concrete mixes and the use of curing techniques that promote the development of required opening strengths at intervals ranging between 24 hours and 6 or fewer hours. The result is that concrete can now be used for repair purposes on projects that, in the past, were not considered feasible for concrete due to the need for lengthy cure times. The ability to open early to traffic is especially important in areas where high costs are associated with pavement downtime, such as in urban locations (especially at intersections) where traffic flow is very heavy and rerouting is not easily accomplished.

This presentation emphasizes determination of production rates for various activities to avoid time-consuming bottlenecks. The most efficient methods for concrete removal and dowel bar insertion into adjacent slab faces are discussed. Scheduling to complete all activities without delay in subsequent construction is emphasized. The criteria for opening and the use of in-place test methods for determining opening times are also treated in the video.

7

Conclusions and Recommendations

SHRP C-206, *Optimization of Highway Concrete Technology*, was a multifaceted program consisting of a number of parallel activities. In contrast to a classically focused research project in which investigators attempt to evaluate a single hypothesis, C-206 combined the fields of materials research, test method evaluation, technology transfer, and technical training in a single effort. Included were the areas of knowledge assimilation and transfer (represented by the synthesis document and the HWYCON modules), field research (represented by the efforts on bridge deck overlays and early opening of full-depth repairs), technology transfer and education (represented by the instructional videos and applications manuals), and quality control (represented by evaluation of a variety of new concrete test procedures). This chapter presents conclusions that can be drawn from each of these efforts, along with recommendations as to how the findings can best be implemented into current highway practice. In addition, when further work is required before the results obtained in this project can be implemented, recommendations about how to proceed are given.

As noted above, it is difficult to draw any global conclusion from a project so structured. However, if any one lesson can be learned from these efforts (as well as the efforts of the other SHRP projects in the concrete series), it is that concrete is a highly complex product sensitive to both its constituent materials and how it is produced and handled under real-world conditions. Many specifiers do not recognize this primary characteristic of concrete and treat it like a highly controlled metallic alloy or similar material that is produced under controlled conditions using practically the same constituent materials irrespective of locality. Concrete materials are highly dependent on local sources, and the concrete that results from the combination of these materials reflects this uniqueness. Furthermore, given two identical concretes, how they are transported, placed, consolidated, and cured can result in dramatically different performances. In the past, these characteristics have been learned after long years of field experience; the abilities of our highway system to carry traffic loads not anticipated by original designers speaks well of the knowledge and expertise of dedicated and experienced personnel in the many U.S. highway agencies. However, a great number of these personnel are completing their terms of service, and responsibilities are handed to new, less experienced engineers and technicians. To be able to effectively utilize the many new concrete materials and mixes being introduced, young engineers need to gain a rapid appreciation of the material qualities and handling characteristics of concrete. They should read this report and be intrigued by the fact that this information is now available through such formats as HWYCON and the implementation packages. Using these and other educational materials should contribute to a better appreciation of the care needed in testing and producing concrete by this new generation of practitioners, and concrete highways and structures should therefore continue to provide safe and efficient transportation facilities.

Synthesis of Current and Projected Concrete Highway Technology

The synthesis developed under SHRP C-206 and entitled *Synthesis of Current and Projected Concrete Highway Technology* should serve as a useful primer covering many aspects of portland cement concrete, its constituent materials, and their applications to important areas of concrete repair and rehabilitation. New engineers can use the document to obtain information on available cements, characteristics of aggregate materials, and performance of materials more recently introduced, such as silica fume and high-range water reducers. It is often difficult for highway personnel—especially those serving in district or field offices—to find such information condensed into one volume. The summary material on concrete batching and highway construction should be especially useful to field personnel. The chapters on new control methods and statistical quality assurance should be helpful to anyone seeking to implement new procedures within their own agencies.

The synthesis document should be advertised as an aid to practitioners in the highway field rather than as a reference document for the literature reviews of researchers and others. If the new approaches described in the document are to be used, agency personnel must be made aware of their existence and must have access to full descriptions of and reference to the original publications contained in the synthesis. Thought should also be given to procedures for periodic updates of the information in the document, perhaps by scheduling a revised document within the NCHRP synthesis series or by publishing revised editions at later dates under FHWA technology transfer programs.

Highway Concrete Expert System

In this project, an expert system (HWYCON) was developed to assist state highway departments in (1) identifying and determining the cause of materials-related distresses; (2) selecting materials to avoid such problems as alkali-aggregate reactivity, freeze-thaw action, corrosion of reinforcing steel, and sulfate attack; (3) selecting materials for recycled concrete, permeable bases, and Fast-Track concrete; and (4) selecting materials for full-depth and partial depth repairs, bonded and unbonded overlays, and diamond grinding and milling. Structures addressed in the system include jointed and continuously reinforced pavements, bridge decks, and other highway structures (e.g., columns, piers, and parapet walls). The system can be used by inspectors and engineers, concrete specifiers, and decision makers whose knowledge about the activities range from novice to mid-level. The conclusions and recommendations given by the expert system are intended as decision-making tools. The final responsibility for the decisions still lies with the user. Users should first apply the system to hypothetical situations or noncritical applications so they can become familiar with the scope and limitations of the system before applying it in more critical circumstances.

A platform by which the system can be updated, enlarged, and/or customized to fit the needs of particular agencies should be found. This could include maintenance by the FHWA Office of Technology Applications, under NCHRP programs, or under national or regional levels of AASHTO. New knowledge that arises from research or demonstration programs could then be added to this system so that users can obtain the most up-to-date information available. In addition, the system should be maintained so that it remains compatible with the most recent computer software and hardware.

Field Studies of New Test Procedures, Materials, and Criteria for Concrete Rehabilitation

Field studies were carried out to evaluate concrete materials and mixtures that show great promise for use in early opening applications and for use where concrete is subject to extremely adverse environments. The applications chosen included (1) full-depth pavement repairs for when traffic must return to the pavement in a relatively short time and (2) bridge deck overlays for when early opening would help reduce the need for extended bridge closures.

Concrete materials and mixtures for these studies were chosen on the basis of development carried out in other SHRP studies, independent R&D programs, and private industry. These included Very Early Strength (VES) and High Early Strength (HES) mixes developed under SHRP C-205 (*Mechanical Behavior of High Performance Concrete*); Fast-Track concrete and early opening mixes using chemical accelerators; and mixtures utilizing the proprietary materials Pyrament cement and Rapid-Set cement. For bridge deck overlays, latex-modified concretes using Type III cement and silica fume concretes were evaluated. Again, the results obtained with these materials evaluations are strictly valid only for the particular materials and mixes used in this research program. Concrete is a heterogeneous material, the performance of which is highly dependent on local materials and conditions. Extrapolation of these conclusions to other materials or circumstances should be done only with caution after consideration of the particular conditions that may be encountered. From the results of these series of field evaluations, the following conclusions were drawn.

- Concretes used for early opening applications have high rates of early strength gain and develop considerable heat during the first few hours after placement.

Temperature increase is greatest for mixes using large amounts (more than 800 lb/yd^3 [470 kg/m^3]) of Type III cements and chemical accelerators. These high temperatures and accompanying rapid strength gains allow sections cast with these mixes to be opened to traffic in 4–6 hours. Proprietary rapid strength gain cements, such as Pyrament and Rapid-Set, exhibit rapid rates of strength gain but release less heat and generate lower peak temperatures within the repair slabs.

- A variety of methods can be used to predict in-place strength gain of concretes designed for early opening applications. Techniques such as maturity, pulse velocity, temperature-matched curing, and the use of well-insulated test cylinders could all estimate strength within an average of ±20% of the strengths determined on cores obtained from the sections. The use of plastic cylinder molds (4 × 8 in. [100 × 200 mm]) placed in a thick section of rigid foam insulation cored to accept the cylinders offers an inexpensive and simple alternative to many of the more complex nondestructive testing methods, although a portable compression tester is needed to test the cylinders in the field.

- Most of the early opening mixes continued to gain strength up to at least 90 days after placement (the longest test period included in this study). Strengths as high as 10,000 psi (70 MPa) were reached with mixes prepared with Pyrament cement. Although exhibiting more rapid rates of gain of early strength (2–4 hours), Rapid-Set cement showed only a moderate increase in strength after 1 day.

- On the basis of results obtained from fatigue analyses, it appears that the strengths typically required for early opening to traffic—a flexural strength (MOR) of 300 psi (2.1 MPa) or a compressive strength of 2,000 psi (13.8 MPa)—are reasonable criteria under most conditions for repairs up to 12 ft (3.7 m) in length. The compressive strength is a more accurate indicator of early opening for very short sections—6 ft (1.8 m) or less—because dowel bearing stress is the critical factor in these cases.

- Proper air content and adequate air-void systems are necessary—but not sufficient—conditions for obtaining the desired freeze-thaw durability of the early opening mixes. High temperatures generated early in the lives of these concretes may contribute to microcracking and poor freeze-thaw durability.

- Bridge deck overlays that utilize latex-modified concretes can be opened to traffic within 24 hours of placement if Type III (High Early Strength) cement is substituted for Type I cement in the mix. Strengths at 24 hours should exceed 2,000 psi (14 MPa) when this approach is used, provided that w/c is maintained below 0.40 and minimum temperatures during the first 24 hours after placement do not fall below 50°F (10°C).

- Durable concrete overlays having very low permeabilities can be produced using silica fume concrete (SFC). Compressive strengths of SFC mixes will generally be significantly higher than those of latex-modified concretes (LMC) used for overlays, while the tensile strengths of the two systems are roughly equivalent. Permeability to chloride ions, as measured by the AASHTO T 277 test, will generally be lower for SFC than for LMC mixes.

Long-term follow-up studies (covering up to 10 years) should be conducted on these test sections to evaluate their performance under the combined actions of environment and traffic loading. For the pavement sections, visual ratings, falling-weight deflectometer, and coring to allow for strength and petrographic examinations are recommended. Recommendations for follow-up studies on the bridge decks include half-cell corrosion surveys, chloride sampling, delamination mapping, and measurements of the rate of corrosion.

Questions regarding the durability of concretes designed for early opening applications still remain. As noted above, microcracking in the concretes, which may have been thermally induced, may account for some of the poor performance in freeze-thaw testing. The use of calcium chloride should be avoided because it contributes to reduced freeze-thaw resistance. While the freeze-thaw test used to evaluate durability is admittedly very severe, additional work on the durability of early opening mixes should be performed before their use in northern areas subjected to harsh winter conditions is unequivocally recommended.

These questions and the need to monitor the test sections over the long term should not inhibit additional applications of early opening technology to pavement repair projects. Indeed, as was observed during the conduct of the research, some highway agencies have already developed their own early opening mixtures, although the handling characteristics and durability of these mixtures may need further refinement. It is recommended that the mixes and guidelines included in this report and the accompanying implementation materials (HWYCON expert system and implementation packages) be applied by state agencies to projects in which early opening to traffic may result in cost savings and reduction in inconveniences to the public.

Development of Test Methods for Fresh Concrete

Under this project, four areas dealing with control methods for fresh concrete and mix optimization were investigated. These were evaluation of a technique for field determination of the water content of fresh concrete using microwave drying, development of a guide for prediction of temperature effects in newly placed concrete slabs, development of a handbook for use in optimizing the proportioning of aggregates for concrete mixes, and development of a technique for measuring the density of fresh concrete at various depths within a freshly placed concrete slab.

From the results of these evaluations, the following conclusions and recommendations were developed.

- A microwave oven procedure for determining the water content of fresh concrete was successfully developed. The procedure consists of drying a 1,500-g sample of fresh concrete in a 900-watt microwave oven equipped with a turntable to provide uniform drying. The total test time is approximately 16 minutes, including time for sample manipulation and check-weighing twice during the drying process. The technique was sufficiently reproducible for purposes of field control and worked independently of absorption of aggregates or consistency of concrete. The method can be applied to latex-modified and silica fume concretes as well as more conventional mixes. The method should be adopted by agencies on a trial basis so it can be further evaluated by using a wider variety of materials and conditions than could be included in the limited test program described in this report. A preliminary test procedure following AASHTO format is included as Appendix C of this report. Modifications to this procedure may be appropriate when further experience with the technique has been obtained.

- A guide developed by SHRP under C-206 (Andersen, Andersen, and Whiting 1992) demonstrates how knowledge of concrete temperature, air temperature, and concrete mix characteristics allows the user to determine whether temperature-induced problems may be expected.

  The following parameters must be entered before the tables can be used: type of cementitious material, content of cementitious material, concrete temperature, air temperature, and thickness of concrete pavement. Examples of the use of the tables for various combinations of parameters are included in the guide. The guide is not all inclusive, and situations falling outside of the limits imposed on the variables cannot be treated by use of the tables. Further development in this area would be beneficial, as would development of computer-assisted guidance for avoiding thermal effects that could also take into account stresses imposed by shrinkage and early loading.

- A handbook produced under C-206, *A Guide to Determining the Optimal Gradation of Concrete Aggregates* (Andersen and Johansen 1993) describes the application of particle packing theory to the production of densely packed aggregate skeletons in a concrete mixture. This handbook consists of tables showing the relationship between the characteristics of the aggregate particle size distribution as well as the packing density and the packing of two or more different materials. The tables can be used to efficiently blend coarse aggregate fractions or mixtures of sand and coarse aggregate. Although some promising results were obtained, a more comprehensive evaluation is warranted before the tables can be recommended for widespread use.

- A twin-probe nuclear density meter that can be used to determine the density of horizontal strata in a fresh concrete slab was evaluated as part of this research study. The gage was tested in a series of void-free concretes; differences between densities measured by using the gage and the density of the fresh concrete as determined by unit weight measurement averaged approximately 1.2 lb/ft^3 (19 kg/m^3). When applied to specimens containing steel dowel bars, the gage accurately determined the density of successive layers of concrete to within 0.8 in. (20 mm) of the dowel bar. The presence of voided areas in the concrete between the source and detector can easily be detected by the gage. Reproducibility of the test method, as measured by standard deviation, was 1.8 lb/ft^3 (28.8 kg/m^3) at a concrete density of 150 lb/ft^3

(2,400 kg/m^3). Additional equipment modifications that would make the test method easier to use in the field should include reducing test time, decreasing equipment size and weight, updating electronics and displays, and providing for the escape of air from the sampling tubes so the tubes can be more easily inserted into the concrete.

Implementation Packages for Highway Personnel

A set of implementation packages was prepared to allow for the transfer of many of the products developed under the SHRP concrete research program to state highway department personnel. The packages consist of videotapes and other educational materials dealing with the following four areas: durability testing of concrete and aggregates, quality control of concrete on site, rehabilitation of highway concrete, and early opening of full-depth concrete repairs. The tapes should be distributed to all highway agencies, and states should be granted rights to produce a limited number of copies for district office use. The tapes can also be incorporated into more formal training programs carried out by state agencies or by FHWA in technology transfer programs.

References

Abdulshafi, O., B. Kvammen, and K. Kaloush. 1990. *Premature failure of latex modified concrete bridge decks overlaid in Ohio.* FHWA/OH-89/009. Columbus, OH: Ohio Department of Transportation.

ACI Standard 306 R88. 1992a. Cold weather concreting. *1992 ACI manual of concrete practice.* Part 2. Detroit, MI: American Concrete Institute.

ACI Standard 214-77 R83. 1992b. Recommended practice for evaluation of strength test results of concrete. *1992 ACI manual of concrete practice.* Part 2. Detroit, MI: American Concrete Institute.

ACI Standard 211.1-91. 1992c. Standard practice for selecting proportions for normal, heavyweight, and mass concrete. *1992 ACI manual of concrete practice.* Part 1. Detroit, MI: American Concrete Institute.

American Association of State Highway and Transportation Officials (AASHTO). 1986. *Guide for the Design of Pavement Structures.* Washington, DC.

American Society for Testing Materials (ASTM). 1976. *Living with marginal aggregates.* Special Technical Publication 597. Philadelphia, PA.

Andersen, P., M. Andersen, and D. Whiting. 1992. *A guide to evaluating thermal effects in concrete pavements.* SHRP-C-321. Washington, DC: Strategic Highway Research Program, National Research Council.

Andersen, P., and V. Johansen. 1993. *A guide to determining the optimal gradation of concrete aggregates.* SHRP-C-334. Washington, DC: Strategic Highway Research Program, National Research Council.

Babaei, K., and N. M. Hawkins. 1987. *Evaluation of bridge deck protective strategies.* NCHRP Report 297. Washington, DC: National Research Council.

Barrow, R. S., K. M. Hadchiti, P. M. Carrasquillo, and R. L. Carrasquillo. 1989. Temperature rise and durability of concrete containing fly ash. *ACI SP-114. Fly ash, silica fume, slag, and natural pozzolans in concrete*, 331–347. Detroit, MI: American Concrete Institute.

Bishara, A. G. 1980. Latex modified concrete bridge deck overlays: Field performance analysis. *Transportation Research Record* 785:24–32.

Bower, L. C., and B. B. Gerhardt. 1971. The effect of good vibration on the durability of concrete pavement. *Highway Research Record* 357:1–23.

Brown, D. N., and O. O. Thompson. 1973. *Lateral distribution of aircraft traffic.* Miscellaneous Paper S-73-56. U.S. Army WES. Springfield, VA: National Technical Information Service.

Burg, R. G., and B. W. Ost. 1992. *Engineering properties of commercially available high-strength concretes.* Research and Development Bulletin RD 104T. Skokie, IL: Portland Cement Association.

Carino, N. J., H. S. Lew, and C. K. Volz. 1983. Early age temperature effects on concrete strength prediction by the maturity method. *ACI Journal Proceedings* 80 (2):199–208.

Carino, N. J., and M. Sansalone. 1990. Impact-echo: New method for inspecting construction materials. *Nondestructive Testing and Evaluation for Manufacturing and Construction*, 209–223. New York: Hemisphere Publishing.

Clear, K. C., and B. H. Chollar. 1978. *Styrene-butadiene latex modifiers for bridge deck overlay concrete.* FHWA-RD-78-35. Springfield, VA: National Technical Information Service.

Darter, M. I., E. J. Barenberg, and W. Yrjanson. 1985. *Joint repair methods for portland cement concrete pavements.* NCHRP report no. 281. Washington, DC: National Research Council.

Dempsy, D. J., W. A. Herlache, and A. J. Patel. 1983. *Environmental effects on pavements.* FHWA/RD-84/115. Springfield, VA: National Technical Information Service.

Ferragut, T. R. 1990. Accelerated rigid paving techniques. *Concrete in Highway Transportation, No. 7.* Skokie, IL: Portland Cement Association.

Glanville, W. H., A. R. Collins, and D. D. Matthews. 1947. *The grading of aggregates and the workability of concrete.* Great Britain, Department of Scientific and Industrial Research, Road Research Laboratory technical paper no. 5. London: Her Majesty's Stationery Office.

Good Mojab, C. A., A. J. Patel, A. R. Romine. 1993. *Innovative materials development and testing volume 5: partial depth spall repair.* Report no. SHRP-H-356. Washington, DC: Strategic Highway Research Program, National Research Council.

Grove, J. D., K. B. Jones, K. S. Bharil, A. Abdulshafi, and W. Calderwood. 1990. Fast-Track and Fast-Track II, Cedar Rapids, IA. *Transportation Research Record* 1282:1–7.

Hall, K. T., J. M. Connor, M. I. Darter, and S. H. Carpenter. 1989. *Rehabilitation of concrete pavements, Volume III: Concrete pavement evaluation and rehabilitation system.* FHWA-RD-88-073. Springfield, VA: National Technical Information Service.

Hall, K. T., M. I. Darter, M. B. Snyder, S. H. Carpenter, and S. W. Forster. 1989. Nationwide performance of concrete pavement rehabilitation. *Proceedings, 4th International Conference on Concrete Pavement Design and Rehabilitation.* FHWA-RD-89-208:137-149. Springfield, VA: National Technical Information Service.

Hansen, F. P., and J. Pederson. 1977. Maturity computer for controlled curing and hardening of concrete. *Nordisk Betong* 1:19–34.

Heinrichs, K. W., M. J. Liu, M. I. Darter, S. H. Carpenter, and A. M. Ioannides. 1989. *Rigid pavement analysis and design.* FHWA-RD-88-688. Springfield, VA: National Technical Information Service.

Howard, J. D. 1988. Marquam bridge repair: Latex modified concrete overlay and joint replacement. *Transportation Research Record* 1204:59–70.

Iddings, F. A., and J. L. Melancon. 1981. Feasibility of development of a nuclear density gage for determining the density of plastic concrete at a particular stratum. Final Report FHWA/LA-81/149. Springfield, VA: National Technical Information Service.

In-place methods for determination of strength of concrete. 1988. ACI Committee report 228.1R. *ACI Materials Journal* 85 (5):446–472.

Janssen, D. Forthcoming. *Resistance of concrete to freezing and thawing.* Washington, DC: Strategic Highway Research Program, National Research Council.

Janssen, D., and D. K. Almond. 1991. A comparison of four aggregates using the Washington hydraulic fracture test. *Transportation Research Record* 1301:57–67.

Kaetzel, L. J., and J. R. Clifton. 1991. *Expert knowledge based systems for cement and concrete: State-of-the-art report.* SHRP-C/UWP-91-527. Washington, DC: Strategic Highway Research Program, National Research Council.

Kaplan, M. F. 1960. Effect of incomplete consolidation on compressive and flexural strength, ultrasonic pulse velocity, and dynamic modulus of elasticity of concrete. *ACI Journal* 31 (9):853–867.

Kelly, R. T., and J. W. Vail. 1968. Rapid analysis of fresh concrete. *Concrete* April:140–145.

Klemens, T. L. 1990. When slab replacement resembles a production line. *Highway and Heavy Construction* 132 (11):28–31.

Knab, L. I., M. M. Sprinkel, and O. J. Lane, Jr. 1989. *Preliminary performance criteria for the bond of portland-cement and latex-modified concrete overlays.* NISTIR 89-4156. Gaithersburg, MD: U.S. Department of Commerce.

Kosmatka, S. H., and W. C. Panarese. 1988. *Design and control of concrete mixtures.* EB001.13T. Skokie, IL: Portland Cement Association.

Lane, D. S. 1993. *Experience with ASTM P 214 in testing Virginia aggregates for alkali-silica reactivity.* Paper no. 930989. Transportation Research Board Annual Meeting, Washington, DC.

Malone, P. G., C. A. Randail, Jr., and T. Kirkpatrick. 1985. *Potential applications of alkali-activated alumino-silicate binders in military operations.* Waterways Experiment Station. Miscellaneous paper no. GL-85-15. Springfield, VA: National Technical Information Service.

Marusin, S. L. 1986. Chloride ion penetration in conventional concrete and concrete containing condensed silica fume. *ACI SP-91 Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete* 2:1119–1133. Detroit, MI: American Concrete Institute.

Mitchell, T. M., P. L. Lee, and G. J. Eggerts. 1979. The CMD: A device for continuous monitoring of the consolidation of plastic concrete. *Public Roads* 42 (4):148–155.

Munn, W. D. 1989. Freeway slab replacements carry traffic the same day. *Highway and Heavy Construction* 132 (11):54–55.

NCHRP. 1990. *Determination of water:cement ratio in fresh concrete.* Research Results Digest No. 174. Washington, DC: National Research Council.

North Dakota State Highway Department. 1978. *Determination of water content of plastic concrete using a microwave oven.* North Dakota State Highway Department, Materials and Research Division. Item (4)-77B. Bismarck, ND: North Dakota Department of Transportation.

Nurse, S. W. 1949. Steam curing of concrete. *Magazine of Concrete Research* 1 (2):79–88.

Ohio Department of Transportation. 1992. *Construction and materials specifications.* Section 499.032. Columbus, OH: Ohio Department of Transportation.

Okamoto, P. A., C. L. Wu, S. M. Tarr, and L. W. Cole. 1993. Early Opening for PCC Pavements. Proceedings, *5th International Conference on Concrete Pavement Design and Rehabilitation.* April 20-22, Purdue University, West Lafayette, Indiana.

Ozyildirim, C. 1993. High-performance concretes for transportation structures. *Concrete International* 15 (1):33–38.

Pearson, R. I. 1988. Fast-Track concrete paving. *Concrete International* 10 (8):33–35.

Pistilli, M. F., and T. Willens. 1993. Evaluation of cylinder size and capping method in compression strength testing of concrete. *Cement, Concrete, and Aggregates* 15 (1).

Salami, M. R. 1992. Analytical expressions for uniaxial tensile strength of concrete in terms of uniaxial compressive strength. *Transportation Research Record* 1335:52–54.

Salsilli, R. A. 1991. Calibrated mechanistic design procedure for jointed plain concrete pavements. Ph.D. diss., University of Illinois, Urbana.

Sansalone, M., and N. J. Carino. 1989. Detecting delaminations in concrete slabs with and without overlays using the impact-echo method. *ACI Materials Journal* 86 (2):175–184.

Saul, A.G.A. 1951. Principles underlying the steam curing of concrete at atmospheric pressure. *Magazine of Concrete Research* 2 (6):127–140.

Shideler, J. J. 1952. Calcium chloride in concrete. *Journal of the American Concrete Institute* 48:537–559.

Shilstone, J. M., Sr. 1988. Interpreting the slump test. *Concrete International Design and Construction* 10 (11):68–69.

SHRP. 1992. *SHRP product catalog*. Washington, DC: Strategic Highway Research Program, National Research Council.

Smith, K. D., A. L. Muller, M. I. Darter, and D. G. Peshkin. 1990. *Performance of jointed concrete pavements: Volume II—Evaluation and modification of concrete pavement design and analysis models.* FHWA-RD-89-137. Springfield, VA: National Technical Information Service.

Snyder, M. B., M. J. Reiter, and K. T. Hall. 1989. *Rehabilitation of concrete pavements: Volume I—Repair rehabilitation techniques*. FHWA-RD-88-071. Springfield, VA: National Technical Information Service.

Sprinkel, M. M. 1988. High early strength latex-modified concrete overlay. *Transportation Research Record* 1204:42–51.

Sprinkel, M. M., and K. K. McGhee. 1989. *Evaluation of concrete pavement patching techniques*. FHWA/VA-89-22. Springfield, VA: National Technical Information Service.

Stark, D. 1991. *Handbook for the identification of alkali-silica reactivity in highway structures.* SHRP-C-315. Washington, DC: Strategic Highway Research Program, National Research Council.

Teng, T.C.P. 1972. *Proper vibration of portland cement concrete pavements.* MSHD-RD-043-01. Washington, DC: Federal Highway Administration.

Tom, J. G., and A. D. Magoun. 1986. *Evaluation of procedures used to measure cement and water contents in fresh concrete.* NCHRP report no. 284. Washington, DC: National Research Council.

Transportation Research Board. 1991. *Highway deicing—Comparing salt and calcium magnesium acetate.* Special Report 235. Washington DC: National Research Council.

Transportation Research Board. 1979. *Joint related distresses in PCC pavement—Causes, prevention, and rehabilitation.* NCHRP synthesis of highway practice no. 56. Washington, DC: National Research Council.

Transportation Research Board. 1977. *Rapid setting materials for patching of concrete.* NCHRP synthesis of highway practice no. 45. Washington, DC: National Research Council.

Weyers, R. 1993. *Concrete bridge protection and rehabilitation: chemical and physical techniques—rapid bridge deck protection, repair, and rehabilitation.* Report no. SHRP-S-344. Washington, DC: Strategic Highway Research Program, National Research Council.

Whiting, D. 1988. Permeability of selected concretes. *Permeability of concrete,* ACI SP-108, 195–222. Detroit, MI: American Concrete Institute.

Whiting, D., et al. 1993. *Synthesis of current and projected concrete highway technology.* SHRP-C-345. Washington, DC: Strategic Highway Research Program, National Research Council.

Whiting, D., and W. Dziedzic. 1989. Chloride permeabilities of rigid concrete bridge deck overlays. *Transportation Research Record* 1234:24–29.

Whiting, D., G. W. Seegebrecht, and S. Tayabji. 1987. Effect of degree of consolidation on some important properties of concrete. *Consolidation of concrete,* ACI SP-96, 125–160. Detroit, MI: American Concrete Institute.

Whiting, D. A., and S. D. Tayabji. 1988. *Relationship of consolidation to performance of concrete pavements.* FHWA/RD-87/095. Springfield, VA: National Technical Information Service.

Appendix A

Bridge Deck and Pavement Repair Evaluations

Bridge Deck Overlays

Research Plan

For the study of bridge deck overlay applications, two important developments were evaluated. These are the use of Type III cement combined with latex modifiers (LMC-III) in order to achieve early (24 hour) opening of a latex-modified concrete overlay, and the use of silica fume concrete (SFC) overlays to achieve extremely low chloride permeability and hence increase the life of the overlay. Two states were chosen for evaluation of these new overlay systems. Within each state two test sites were selected. Each site consisted of twin bridges, so that LMC-III could be placed on one bridge and SFC on the opposing bridge. This would allow for long-term monitoring on an equivalent exposure basis. Thus four bridge sites were included in the experiment, with a total of eight test sections.

The experimental design matrix is shown in Table A.1. It can be seen that for each mixture SFC or LMC-III, a total of four different sets of materials are included. At each individual site, however, aggregate materials remained the same, therefore a comparison of SFC and LMC-III can be made at each site.

Table A.1. Design matrix for bridge deck overlay experiment.

	State 1				**State 2**			
	Site 1		Site 2		Site 3		Site 4	
	Bridge 1	Bridge 2	Bridge 1	Bridge 2	Bridge 1	Bridge 2	Bridge 1	Bridge 2
SFC	A		B		C		D	
LMCIII		A′		B′		C′		D′

Timing of this phase of the project was such that selection of test sites had to be made after most contracts had been let for the 1992 construction season. This reduced the number of sites which would satisfy the test plan, and required close cooperation by the states and contractors approached on existing projects to alter their plans to include both the use of Type III cement and SFC and LMC overlays on the same jobs. After considerable searching, the minimum number of tests sites were located in Ohio and Kentucky.

Site descriptions, materials, and mixes used at each site are included in the following section. Results of field testing and laboratory testing of specimens cast from the field mixes are presented following the site descriptions.

Placements

Site No. 1—US 52 over Twelve Mile Creek, New Richmond, Ohio

This site consists of twin bridges (Bridge No. CLE-52-0497 L&R) located on US 52 over Twelve Mile Creek just west of the town of New Richmond along the Ohio River. Each three-span bridge is 225 ft. (68 m) in length and the decks are supported over continuous structural steel I-beams and crossframes. Length of end spans are 71 ft. (22 m) each, with the center span being 82 ft. (25 m) long. The decks were originally constructed in 1965. The most recent bridge inspection carried out by state forces in 1991 indicated the decks to be in fair condition, with delaminations and patches over about 20% of the surfaces. While these decks were classified as being in "satisfactory" service condition, they were included for overlayment as part of a larger rehabilitation project (ODOT [Ohio Department of Transportation] 253-91) extending the length of US 52 from New Richmond west to the interchange of I-275, a distance of approximately 12 miles (19 km). The deck in the westbound lanes was selected for overlay with silica fume concrete (SFC) and that in the eastbound lanes with latex modified concrete using Type III cement (LMC-III).

Materials selected for the LMC-III mix consisted of: (1) a Type III cement from Southwestern Portland Cement Co.; (2) an angular to rounded natural sand consisting of a mixture of siliceous and calcareous minerals having specific gravity of 2.68, absorption of 1.16%, and FM of 2.67; and (3) a partially crushed limestone coarse aggregate having a maximum topsize of 1/2 in. (12 mm) with specific gravity of 2.63 and absorption of 2.09%. Latex used was DPS Modifier A, a styrene-butadiene latex emulsion produced by Dow Chemical Co.

A Type I cement produced by Lehigh Portland Cement Co. was used in the SFC mix. Aggregates were from the same sources used for the LMC-III mix at this site. Admixtures included: (1) Densified Microsilica (compacted powder form); (2) Amex-210, a benzyl-sulfonate based air-entraining agent; (3) Hy-Kon 2000R, a Type D water-reducer/retarder; and (4) Hy-Con Super, a Type F high-range water reducer. The air-entraining agent and silica fume were produced by Cormix Construction Chemicals and the two water reducers were produced by Hydration Kontrol Co.

Mixes used at site no. 1 are shown in Table A.2. ODOT specifications on fresh properties of LMC-III allow a range of slump from 4 to 6 in. (100 to 150 mm) and an air content not to exceed 7%. Specifications for SFC mixes included a slump range of 4 to 8 in. (100 to 200 mm) and an air content range of 6% to 10%.

Table A.2. Mixes used at US 52 bridge deck overlay site.

Material (cubic yard basis)	LMC-III	SFC
Cement (lb.)	Type III-658	Type I-700
Fine aggregate (lb.)	1,703	1,480
Coarse aggregate (lb.)	1,333	1,297
Water (lb.)	238	285
Latex modifier (gal.)	24.5	—
Silica fume (lb.)	—	70
Water reducer (oz.)	—	14
HRWR (oz.)	—	175
AEA (oz.)	—	28
w/c ratio	0.36	0.37*

*water-to-cement plus silica fume
Note: 1 yd^3 = 0.7645 m^3; 1 lb. = 0.4535 kg.; 1 gal. = 3.78 L; 1 oz. = 29.57 mL

Both overlays were placed on April 23, 1992. LMC-III was placed first on the passing lane of the westbound deck. Delivery was accomplished using 2 mobile concrete mixers. The deck was wetted one hour prior to placement of the overlay. A cement/sand grout was scrubbed into the deck immediately prior to placement. Placement went smoothly, with no apparent problems. The surface was textured with steel tines transverse on 5/8 in. (16 mm) centers, then covered with wet burlap and polyethylene sheeting and continuously soaked for a period of 72 hours. SFC was placed later in the day on the passing lane of the eastbound deck. A cement/sand grout was scrubbed into the deck immediately prior to placement. With the exception of problems in meeting slump requirements in the initial few truckloads of SFC (which were rejected by state inspectors), the placement went well. The same texture as for the LMC-III section was applied, and the overlay was moist cured for 72 hours under soaked burlap. Both sections were opened to traffic on April 29, 1992.

Site No. 2—Interstate 270 over Raymond Run, Columbus, Ohio

This site consists of twin bridges (Bridge No. FR 270-1003L&R) located on I-270 over Raymond Run approximately 0.5 mile (0.8 km) south of the Roberts Road overpass over I-270. These are very short single-span bridges over the small creek at this location, each bridge being only 20 ft. (6 m) long. The 15-in. (381-mm) thick decks are simply supported at the abutments. The decks were originally constructed in 1969 and were covered with a 1/4-in. (6-mm) thick latex emulsified asphaltic concrete overlay. The most recent bridge inspection carried out by state forces in 1991 indicated the deck to be in satisfactory condition, with delaminations and patches over about 10% of the deck surface. There were obvious transverse cracks in the decks. While these decks were classified as being in "satisfactory" service condition, they were included for overlayment as part of a larger rehabilitation project (ODOT 853-91) extending 2.7 miles (4.3 km) along I-270 between mileposts 12 and 15. The project included repaving and widening of the Interstate as well as rehabilitation of other bridges along the route. The northbound lanes of the test structure were selected for overlay with SFC and the southbound lanes with LMC-III.

Materials selected for the LMC-III mix consisted of: (1) a Type III cement from Southwestern Portland Cement Co.; (2) a subrounded to rounded natural sand, consisting of a mixture of siliceous and calcareous minerals having specific gravity of 2.62, absorption of 1.54%, and FM of 3.36; and (3) a crushed limestone coarse aggregate having a maximum topsize of 1/2 in. (12 mm) with specific gravity of 2.62 and absorption of 1.96%. Latex used was Styrofan 1186, a styrene-butadiene latex emulsion produced by BASF Corp.

A Type I cement produced by Holnam, Inc. was used in the SFC mix. Aggregates were from the same sources used for the LMC-III mix at this site. Admixtures included: (1) Sika-Crete silica fume (slurry form); (2) Sika AER, a neutralized Vinsol resin air-entraining agent; (3) Plastocrete 161 MR, a Type D water-reducer/retarder; and 4) Sikament 300, a Type F high-range water reducer. All admixtures were supplied by Sika Corp. Mixes used at site no. 2 are shown in Table A.3.

Table A.3. Mixes used at I-270 bridge deck overlay site.

Material (cubic yard basis)	LMC-III	SFC
Cement (lb.)	Type III-658	Type I-700
Fine aggregate (lb.)	1,591	1,470
Coarse aggregate (lb.)	1,306	1,292
Water (lb.)	321	223
Latex modifier (gal.)	25.0	—
Silica fume (lb.)	—	70
Water reducer (oz.)	—	25
HRWR (oz.)	—	218
AEA (oz.)	—	5
w/c ratio	0.48	0.29*

*water-to-cement plus silica fume
Note: 1 yd^3 = 0.7645 m^3; 1 lb. = 0.4535 kg.; 1 gal. = 3.78 L; 1 oz. = 29.57 mL

ODOT specifications on fresh properties of LMC-III include a range of slump from 4 to 6 in. (100 to 150 mm) and an air content not to exceed 7%. Specifications for SFC mixes included a slump range of 4 to 8 in. (100 to 200 mm) and an air content range of 6% to 10%.

The asphaltic overlay was removed from the existing decks and the surface scarified in preparation for placement. Both overlays were placed on May 27, 1992. SFC was placed first on the passing lane of the northbound deck. Delivery was accomplished with two 5 yd^3 (3.8 m^3) batches. The deck was wetted one hour prior to placement of the overlay. Placement went smoothly, with no apparent problems. There was quite a bit of hand finishing work, required, however, as this was a small deck and it was difficult to reach all areas with the finishing machine. The surface was textured with steel tines transverse on 5/8 in. (16 mm) centers, then covered with wet burlap and polyethylene sheeting and continuously soaked for a period of 48 hours. LMC-III was placed in the afternoon of the same day on the passing lane of the southbound deck. Although the placement appeared to go smoothly, the state inspectors questioned the calibration of the mobile mixer during the pour. Later calculations by state inspectors (which were forwarded to the investigators) indicated that higher amounts of water than desired had been discharged, leading to an abnormally high w/c ratio (0.48) in the mix. The same texture as for the SFC section was applied, and the overlay was moist cured for 48 hours under soaked burlap. Both sections were opened to traffic on June 19, 1992.

Site No. 3—Interstate I-265 over KY 22, Jefferson County, Kentucky

This site consists of twin bridges (Bridges No. B00087 and B00087P) located on I-265 over Kentucky State Highway 22, approximately 7 miles (11 km) northeast of the Louisville city limits on KY 22. Both structures are three-span bridges supported on steel I-beams and are 243 ft. (74 m) in length. The end spans are each 49.5 ft (15 m) long, with the center span being 137 ft. (42 m) long. The decks were originally constructed in 1969. The most recent bridge inspection carried out by state forces in 1991 indicated the deck to be in satisfactory condition, with a few transverse cracks and isolated spalled areas which had been patched with asphaltic material. As it appeared likely that continual patching would need to be carried out to maintain the riding surface, and there was a need to replace deteriorated joints, the deck was scheduled for rigid concrete overlay. The northbound lanes of the test structure were selected for overlay with LMC-III and the southbound lanes with SFC.

Materials selected for the LMC-III mix consisted of: (1) a Type III cement from Essroc Materials, Inc.; (2) an subangular to rounded natural sand consisting of a mixture of siliceous and calcareous minerals having specific gravity of 2.60, absorption of 1.40%, and FM of 2.56; and (3) a crushed limestone coarse aggregate having a maximum topsize of 1/2 in. (12 mm) with specific gravity of 2.71 and absorption of 0.6%. Latex used was DPS Modifier A, a styrene-butadiene latex emulsion produced by Dow Chemical Co.

A Type I cement produced by Lehigh Portland Cement Co. was used in the SFC mix. Aggregates were from the same sources used for the LMC-III mix at this site. Admixtures included: (1) Force 10,000 silica fume (slurry form); (2) Darex II AEA an organic-acid salt based air-entraining agent; and (3) WRDA-19, a Type F high-range water reducer. All admixtures were supplied by W.R. Grace & Co.

Mixes used for the overlays at this site are shown in Table A.4. Kentucky Department of Highways (KYDOH) specifications on fresh properties of LMC-III include a range of slump from 4 to 6 in. (100 to 150 mm) and an air content not to exceed 7%. Specifications for SFC mixes included a slump range of 4 to 7 in. (100 to 175 mm) and an air content range of 5% to 8%.

Table A.4. Mixes used at I-265 bridge deck overlay site.

Material (cubic yard basis)	LMC-III	SFC
Cement (lb.)	Type III-658	Type I-658
Fine aggregate (lb.)	1,708	1,395
Coarse aggregate (lb.)	1,162	1,531
Water (lb.)	219	240
Latex modifier (gal.)	24.5	—
Silica fume (lb.)	—	50
HRWR (oz.)	—	105
AEA (oz.)	—	39
w/c ratio	0.33	0.34*

*water-to-cement plus silica fume

Note: 1 yd^3 = 0.7645 m^3; 1 lb. = 0.4535 kg.; 1 gal. = 3.78 L; 1 oz. = 29.57 mL

At this site the contractor preferred to finish work on one deck before proceeding with the twin structure. Therefore, the LMC-III was placed first on the rightmost driving lane of the northbound deck on June 8, 1992. The placement was at night, as required by state specifications in order to avoid exposure of the overlay to hot conditions prevalent during the summer months. Concrete was delivered to the site by two mobile concrete mixers. While there were no serious problems in placement, it was noted that water used to pre-wet the deck was excessive, and was not removed from some of the deeply excavated areas prior to placing the latex concrete. The surface was textured using a stiff broom finish; no tinting was carried out. The surface was covered with wet burlap and polyethylene film immediately behind the paving operation. However, no soaker hoses were employed. The burlap and poly sheeting were removed after 24 hours and the overlay allowed to air cure. The LMC-III overlay was opened to traffic on June 10, 1992. The placement of the SFC was carried out on June 29, 1992, on the wide 15 ft. (4.6 m) righthand shoulder of the southbound deck. This placement was also carried out at night. Concrete was delivered via ready mix trucks and the operation went smoothly with no obvious problems or signs of poor workmanship. The surface was finished and cured in a manner similar to the northbound deck, with the exception of the moist curing was carried out for a period of 72 hours. Although the specifications required that the burlap be kept continuously wet during this period, soaker hoses were not employed. Rather, the burlap was wetted down at the start of each day. This may have resulted in less than saturated conditions being maintained at the surface of the overlay throughout the curing period. The section was opened to traffic July 12, 1992.

Site No. 4—US 41 over KY 351, Henderson, Kentucky

This site consists of twin bridges (Bridge Nos. B00050 and B00050P) located on US 41 over Kentucky State Highway 351 at milepost 14 on the Henderson city bypass route. Both structures are three-span bridges of reinforced concrete composite deck/girder construction and are 159 ft. (48 m) long. Spans are each 53 ft. (16 m) long. The decks were originally constructed in 1959. A low slump concrete overlay had been placed on the deck in the mid-1970s (exact date of placement of the overlay was not available). The most recent bridge inspection carried out by state forces in 1992 indicated that the overlay was mapcracked and exhibiting considerable wear. The surface had been patched numerous times, and the patches were beginning to deteriorate. Because of this deterioration of the riding surface, the deck was scheduled for rigid concrete overlay. The northbound lanes of the test structure were selected for overlay with LMC-III and the southbound lanes with SFC.

Materials selected for the LMC-III mix consisted of: (1) a Type III cement from Lehigh Portland Cement Co.; (2) an angular to rounded siliceous natural sand having specific gravity of 2.61 and absorption of 1.00%; and (3) a crushed limestone coarse aggregate having a maximum topsize of 1/2 in. (12 mm) with specific gravity of 2.71 and absorption of 0.7%. Latex used was DPS Modifier A, a styrene-butadiene latex emulsion produced by Dow Chemical Co.

A Type I cement produced by Lehigh Portland Cement Co. was used in the SFC mix. Aggregates were from the same sources used for the LMC-III mix at this site. Admixtures included: (1) MB-SF silica fume (compacted powder form); (2) Micro-Air, a synthetic air-entraining agent; and (3) Rheobuild-1000, a Type F high-range water reducer. All admixtures were supplied by Master Builders, Inc.

Mixes used for the rigid concrete overlays at the US 41 decks are presented in Table A.5.

Table A.5. Mixes used at US 41 bridge deck overlay site.

Material (cubic yard basis)	LMC-III	SFC
Cement (lb.)	Type III-658	Type I-658
Fine aggregate (lb.)	1,610	1,366
Coarse aggregate (lb.)	1,260	1,536
Water (lb.)	260	236
Latex modifier (gal.)	24.5	—
Silica fume (lb.)	—	50
HRWR (oz.)	—	98
AEA (oz.)	—	8
w/c ratio	0.39	0.33*

*water-to-cement plus silica fume
Note: 1 yd^3 = 0.7645 m^3; 1 lb. = 0.4535 kg; 1 gal = 3.78 L; 1 oz. = 29.57 mL

At site no. 4 the contractor preferred to finish work on one deck before proceeding with the twin structure. Therefore, the SFC was placed first on the driving lane of the southbound deck on July 8, 1992. The placement was at night, as required by state specifications. Concrete was delivered to the site by ready mix trucks. While there were no serious problems in placement, it was noted that the bonding grout was applied unevenly, and in some cases grout was not applied and mortar was broomed out of the concrete instead. The surface was textured using a stiff broom finish; no tinting was carried out. The surface was covered with wet burlap and polyethylene film immediately behind the paving operation. Soaker hoses were used to maintain the deck in a saturated condition for the full 72-hour curing period. The SFC overlay was opened to traffic on July 20, 1992. The placement of the LMC-III was delayed until August 13, 1992. The placement of the test concrete was located on the passing lane of the northbound structure. This placement was also carried out at night. Concrete was delivered using mobile mixers. Similarly to the southbound deck, there was evidence of non-uniform application of the mortar broomed from the LMC. The surface was finished and cured in a manner similar to the northbound deck, with the exception that the moist curing was carried out for a period of 24 hours. The section was opened to traffic on August 18, 1992.

Field Testing of Bridge Deck Overlays

In-place testing was carried out to evaluate materials properties of fresh and hardened overlay concrete and to monitor strength gain at early ages in the LMC-III overlays. Slump, air content, and unit weight were determined using standard techniques. Fresh concrete data are summarized in Table A.6. Descriptions of additional testing and monitoring procedures, and results obtained, are presented in subsequent sections.

Table A.6. Fresh concrete data determined at bridge deck overlay placements.

Site No.	Mix	Date	Time (hr)	Weather	Slump (in.)	Air Content (%)	Initial Temp. (°F)	Unit Weight (lb/ft^3)
1. US 52	LMC-III	4/23/92	1145	Clear	5.5	5.2	72	145.4
	SFC	4/23/92	1940	Clear	6.5	9.5	74	139.4
2. I-270	LMC-III	5/27/92	1430	Clear	5.8	5.7	72	137.9
	SFC	5/27/92	0800	Ptly.cldy	7.0	9.4	70	138.8
3. I-265	LMC-III	6/8/92	1950	Clear	5.0	5.5	79	144.0
	SFC	6/29/92	2000	Clear	6.0	6.5	82	143.8
4. US 41	LMC-III	8/13/92	1935	Clear	6.5	6.0	77	138.0
	SFC	7/8/92	2115	Clear	3.8	5.6	75	140.8

Note: 1 in. = 25.4 mm; °F = 1.8 × °C + 32; 1 lb/ft^3 = 16.02 kg/m^3

Results for the Ohio sites (1 and 2) were within specification for slump and air content. While the slump and air content at the I-265 site in Kentucky were within specifications, slump values at the US 41 site exceeded specification for LMC-III and fell below the lower limit for SFC. The concrete, however, was accepted for placement. As can be seen from the weather data, conditions at all sites were relatively temperate. The only extreme conditions encountered were at site no. 2 (I-270) where overnight temperatures fell into the 40°F (4°C) range.

Determination of Water Content via Microwave Drying

As part of project C-206, a technique was evaluated for determination of the water content of fresh concrete using a microwave oven. The background, development, and procedures for the method are described in Chapter 3 of this report. For two of the overlay placement sites (site nos. 1 and 3) the technique was used to determine water content of LMC-III and SFC mixes. The technique could not be applied at all of the sites as laboratory verification work was still being carried out and the oven was not available at all times. Measurements were carried out in field trailers as power was readily available. Later field trials carried out at pavement test sites (see subsequent sections) showed that the oven could be operated using a portable 3000-watt generator set. A battery powered 5000-gm portable balance was used to determine mass loss. Results are presented in Table A.7. At site no. 3, replicate batches were tested. Results were varied. In some cases, measured water contents were very close to reported values from batch tickets. For others there were significant deviances, mainly for the latex mixes produced using mobile concrete mixers. Mobile concrete production is subject to greater errors than weigh batching, which may help to explain some of the discrepancies. In addition, during production of latex overlays contractors many times adjust the settings on the mobile mixer in order to improve workability of the concrete.

Table A.7. Field water contents determined using microwave drying.

Site No.	Mix Type	Reported Total Water (lb/yd^3)	Measured Total Water (lb/yd^3)	Difference (% of reported)
1. US 52	SFC	321	316	-1.55
	LMC-III	285	273	-4.20
3. I-265	LMC-III	219	233	+6.39
	LMC-III	218	246	+12.84
	SFC	263	253	-3.80
	SFC	263	265	+0.80

Note: 1 yd^3 = 0.7645 m^3

Strength Gain Estimation Using Maturity Method

Strength of concrete is the result of a chemical reaction (hydration) between cement and water. Since rate of hydration depends on temperature, the strength of concrete may be evaluated from a concept of maturity which is expressed as a function of the time and the temperature of curing (Carino et.al. 1983). By measuring the temperature in the concrete in a given time after casting, maturity can be calculated and compressive strength estimated if a preestablished relationship between maturity and compressive strength for a given mixture exists.

Maturity is most often calculated by the formulae first developed by Nurse (1949) and Saul (1951):

$$M(t)=\Sigma(T-T_o)\Delta t \qquad \textbf{(1)}$$

where M(t) = maturity at age t, degree-days or degree-hours
Δt = a time interval, days or hours
T = average concrete temperature during time interval, Δt, °C and
T_o = datum temperature, °C

Datum temperature is that temperature at which concrete ceases to gain strength (hydration stops) with time. This temperature reportedly ranges from 14 to 32°F (-10.6 to 0°C), but may be higher depending on the specific materials used. This formula is based on the assumption that maturity increases linearly with temperature. However, it is known from chemical reaction kinetics that the rate of chemical processes increase with temperature, not linearly, but exponentially according to the Arrhenius equation:

$$K = A \cdot \exp(-\frac{E}{RT}) \qquad (2)$$

where

K = rate constant (1/time)
A = constant (1/time)
E = activation energy
R = gas constant
T = temperature (°K).

Based on this equation, the variation in maturity or the "equivalent age" at specified temperature can be computed (Hansen and Pederson 1977):

$$t_e = \sum \exp Q(\frac{1}{T_a} - \frac{1}{T_s})\Delta t \qquad (3)$$

where

t_e = equivalent age at a specified temperature T_s, days or hours
Q = activation energy divided by the gas constant, R
T_a = average temperature of concrete during time interval Δt, °K
T_s = specified temperature, °K
Δt = time interval, days or hours

Equations 1 and 3 are considered in ASTM C 1074, "Standard Practice for Estimating Concrete Strength by the Maturity Method". In order to estimate the in-place strength of concrete in highway or other structures based on maturity concepts, the concrete temperature should be continuously monitored and the in-place maturity then computed using either the temperature-time factor (eqn. 1) or equivalent age (eqn. 3). Temperature monitoring starts as soon as practicable after concrete placement, and according to ASTM C 1074, the recording time interval shall be 1/2 hr or less for the first 48 hrs and 1 hr or less thereafter. A thermocouple was inserted at mid-depth of the overlays, a second in a 4 × 8 in. (102 × 204 mm) test cylinder placed adjacent to the overlay, and a third was maintained in the air away from the edges of the overlays.

Temperature profiles for all four sites for the LMC-III overlays are shown in Figures A.1 through A.4. Shown are temperatures measured in the air, at mid-depth of the overlay, and in the field cylinder placed under the burlap curing mat. For the early spring placements, the decrease in temperature of air and overlay for the first 15 hours represents the cool nights experienced at these sites. Temperatures then increase the following day. For site no. 3 (I-265) the profiles are somewhat more complex, the overlay holding virtually constant

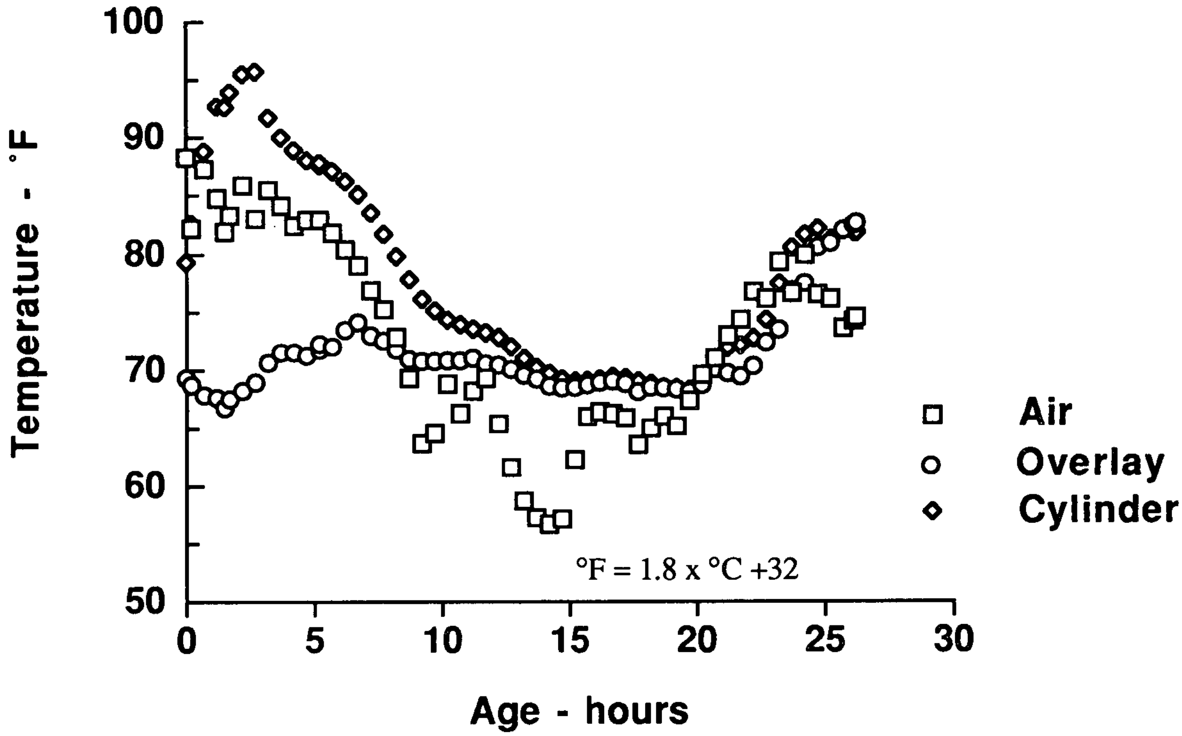

Figure A.1. Temperature profiles for LMC-III US 52 overlay.

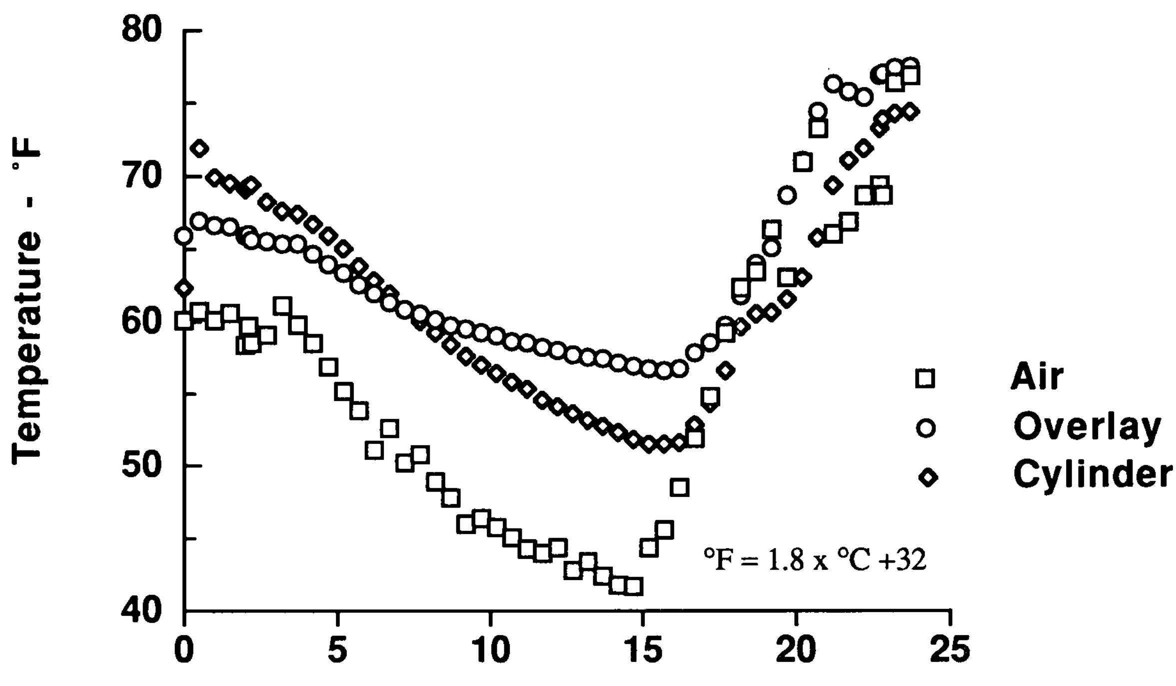

Figure A.2. Temperature profiles for LMC-III I-270 overlay.

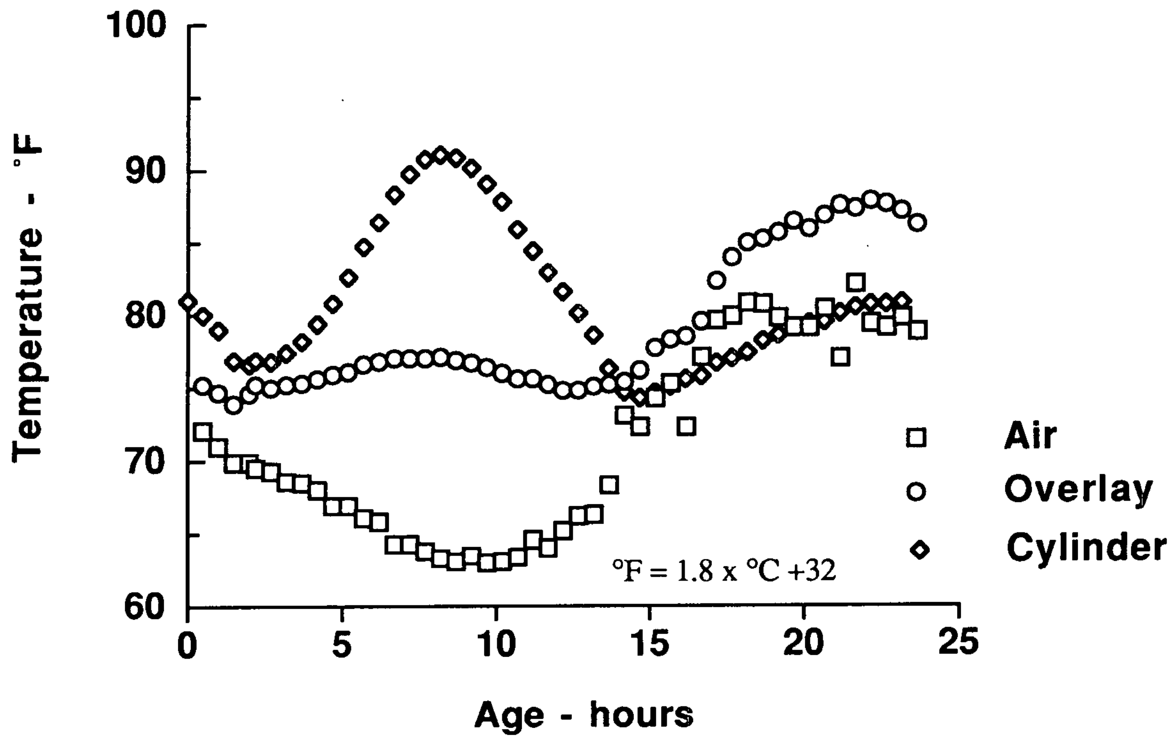

Figure A.3. Temperature profiles for LMC-III I-265 overlay.

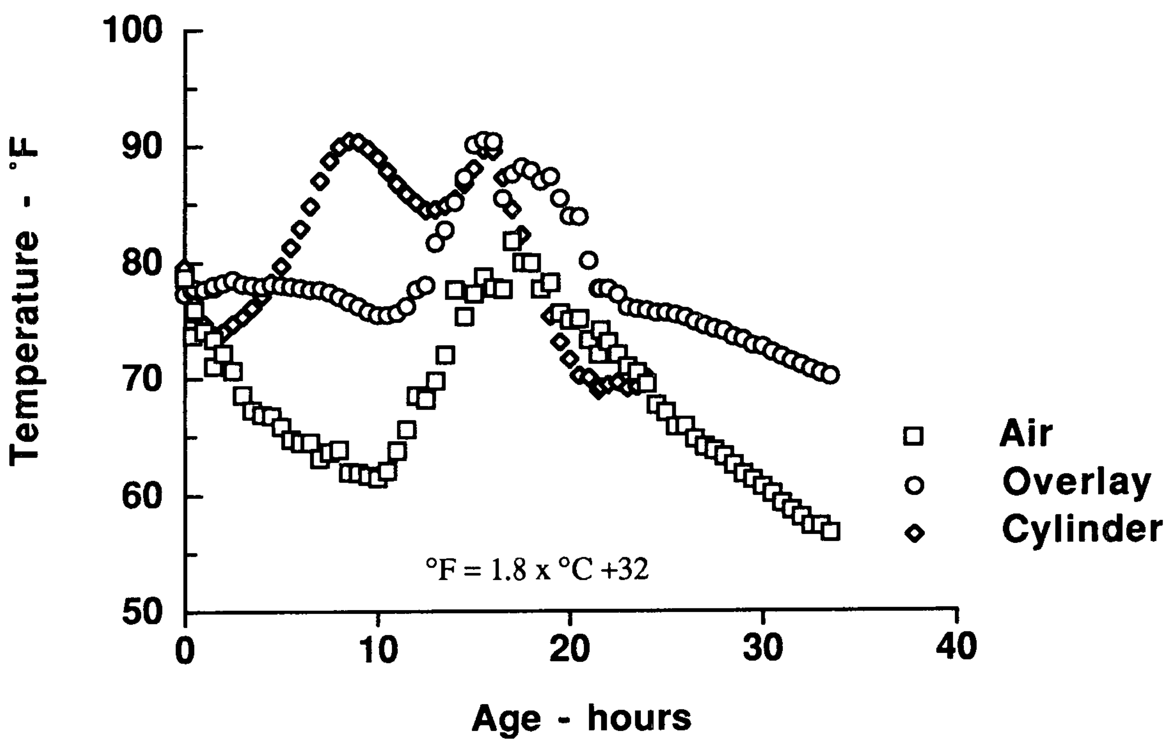

Figure A.4. Temperature profiles for LMC-III US 41 overlay.

temperature overnight reflecting the relatively warm night at this location. The cylinder exhibited a heat peak, most likely due to the insulating nature of the plastic mold. At the US 41 overlay, where the highest initial air temperatures were encountered, the heat generated by hydration of cement could not dissipate as quickly into the surroundings, and was reflected in a rise in overlay temperature. At site no. 3, the cylinder exhibited heat peaks at 9 and 16 hours after placement.

The maturity method was applied to the four LMC-III overlays, as these were designed for early opening at 24 hours. In accordance with C 1074, samples of cement, sand, and latex modifier were returned to the laboratories and used to produce mortar cube specimens from which the datum temperature (for the Nurse-Saul approach) and activation energy (for the Arrhenius approach) were determined. Field cylinders companion to the instrumented cylinder were tested in the field at various time intervals and used to establish the maturity/strength relationship for each overlay. Finally, time/strength curves over the full 24 hour curing period were generated using these relationships. It should be noted that at sites 1 (US 52) and 4 (US 41) the full method using mortar cube preparation was not used. Instead, the datum temperature was assumed as 32°F (0°C) (based on results obtained at sites 2 and 3) and the time/strength relationships then generated.

Maturity/strength relationships calculated via the Nurse-Saul and Arrhenius approaches are shown in Table A.8. By transformation of variables very good linear fits were obtained in all cases. The equations shown in Table A.8 were then used to generate plots of strength vs. time using maturities calculated from the temperatures measured in the overlays at each location. These relationships are shown in Figures A.5 through A.8. While full time/strength curves are shown, it must be realized that the equations used to generate these curves were based on strengths taken not less than 8 hours after casting. Therefore, early age strengths, especially those before 5 hours, represent an extrapolation outside of the region used to develop the equations and may not represent the true strength development at early ages for the overlays. In each figure, the 24-hour strength as determined on the field cylinder left under the burlap mat is also shown. For site no. 3 (I-265) both maturity relationships exactly match the strength of the field cylinder. For the other sites predicted overlay strengths are 15% to 20% below strengths measured on the field cylinder for the Nurse-Saul predictor. Site no. 2 is 9% below the field cylinder for the Arrhenius predictor. The difference between predicted strength of the overlay and measured strength of the cylinders should not be taken as a measure of the ability of maturity to predict actual strength, as one might expect field cylinder strength to differ from actual in-place strength. No closer comparisons could be made, however, as it was not possible to obtain a core of sufficient length for compressive test purposes from such thin overlays.

Tensile Bond Testing

Bond of overlay to substrate is an extremely important consideration in placement of overlays, as any deficiencies in bond may lead to later delamination or punchout of the overlay. This is especially important where overlays are to be opened early to traffic, such as for the LMC-III overlays. Bond pull-off was measured in the field after placement and curing of the LMC-III overlays only, the SFC overlays not being scheduled for early opening. The tines

Table A.8. Developed maturity/strength relationships for LMC-III overlay mixes.

Site No.	Type	Datum/ Activation Temperature	No. Points	Equation	Correlation Coefficient
1. US 52	Nurse-Saul	Assume 32°F	6	$f'c = -19.86 \times 10^5/MAT + 4,443$	0.95
2. I-270	Nurse-Saul	32°F	20	$\log f'c = -552/MAT + 3.82$	0.98
	Arrhenius	9300°K	20	$1/f'c = 0.0342/MAT - 1.4 \times 10^{-4}$	0.99
3. I-265	Nurse-Saul	32°F	13	$1/f'c = -2.27 \times 10^{-4} \log MAT + 8.97 \times 10^{-4}$	0.97
	Arrhenius	3600°K	13	$1/f'c = -2.27 \times 10^{-4} \log MAT + 5.93 \times 10^{-4}$	0.97
4. US 41	Nurse-Saul	Assume 32°F	8	$\sqrt{f'c} = -2.72 \times 10^4/MAT + 81.2$	0.99

Note: $°F = 1.8 \times °C + 32$

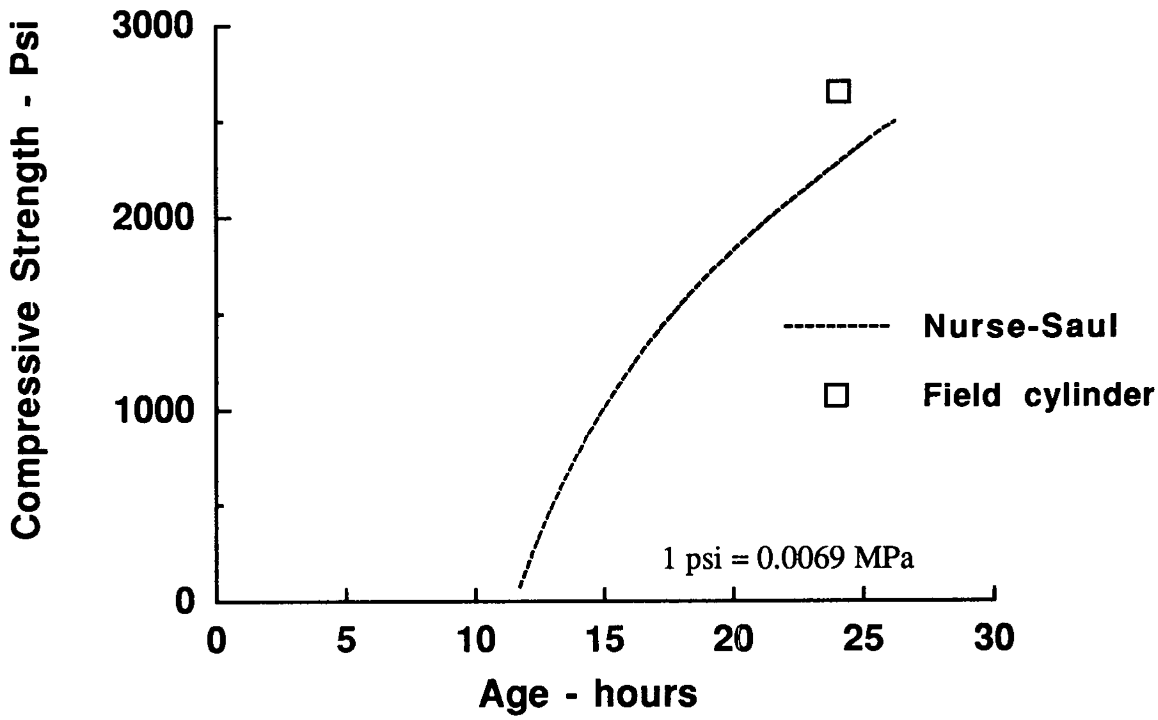

Figure A.5. Prediction of overlay strength on US 52 LMC-III overlay.

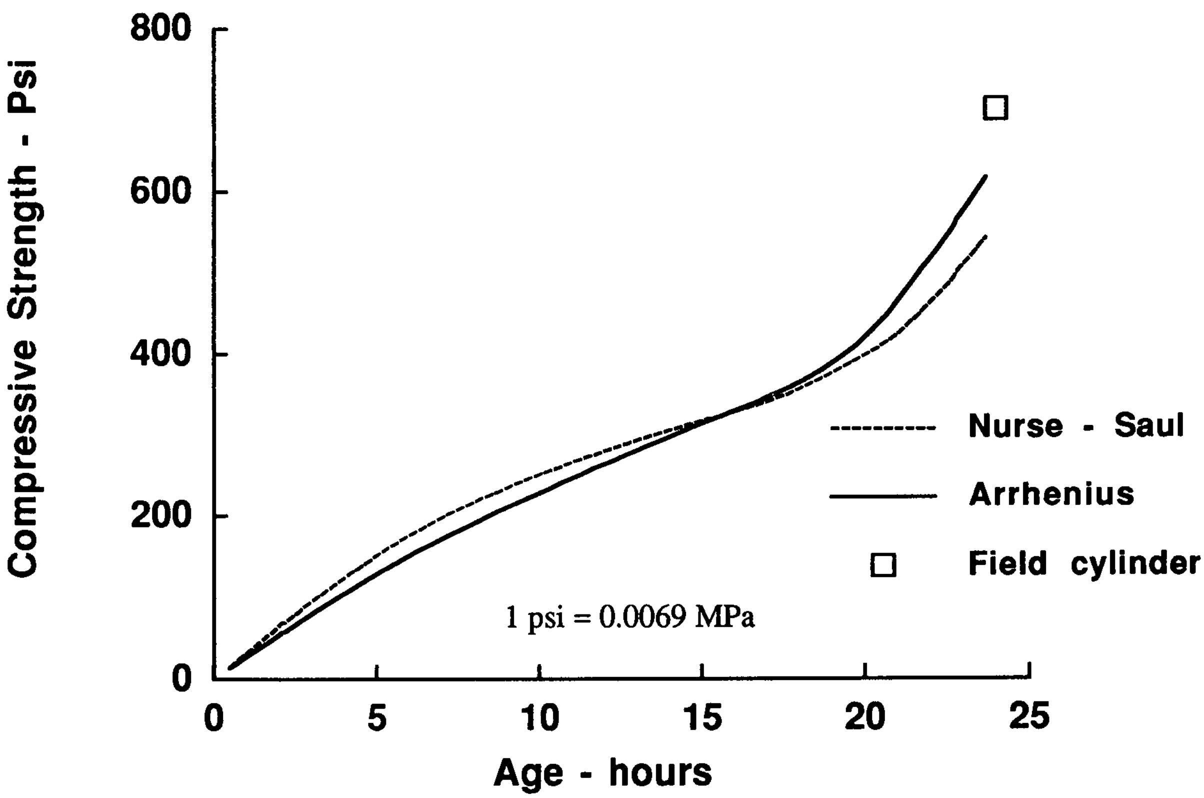

Figure A.6. Prediction of overlay strength on I-270 LMC-III overlay.

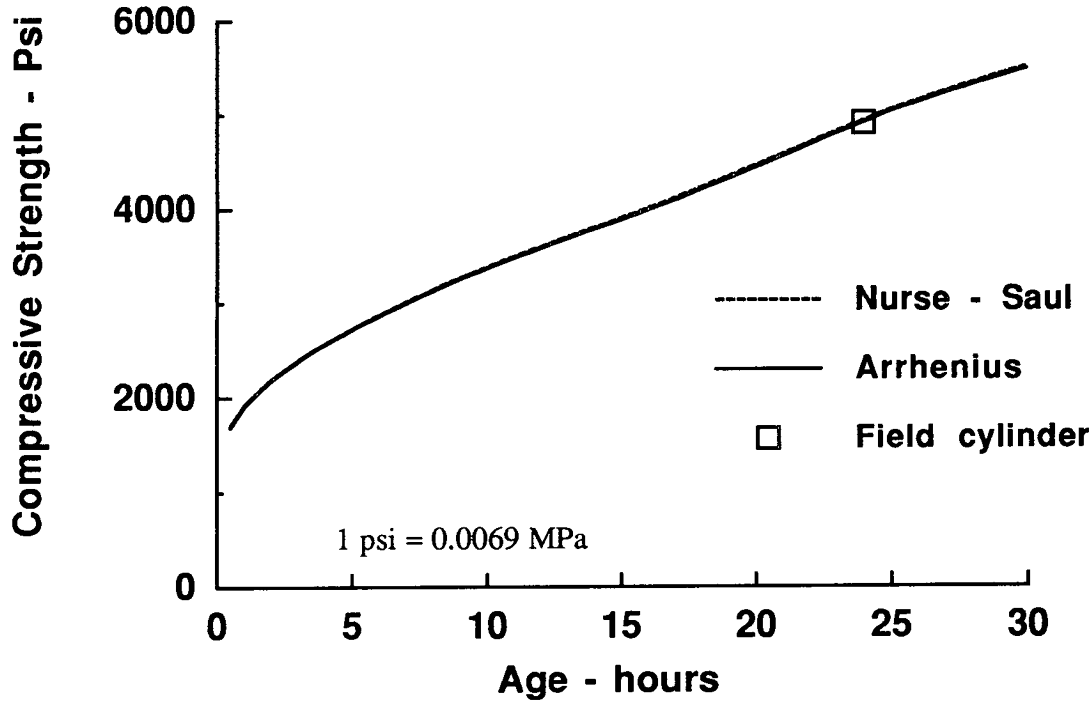

Figure A.7. Prediction of overlay strength on I-265 LMC-III overlay.

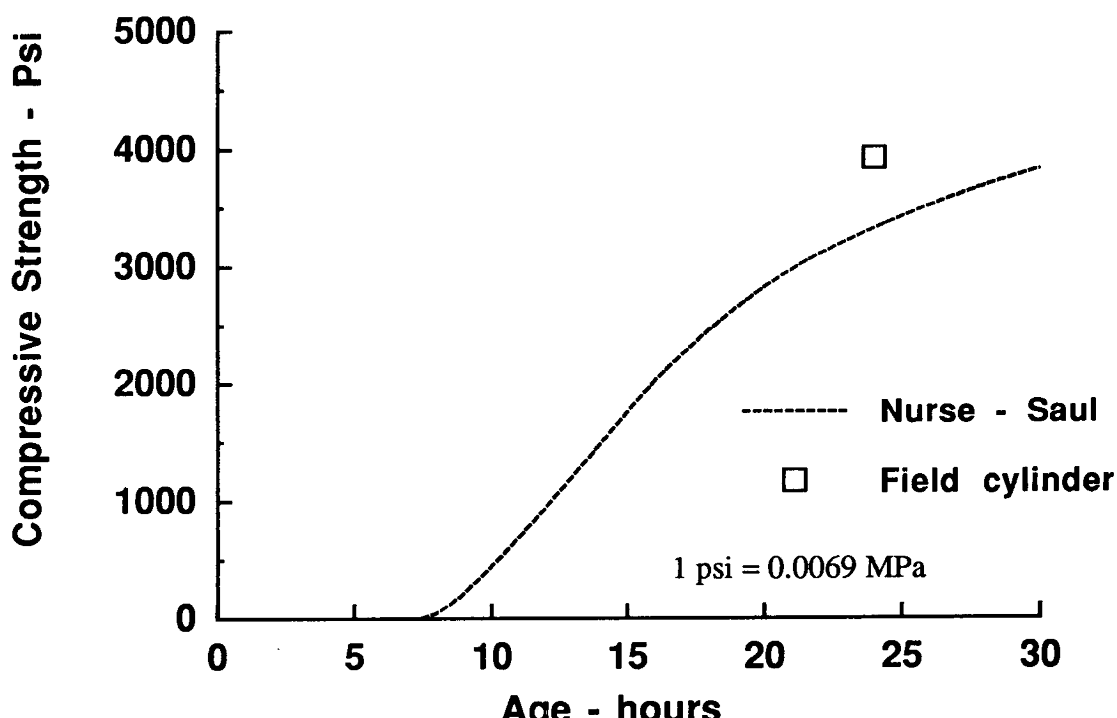

Figure A.8. Prediction of overlay strength on US 41 LMC-III overlay.

and rough broom finish on the overlay surface at the test areas were first removed with a power grinder. The in-place test specimen was fashioned by partial depth coring through the overlay into the existing slab using a 2 in. (50 mm) diameter water-cooled diamond core drill. A 2 in. (50mm) steel puck was then affixed to the top of the in-place core using rapid setting epoxy adhesive. A reaction frame and calibrated load cell were then used to measure direct tensile strength of the overlay bond. The test method is similar to the field test for surface soundness and adhesion described in ACI 503R-89 "Use of Epoxy Compounds with Concrete (Appendix A Test Methods)".

Bond pull-off was measured for the LMC-III overlays at site nos. 1 (US 52), 3 (I-265) and 4 (US 41). Tests were not carried out at site no. 2 due to the failure of the overlay to reach its intended 24 hour strength. This was attributed to the high water content in the mix. Results of tensile bond tests are summarized in Table A.9. While no criteria have been established for this particular test, recent laboratory work (Knab et.al. 1989) carried out on bond strength of latex-modified concretes to concrete substrates indicates a minimum strength of 200 psi (1.4 MPa) using a direct shear "guillotine" type bond test method to be a reasonable performance criterion. The cited report also indicates that, for the same concretes, uniaxial test methods tend to yield lower bond strength values and that bond strengths measured in the field would also tend to be lower than those measured under controlled laboratory conditions. For these reasons the investigators believe that a minimum bond pull-off strength of 100 psi (0.7 MPa) after only 24 hours of field cure indicates a concrete which has the potential for achieving excellent long-term bond strength. Cores obtained from the overlay and tested in the lab using a direct shear technique (see subsequent section) support this conclusion. More development work, needs to be done, however, before such a method could be used for acceptance purposes.

Table A.9. Tensile bond strength at 24 hours for LMC-III overlays.

Site No.	Test No. 1 (psi)	Test No. 2 (psi)	Test No. 3 (psi)	Average (psi)
1. US 52	95	100	130	108
3. I-265	105	95	*	100
3. I-265 (rep.)	160	150	*	155
4. US 41	85	165	155	135

Note: 1 psi = 0.0069 MPa
*only two tests carried out at this location

Laboratory Testing of Overlay Specimens Cast in the Field

Laboratory testing was carried out to develop materials performance data on materials used at each field test site. Specimens were prepared in the field and cast from mixes used to place the bridge deck overlay. Specimens were cured as indicated in the following descriptions and transported back to the laboratory for further curing and testing.

Compressive Strength Testing

Concrete was cast in rigid 4 × 8 in. (100 × 200 mm) plastic cylinder molds at the job sites and consolidated by rodding. Cylinders for standard curing conditions were fabricated and cured at a temperature of 60 to 80°F (15.5 to 26.6°C). For the most part, ambient temperatures at all overlay sites were within this range during the time of placement. Temperature conditions were maintained by transporting specimens in a damp sand bed after final set. Sand cushioned cylinders were transported back to the laboratory and stored in a fog room at 73 ±3°F (23 ±1.7°C) until time of test. Cylinders were kept in their plastic molds with lids taped on to minimize moisture loss during transport. Tests were conducted at 3, 7, 28, and 90 days of age. Additional standard cured cylinders were tested at one day for the early opening LMC-III mixes. Duplicate cylinders were tested at each age. Replicate sets of cylinders were fabricated, transported, cured, and tested at 28 days to evaluate batch-to-batch differences between ready mix trucks (condensed silica fume concrete) and mobile mixers (latex modified concrete). Compressive strength testing was done in accordance with AASHTO T 22. A portable compression testing machine was used to test cylinders on-site at an age of one day.

A second set of specimens was stored as near to the point of deposit of the concrete at the site as possible and was left undisturbed until transport, with the top surface treated identically to the placed concrete. These cylinders were tested at nominal opening times (one day for LMC-III and 3 days for SFC).

Compressive strength test data through 90 days age are summarized in Figure A.9 for LMC-III mixes and in Figure A.10 for SFC. All strengths appear more than adequate, as compressive strength is not a critical determinant of overlay performance. More important are bond strength and permeability to chloride ions (see following discussions). At any age, the lowest of the SFC strengths is almost equivalent to the highest of the LMC-III strengths. Ninety-day strengths for the I-270 SFC mix approach 12,000 psi (82.8 MPa). Field cylinders tested lower than standard-cured cylinders for LMC-III one day tests at sites 1 and 2, which were placed earlier in the year under cool conditions. Field strengths for the SFC mixes at 3 days and for the site 3 and site 4 LMC-III mixes (placed in the warmer summer months) were essentially equivalent to standard cure values. Strengths of cylinders cast from replicate batches were within 5% of those cast from initial batches at 28 days age.

Split Tensile Strength Testing

Duplicate cylinders for split tensile strength testing were cast from the same concrete batches as those for compressive strength cylinders. Standard-cured specimens were fabricated, transported, and cured identically to the compressive strength specimens. Split tensile strength was tested at the same intervals as for compressive strength in accordance with AASHTO T 198. A portable compression testing machine was used to test LMC-III cylinders on-site at an age of one day. Splitting tensile strength test data is summarized in Figure A.11 for LMC-III and Figure A.12 for SFC concrete. The compressive and tensile strengths of

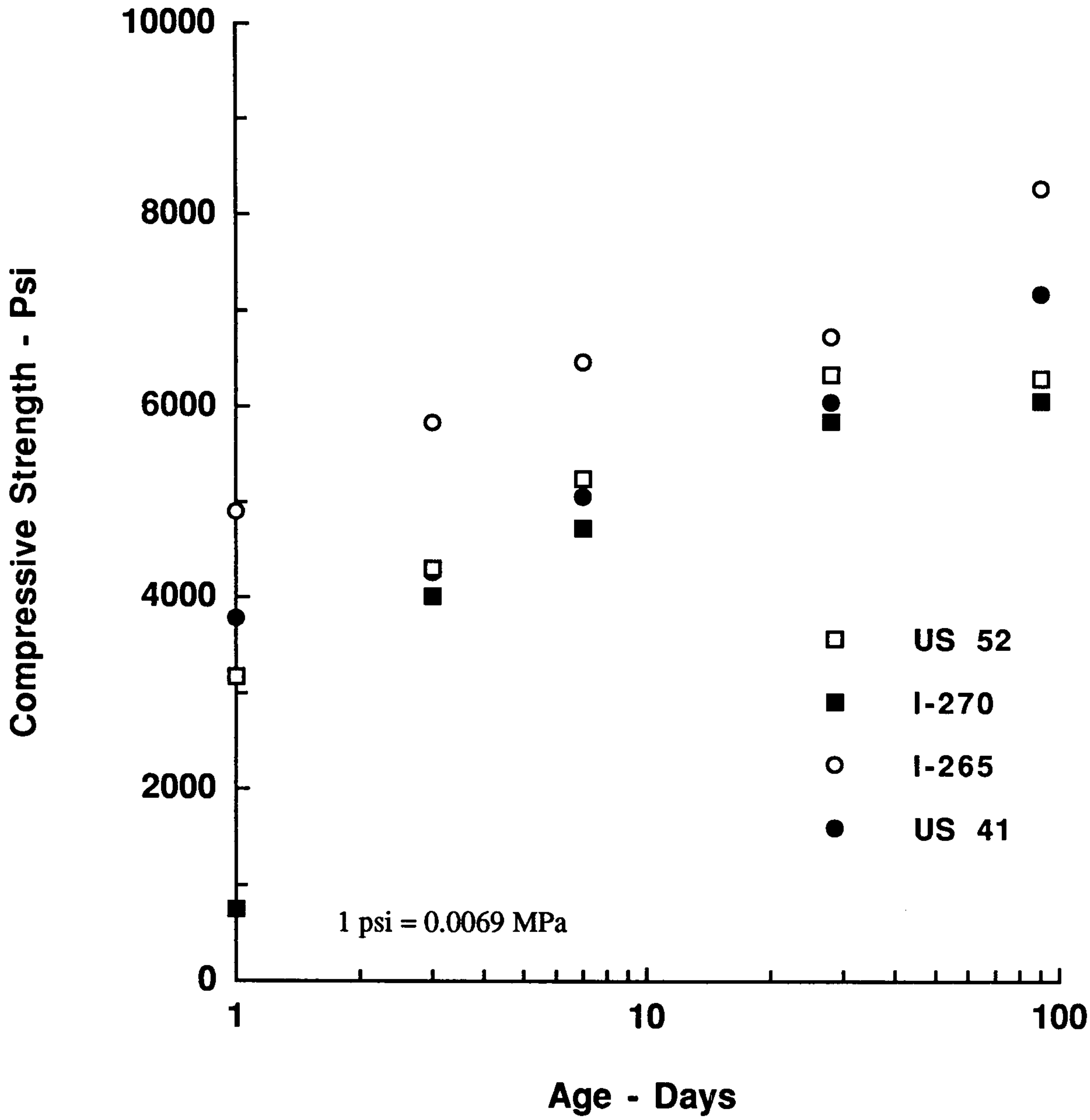

Figure A.9. Compressive strength development of cylinders cast from LMC-III overlay mixes (standard curing).

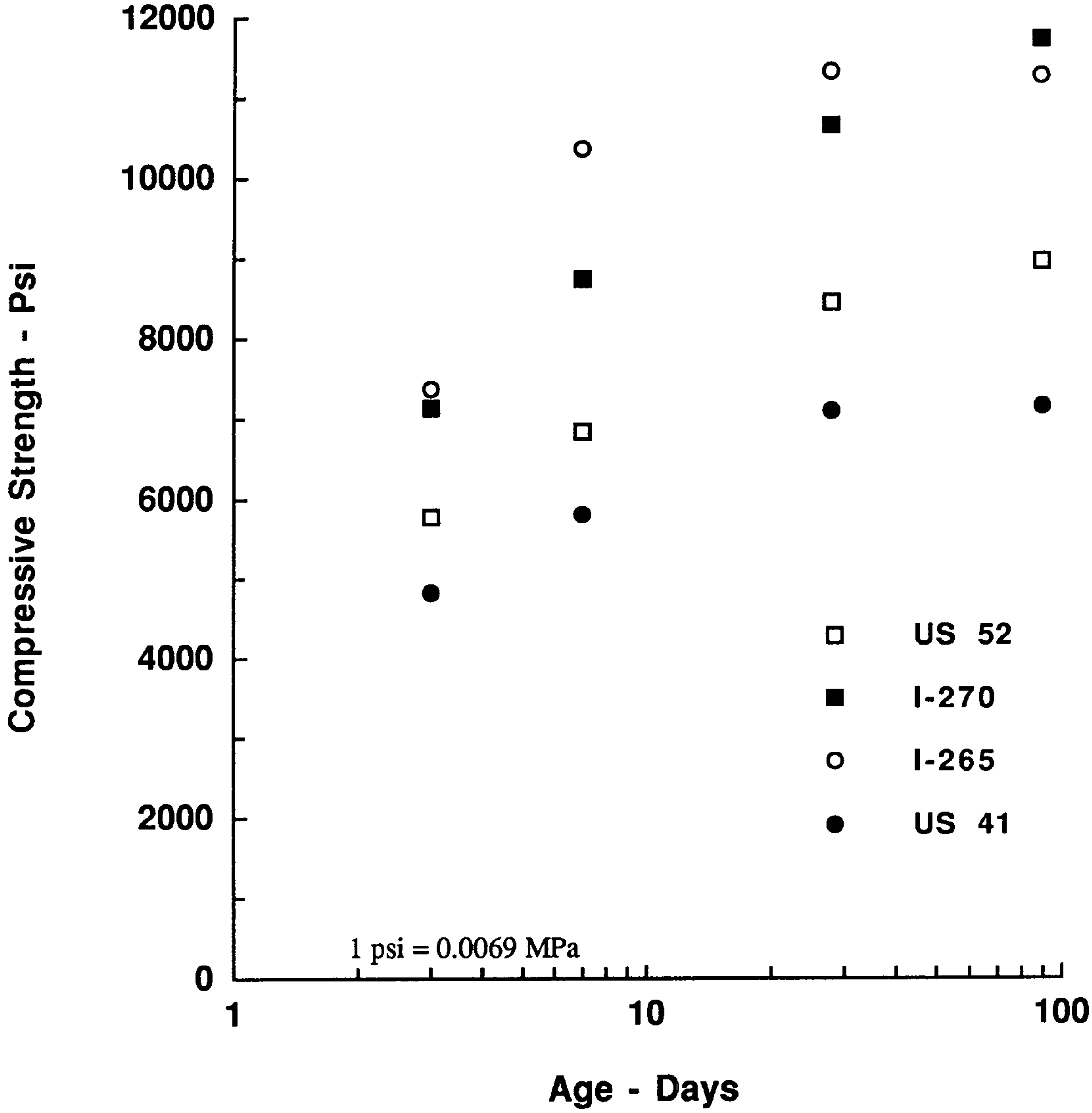

Figure A.10. Compressive strength development of cylinders cast from SFC overlay mixes (standard curing).

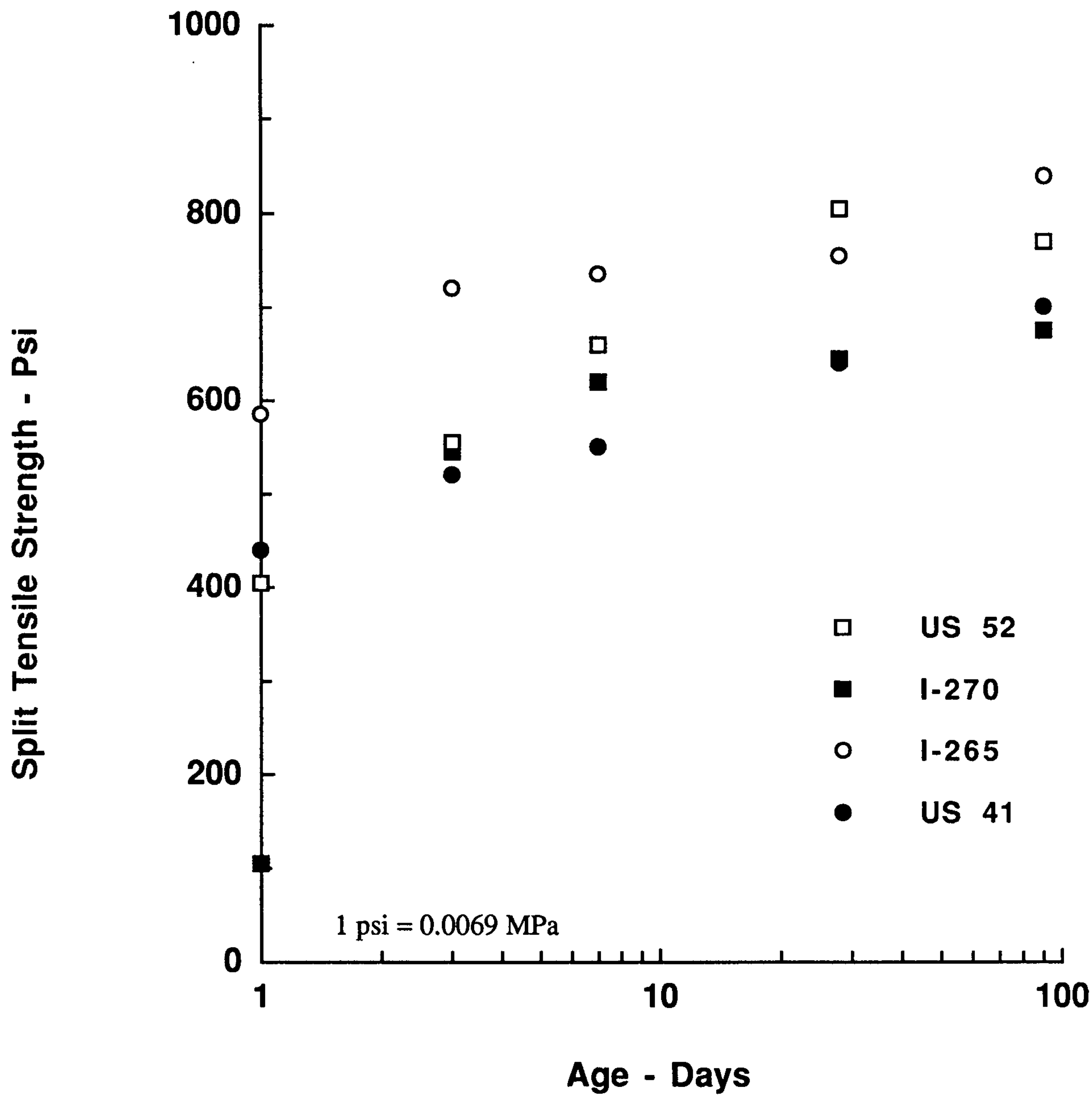

Figure A.11. Split tensile strength development of cylinders cast from LMC-III overlay mixes (standard curing).

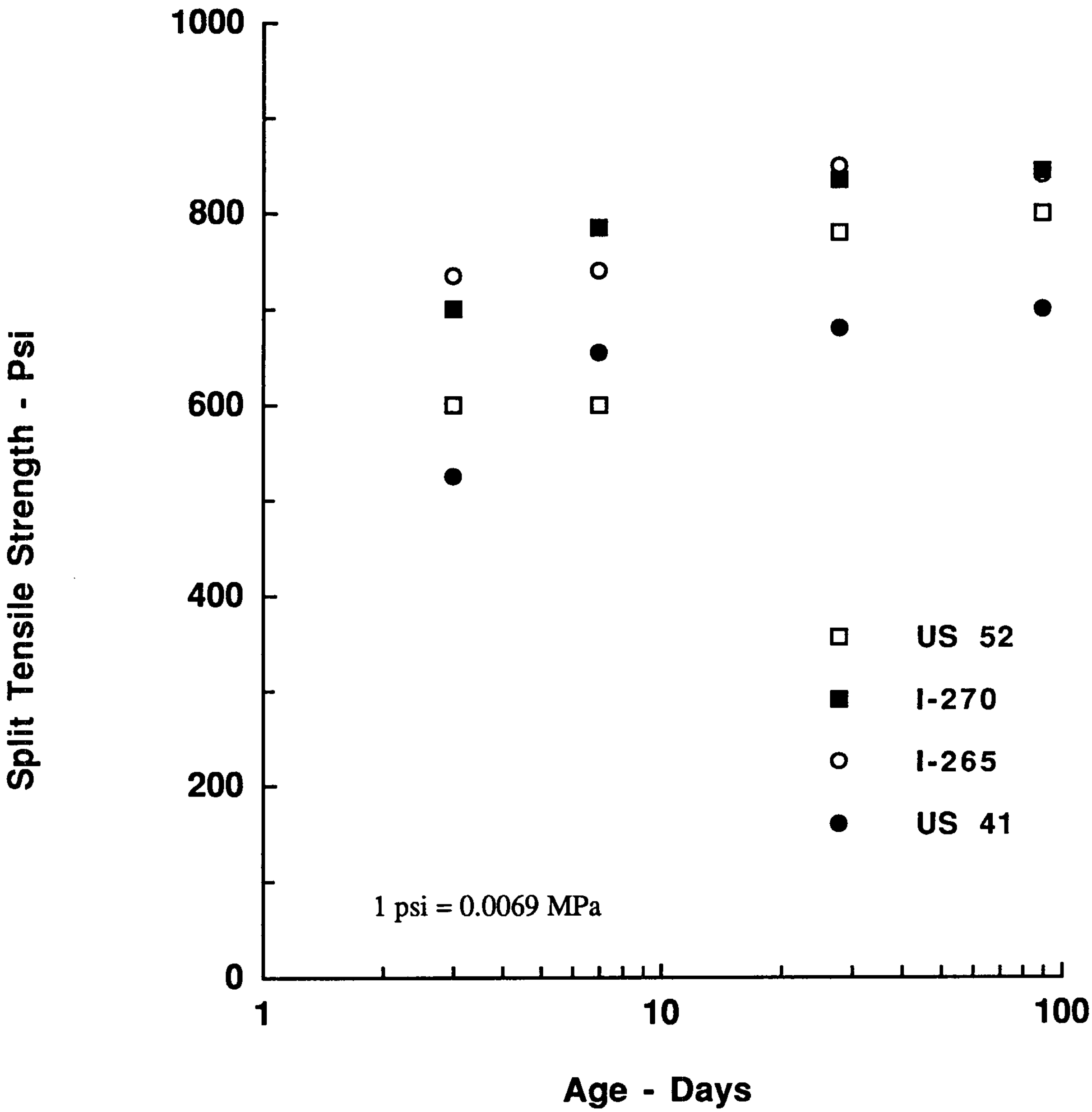

Figure A.12. Split tensile strength development of cylinders cast from SFC overlay mixes (standard curing).

both sets of data are comparable. Comparisons of Figures A.9 and A.11 indicate that highest LMC-III compressive and tensile strengths are obtained with the I-265 mix, and consistently lower strengths with the I-270 mix. In the case of SFC (comparing Figures A.10 and A.12), highest strengths were obtained from the I-265 and I-270 mixes, lowest from the US 41 mix.

Modulus of Elasticity

Static elastic modulus of elasticity was measured at 7 and 28 days on duplicate 4 × 8 in. (100 × 200 mm) cylinders in accordance with ASTM C469 procedures (Table A.10). For any given location and age, moduli of SFC concrete were consistently higher than that of LMC-III concrete, which is to be expected considering the difference in strengths. Moduli followed the same basic trend as for compressive strengths. For LMC-III mixes, lowest moduli and strengths were obtained at the I-270 location, highest at I-265. For SFC, lowest moduli and strengths were recorded at US 41, highest at I-265. Attempts were made to extract cores through the overlay and into a sufficient depth of the original deck to obtain specimens from which the modulus of the original concrete could be determined. Because of variable depth areas, the presence of the bottom mat of steel, and the need to obtain at least a core 8 in. (200 mm) long, the only site at which this could be accomplished was the I-270 location. Modulus of elasticity measured for this concrete was 4.30 million psi (29.6 GPa).

Table A.10. Elastic modulus of specimens cast from overlay concretes.

Site No.	Age (days)	LMC-III modulus (million psi)	LMC-III strength (psi)	SFC modulus (million psi)	SFC strength (psi)
1. US 52	7	—	—	—	—
	28	3.45	6,330	4.45	8,460
2. I-270	7	2.55	4,720	3.95	8,750
	28	3.00	5,840	4.55	10,670
3. I-265	7	3.90	6,460	5.10	10,370
	28	3.95	6,720	5.80	11,330
4. US 41	7	3.95	5,050	4.40	5,810
	28	3.90	6,040	5.00	7,100

Note: 1 million psi = 6.896 GPa

Resistance to Freezing and Thawing

Test specimens were prepared for evaluation of freeze-thaw resistance testing using modified procedures developed under SHRP contract C-203. The modified procedures essentially consist of an AASHTO T 161 B method (freeze in air/thaw in water) with the specimens wrapped in towels during the air freeze so as to reduce drying from the surface during the freeze cycle. Janssen (forthcoming) has found this technique to be no less, and perhaps somewhat more, severe than Procedure A, where specimens are kept continuously immersed in water throughout the period of test. Prismatic test specimens 3 × 4 × 16 in. (75 × 100 × 1025 mm) long were cast, given the standard field cure, and transported back to the laboratory the following day. During transport to the laboratory, specimens were left in their molds, wrapped with moist burlap, and sealed in plastic bags to minimize moisture loss. The LMC-III specimens were air cured until time of test at age 14 days, as moist cure is inappropriate for this material. The SFC specimens were cured in saturated limewater until time of test. Results after 300 cycles of testing are presented in Table A.11. Shown are durability factor (DF), expansion of the prisms, and mass changes. All reported results represent the average of three test prisms. All mixes show acceptable behavior through 300 cycles, commonly taken as the limit of the test. The only mix which exhibited less than ideal behavior was the I-270 SFC mix, where DF was 88 at 300 cycles and expansion was slightly higher than desirable at 0.05%.

Table A.11. Resistance to freezing and thawing of overlay concretes after 300 cycles of testing.

Site No.	Type	Durability Factor	Expansion (%)	Mass Change (%)
1. US 52	LMC-III	102	0.01	+0.40
	SFC	93	0.01	+0.07
2. I-270	LMC-III	96	0.03	+0.54
	SFC	88	0.05	+0.14
3. I-265	LMC-III	100	0.02	+0.14
	SFC	98	0.01	-0.12
4. US 41	LMC-III	100	0.02	+0.27
	SFC	99	0.00	-0.05

Characteristics of Air Void Systems

Companion 4 × 8 in. (100 × 200 mm) cylinders were cast from overlay mixes and transported back to the laboratory. They were then cut longitudinally and one cut surface of each cylinder lapped so as to allow microscopic determination of air content and air void system parameters using the linear traverse technique (ASTM C 457). Only specimens cast from SFC overlays were subjected to the linear traverse analysis, as LMC-III mixes do not require the use of entrained air in order to obtain freeze-thaw resistance. Air void system parameters are presented in Table A.12. All air void systems can be considered adequate.

Table A.12. Air void parameters for SFC bridge overlay mixes.

Site No.	Air Content (%)	Voids per Inch	Specific Surface ($in.^2/in.^3$)	Spacing Factor (in.)
1 (US 52)	8.90	14.60	656	0.0041
2 (I-270)	7.25	11.51	635	0.0052
3 (I-265)	5.60	11.61	830	0.0052
4 (US 41)	5.49	14.16	1032	0.0042

Note: 1 void per in. = 0.0394 void per mm; 1 in^2/in^3 = 0.0394 mm^2/mm^3; 1 in. = 25.4 mm

Permeability

An indication of relative permeability of the overlay concretes was obtained using the rapid determination of the chloride permeability of concrete procedure (AASHTO T 277). Specimens were prepared as 4 × 8 in. (100 × 200 mm) cylinders and cured identically to the compressive strength specimens. Tests were carried out at an age of 28 days. Cylinders were cut to obtain a 2 in. (50 mm) thick test slice from the top of each cylinder and prepared for testing. Test results are summarized in Table A.13. As this test normally exhibits considerable data dispersion, individual test results are shown.

Table A.13. AASHTO T 277 relative permeability results for overlay mixes.

Site No.	Type	Charge Passed (Coulombs)		
		Specimen 1	Specimen 2	Average
1. US 52	LMC-III	1178	1550	1364
	SFC	437	483	460
2. I-270	LMC-III	1354	1114	1234
	SFC	351	346	349
3. I-265	LMC-III	656	703	680
	SFC	311	304	308
4. US 41	LMC-III	715	1115	915
	SFC	484	627	555

Results are in the ranges of what normally is to be expected with LMC and SFC mixes. Typically, LMC exhibits charge passed in the range of 500 to 1500 coulombs. SFC permeabilities are significantly less, normally less than 1000 coulombs. For all sites, charge passed by SFC mixes was much less than for LMC. While SFC permeabilities were roughly equivalent for all sites (falling into what would be classified as the "very low" range), only LMC-III permeability for site 3 fell well into the "very low" range. This may be attributed to the lower w/c ratios at this site, as permeability is known to be a strong function of w/c (Whiting 1988).

Bridge Deck Overlay Core Testing

The actions of concrete placement, consolidation, finishing, and curing, may either separately or in combination, result in as-placed concrete having significantly different properties from specimens prepared in a standard manner from as-received samples of fresh concrete. Testing of concrete cores taken from in-place overlays was done as verification of results obtained from molded specimens. The following tests were conducted on 4 in. (100 mm) diameter cores, obtained by state forces approximately one week after the placement of each overlay section.

Resistance to Deicer Scaling

After receipt at the laboratory, SFC cores were placed in a moist room until 14 days of age so as to obtain the 14 days of moist curing suggested by the ASTM C 672 test procedure. LMC-III specimens were allowed to stand in laboratory air to obtain the recommended air cure for LMC. SFC specimens were also transferred to lab air at 14 days of age until both sets had reached an age of 28 days. They were then tested following ASTM C 672 procedures, modified for the use of cores. This was done by sealing a 4 in. (100 mm) diameter rigid plastic cylinder mold around the periphery of each specimen so that the top surface could be ponded with a 4% solution of calcium chloride. The specimens were rated visually, based on a scale of 0 (no scaling) through 5 (severe scaling with coarse aggregate visible over the entire surface). Average surface ratings (duplicate core specimens) at the recommended termination limit of 50 cycles are shown in Table A.14.

Table A.14. Summary of deicer scaling resistance testing of bridge overlay mixes.

Site No.	LMC-III	SFC
1. US 52	5— severe scaling, coarse aggregate visible over entire surface	1— very slight scaling, no coarse aggregate visible
2. I-270	2— slight to moderate scaling	1— very slight scaling, no coarse aggregate visible
3. I-265	5— severe scaling, coarse aggregate visible over entire surface	0— no visible scaling
4. US 41	1— very slight scaling, no coarse aggregate visible	0— no visible scaling

Results indicate significantly more scaling in most of the LMC-III mixes than in the SFC mixes. While early studies (Clear and Chollar 1978) did indicate tendencies toward scaling for some latex modifiers when subjected to this test procedure, field experience does not indicate scaling to be a widespread problem with LMC overlays. It is likely that the early age at which the lab specimens are subjected to the test, and the severity of test (which does not allow for drying during the test which would normally occur under actual conditions) contributed to the appearance of poor performance of the LMC-III mixes. It is also possible that LMC with some Type III cements is actually more susceptible to scaling than LMC made in the conventional manner with Type I cement. Long-term observations of the test sections should help answer these questions.

Permeability

As for the cast specimens, an indication of relative permeability was obtained on 4 in. (100 mm) cores taken from the in-place overlays through use of the AASHTO T 277 procedure. Cores were deliberately taken at variable depth areas to ensure that a 2 in. (50 mm) thickness could be obtained for test purposes. SFC cores were placed into moist storage after receipt at the laboratory, and testing was carried out at 28 days of age. LMC-III cores were allowed to stand in laboratory air to obtain the recommended air cure for LMC and tested at an age of 28 days. Results are shown in Table A.15.

For the most part, results obtained on core specimens were higher than for cast specimens, especially at sites no. 1 and 2. This may reflect the effect of cooler temperatures at these overlays placed earlier in the season than those at sites 3 and 4. Again, as for the cast specimens, charge passed values for SFC mixes are considerably less than for LMC-III mixes.

Table A.15. AASHTO T277 relative permeability core results for overlay mixes.

Site number	Type	Charge Passed (coloumbs)		
		Specimen 1	Specimen 2	Average
1. US 52	LMC-III	2019	2085	2056
	SFC	616	679	647
2. I-270	LMC-III	2293	2650	2471
	SFC	444	416	430
3. I-265	LMC-III	677	643	660
	SFC	403	291	347
4. US 41	LMC-III	1246	895	1071
	SFC	563	729	646

Direct Shear Bond of Overlays

Direct shear bond of overlays was determined on 4 in. (100 mm) diameter cores drilled through the overlay into the existing slab so as to obtain sufficient substrate material with which to anchor the shear specimen into the test rig. Cores were moist cured and tested at an age of 28 days. The test method used was similar to U.S. Army Corps of Engineers Test Method CRD-C 90-73 "Method of Test for Transverse Shear Strength". To ensure a uniform direct shear bearing area in the circular test frame, cores were set in plaster. A 1/2 in. (12 mm) wide guillotine semi-circular plate was used to produce shear at the overlay/slab interface. Shear strengths are summarized in Table A.16. All sites meet the 200 psi bond (1.4 MPa) criterion previously cited (Knab et. al. 1989).

Table A.16. Direct shear strengths of cores taken from bridge overlays.

Site Number	Mix	Shear Strength (psi)		
		Specimen 1	Specimen 2	Average
1. US 52	LMC-III	525	500	515
	SFC	485	565	525
2. I-270	LMC-III	435	320	380
	SFC	445	320	385
3. I-265	LMC-III	265	300	285
	SFC	350	345	350
4. US 41	LMC-III	195	230	215
	SFC	295	295	295

Note: 1 psi = 0.0069 MPa

Highest shear strengths were obtained for site no. 1 (US 52), lowest for site no. 4 (US 41). As bond strength is a strong function of the quality of surface preparation, this may indicate that the surface was more carefully prepared at site no. 1 than on some of the others. This hypothesis is supported by the visual observations taken during follow-up surveys (see following section). The development of shear strength requires some time, as the initial tensile pull-off data (Table A.9) were relatively equivalent for all sites when tests were carried out 24 hours after placement on the LMC-III sections.

Follow-Up Surveys of Bridge Deck Overlays

Follow-up surveys were carried out after all overlays had been placed. Because the time frame over which the overlays were placed spanned a period of almost 4 months, not all overlays could be surveyed at an equivalent point in elapsed time. The times at which each overlay site was revisited are as follows: site no. 1, 24 weeks; site no. 2, 20 weeks; site no. 3, LMC-III, 19 weeks; SFC, 16 weeks; site no. 4, SFC, 15 weeks; LMC-III, 10 weeks. At each site, the overlay surface was visually inspected for signs of distress including cracking, spalling, or punchouts. The overlays were also sounded with a chain drag and steel rod to acoustically detect subsurface delaminations which could indicate unbonded areas beneath the overlay. As the overlays had not yet experienced their first winter, and as only a few months of traffic flow had been applied, distresses at this point would likely reflect only construction deficiencies which may lead to premature deterioration of the overlays. It is expected that future monitoring of the sections by state forces or as part of FHWA research programs will develop long-term data which will reflect materials durability and the ability of the overlays to carry traffic over long periods of time.

At site no. 1 (US 52), the overlays appeared to be in very good condition. In the LMC-III section, the only deficiencies observed were a few small surface cracks at the peaks of some tine marks and some blemished areas where tines were not cleanly done. In the SFC section the survey revealed a few small areas where tinting did not appear to be sufficiently deep, and a single short (8 in. (200 mm)) longitudinal crack 0.016 in. (0.4 mm) wide. Sounding did not indicate any hollow areas on the decks. The overall appearance was of a job carried out with excellent workmanship and attention to detail.

At site no. 2 (I-270), construction on the major pavement rehabilitation project was ongoing. Traffic had been diverted back onto the test sections cast earlier in the year to allow completion of paving operations. For this reason, the traffic control necessary to allow close inspection of the overlay surfaces could not be supplied, as this would have required closure of the interstate at this location. Limited observations were possible from the shoulder which had been closed to traffic. An area of cracking was observed at the south end of the SFC section (Figure A.13). state inspectors attributed this to errors made by the contractor in sawing the overlay at the end joint. These problems had also occurred on the LMC-III section, and the contractor was in the process of clean cutting the areas and replacing them with new concrete (Figure A.14).

At site no. 3 (I-265), the overlays appeared to be in good condition. There were a few areas, approximately 3 to 5 ft (1 to 1.5 m) in lateral dimension and 1 to 2 ft (0.3 to 0.6 m) long, which showed a greenish-yellow discoloration. This may have been due to improper mixing of latex in this portion of the concrete or staining from burlap curing mats which were in contact with the fresh overlay surface. At the end of the north span there was a small corner which had been repaired after a worker had stepped into the fresh concrete. On the SFC section there was a small area on the middle span where brooming had been inadequate (Figure A.15), and some evidence of pasty areas on the surface near the curbline (Figure A.16). Apart from these minor imperfections, there was no evidence of serious cracking and no hollow areas were detected on either deck.

At site no. 4 (US 41), there was significant evidence of construction deficiencies. On the LMC-III section there were a number of hollow areas (Figure A.17) along the construction joint between the test section and the overlay section placed adjacent to it later in the job. These areas measured about 1 ft. × 2 ft. (0.3 × 0.6 m) each and were scattered along the joint in all three spans. In addition, there were a number of longitudinal cracks (Figure A.18) from 1 to 2 ft. (0.3 to 0.6 m) long and ranging in width from 0.016 to 0.040 in. (0.4 to 1 mm). However, there were only 3 to 4 such cracks per span. The SFC section appeared to be in somewhat better condition. There was only one hollow area, at the construction joint near the north end of the northmost span, and two transverse cracks, one 14 ft. (4 m) long and having an average width of 0.016 in. (0.4 m), the second 5 ft. (1.5 m) long and having the same average width. Both cracks were located near the middle of the northmost span.

The large amount of "baseline" information which was developed for these test sections will make them ideal candidates for continued observation and testing. We would recommend that state forces or other agencies which will assume these functions for SHRP make periodic surveys of the bridge deck overlay test sections. Surveys at yearly intervals are recommended. These surveys should include visual observations, delamination detection, and chloride sampling in the overlay material. Records of traffic data across the bridges, if available, would also help in interpreting overlay performance.

Figure A.13. Cracking at south joint of SFC overlay on I-270 at Site no. 2.

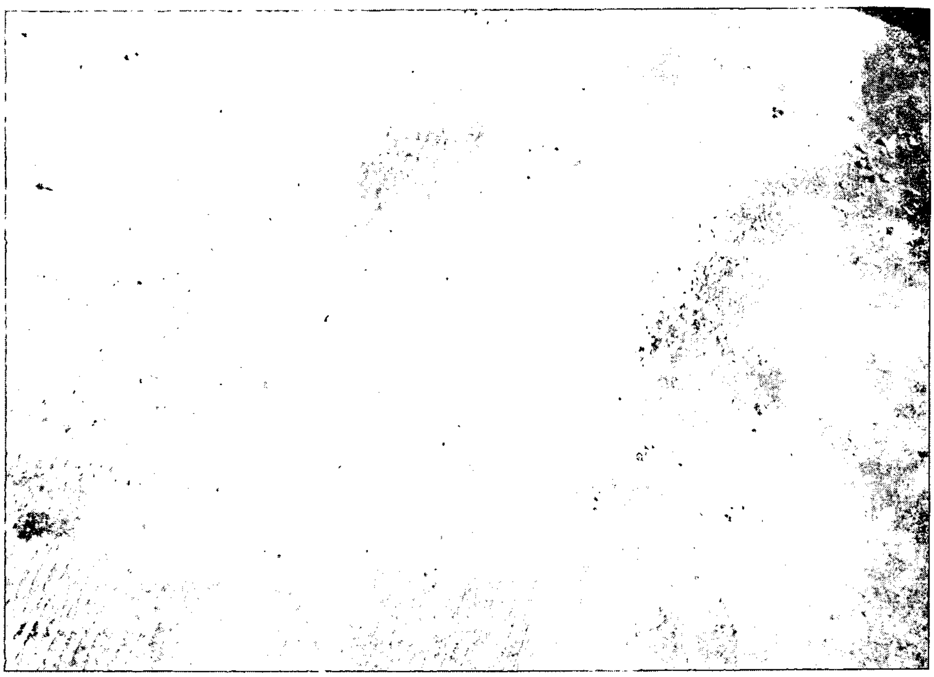

Figure A.14. Repaired area at end joint on LMC-III section at Site no. 2.

Figure A.15. Inadequate finishing on small areas of SFC overlay at Site no. 3.

Figure A.16. Paste visible on surface near curb line of SFC overlay at Site no. 3.

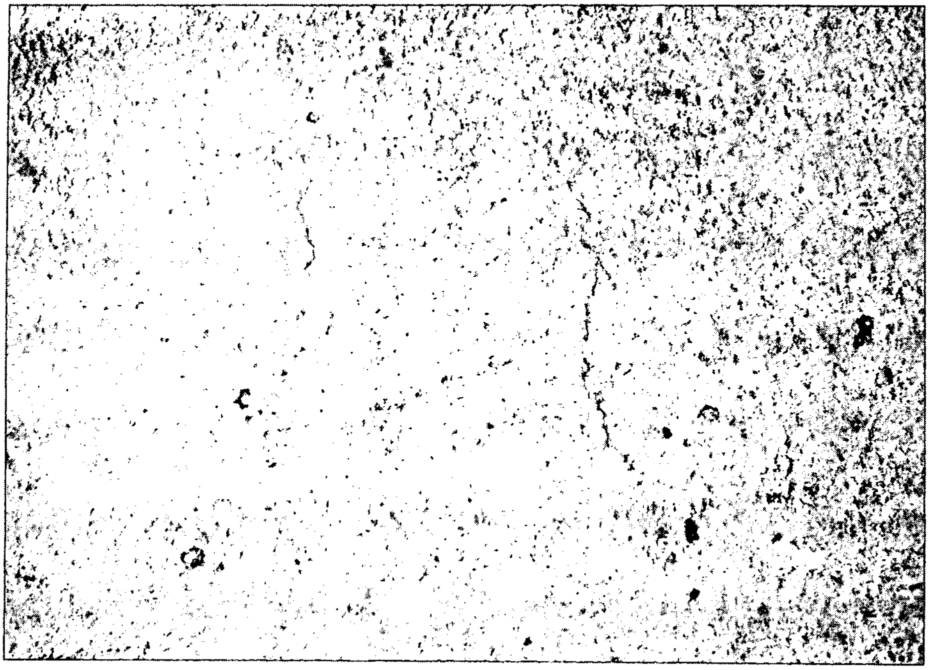

Figure A.17. Longitudinal cracking on LMC-III section at Site no. 4.

Figure A.18. Hollow area at construction joint marked out on LMC-III section.

Full-Depth Pavement Repairs

Research Plan

The need to maintain traffic flow creates an acute need for materials which can be placed and cured in as short a time as possible. The objective of the full-depth repair experiment was to evaluate a number of promising materials under a variety of field conditions and develop data on important materials properties and field performance.

Various methods of determining the in-place strength of concrete were also investigated, as a means of determining when the repair sections had reached the strengths necessary for opening to traffic. In addition to the maturity technique previously used on the bridge deck overlays, disposable maturity meters ("COMA" meters) were employed at one of the test sites. These miniature meters consists of a glass capillary containing a fluid with an activation energy similar to concrete. The amount of evaporation measured on a sight tube is therefore indicative of strength development in the concrete. Pulse velocity and temperature-matched curing techniques were also employed. A brief description of each follows.

The pulse velocity (PV) test method was selected to monitor concrete strength development since equipment is easy to use, portable, and commercially available (In- place methods 1988). With the use of new materials and interest in rapid repairs, early pavement openings, and Fast Track paving, the need exists to rapidly monitor strength gain with time. Pulses of compressional waves are generated by an electro-acoustical transducer that is held in contact with one surface of the pavement. Travel time is electronically measured for the pulses to travel through the concrete from the transmitting transducer to the receiving transducer held in contact with the pavement. Pulse velocity is calculated by dividing the travel distance by transit time. By establishing a pulse velocity-strength relationship, concrete strength can be nondestructively determined from pulse velocity.

The Virginia Transportation Research Council (Sprinkel and McGhee 1989) has experimented with the use of temperature-matched curing (TMC). This process involves maintaining test cylinders at the same temperature as the in-place concrete throughout the cure period by automatically controlling the temperature of a curing box through use of temperature sensors embedded in the placed concrete. Reusable molds are also commercially available as an alternative to an instrumented curing box. These insulated molds contain internal heaters which maintain the temperature of the 4 × 8 in. (100 × 200 mm) test cylinders close to the temperature of the concrete being monitored. When used with concrete mixes which generate considerable heat, the molds are appropriate to situations where concrete is placed in thick sections. For monitoring of thin sections such as bridge deck overlays, where the heat generated is dissipated into cooler substrates, the concrete in the insulated molds heats at a faster rate than the section being monitored; the TMC system is inappropriate for this type of application. Use of the system in the field requires that a portable compression testing machine with sufficient capacity to test the concrete under consideration be available. For the present project, a testing machine with a capacity of 200,000 lbs. (890,000 N) was used. Further details on these and other field testing procedures, including instructions on use of the methods, illustrations, and examples of their use are included in the implementation packages developed under this project and are separately available from SHRP.

The materials selected for evaluation covered a range of opening times which could be associated with various types of early-opening scenarios. Conventional materials, i.e., those used where opening time is not critical and 7 days or more can be allowed for curing were not included, as the objective of the project was to evaluate new technologies. As discussed, early opening times for full-depth repair applications now can be considered to lie in the intervals of 12 to 24 hours (certainly an accelerated schedule but not requiring advanced materials) and 4 to 6 hours (a very rapid schedule requiring modified-conventional or advanced materials). In the future, demands for even earlier openings will become pronounced, leading to demands for openings times of 2 to 4 hours even for full depth repairs, something now seen mainly for patching applications. The latter will require highly modified conventional materials or the use of very fast-setting proprietary products.

Materials and concrete mixes were chosen so as to fall within the projected opening times of 12 to 24 hours, 4 to 6 hours, and 2 to 4 hours. The criterion for opening was selected as a compressive strength of at least 2,000 psi (13.8 MPa) at the nominal opening time. This would correspond (Salami 1992) to an average modulus of rupture between 300 and 400 psi (2.1 and 2.8 MPa), and should be sufficient to resist flexural stresses imposed by traffic loads. Two sites were selected for field evaluations, one a section of I-20 near Augusta, Georgia and the other a section of State Route 2 near Vermilion, Ohio. At the first site three concrete mixes were evaluated. At the second site more sections were made available to the research team, and a total of eight mixes were evaluated. The materials, mixes, placements, and field and laboratory testing are described by site in the following sections. A discussion of loading effects and pavement fatigue analyses of the test sections is given in Appendix B of this report.

I-20 Site—Augusta, Georgia

A test site for installation of full-depth pavement repairs was located on EB I-20 west of the city of Augusta, between mileposts 189 and 192. The C-206 sections were added to a contract that was already in progress for the full-depth repair of working cracks on I-20 in that area. The project involved making full-depth repairs on existing 9 in. (230 mm) JPCP with 30 ft. (9 m) joint spacing. The repair lengths varied from 6 to 30 ft. (2 to 9 m). The ongoing repair work was being carried out each night between the hours of 7 p.m. and 6 a.m. The state required that the road be opened by 6 a.m. each day. Work involved full-depth saw cutting at the joints, removal of the slab by lift-out, drilling of dowel holes by automatic ganged dowel bar drills, and dowel insertion and epoxy grouting. Plastic grout-retention disks were used. Approximately 20 patches were made each night. Concrete deliveries started about 10 p.m. and continued until about 2 a.m. The investigators utilized the first patch of each night for their main materials test program. This allowed all testing and coring to be carried out prior to the opening of the highway. The age of the main test patch would then be greater than the nominal opening time chosen by the length of time needed to place the remaining sections each night. The *last* section placed each night was instrumented by thermocouple so that maturity data could be used to predict its strength at opening. This was used as the lower bound in the loading study of effects of early opening on pavement performance (see Appendix B). Replicate specimens for compressive strength were prepared from this patch as well.

Materials selected for the pavement repair mixes at this site consisted of: (1) Type I and III cements from Holnam, Inc.; (2) an angular quartz natural sand having specific gravity of 2.68, absorption of 0.56%, and FM of 2.62; and (3) a crushed siliceous natural gravel having a maximum topsize of 3/4 in. (19mm) with specific gravity of 2.66 and absorption of 0.5%. Admixtures included: (1) Darex Set Accelerator (DSA), a chloride free Type C accelerator; (2) Darex AEA, an organic-acid salt based air-entraining agent; (3) WRDA-79, a Type A water reducer; (4) Melment, a Type F (melamine-based) high-range water reducer; and (5) a 40% solution of calcium chloride. All admixtures except Melment and the calcium chloride were supplied by W.R. Grace & Co. Melment is supplied by Cormix Construction Chemicals. Note that while the developers of the "VES" mix under SHRP C-205 had used Darex Corrosion Inhibitor (DCI) as a set accelerator, the Georgia contractor had procured Darex Set Accelerator (DSA) for use on the project. As the two admixtures contain the same principal accelerative component (calcium nitrite), the investigators elected to use the DSA since it also contains a set control component which is helpful in maintaining workability under hot weather conditions.

Mixes used for the repairs are presented in Table A.17. These were: very early strength (VES) concrete developed by the SHRP C-205 investigators with the objective of opening approximately 4 to 6 hours after placement; "Fast Track I" developed originally by Iowa DOT (Grove, et.al. 1990) for opening times of 12 to 24 hours, and the mix being used by the contractor on the I-20 repair project, which was designed for opening 4 hours after placement of the last patch. This latter mix was designated GADOT mix by the investigators.

Table A.17. Mixes used at I-20 full-depth pavement repair site.

Material (Cubic Yard Basis)	VES	Fast Track	GADOT
Cement-lb.	Type III-870	Type III-740	Type I-752
Fine Aggregate-lb.	825	1,320	1,025
Coarse Aggregate-lb.	1,720	1,420	1,805
Water-lb.	335	262	285
Water reducer-oz.	—	44.0	—
Accelerator (DSA)-gal.	6.0	—	—
Accelerator ($CaCl_2$)-gal.	—	—	1.6
HRWR-oz.	43.5	—	—
AEA-oz.	43.5	12.0	9.8
w/c ratio	0.39	0.35	0.38

Note: 1 yd^3 = 0.7645 m^3; 1 lb. = 0.4535 kg.; 1 gal. = 3.78 L; 1 oz. = 29.57 mL

Prior to carrying out the field experiment representative materials were shipped from the Georgia ready mix plant to the investigators' laboratories for preparation of trial batches. Admixtures (with the exception of the calcium chloride solution being used on the job) were procured from local Chicago distributors, as these are available nationwide. Mix quantities were adjusted to achieve the desired opening strengths (2,000 psi (13.8 MPa) at 4 hours for VES and 2,000 psi (13.8 MPa) at 12 hrs for Fast Track). Since the GADOT mix could not be changed, the mix was prepared using the supplied design. Strengths of only 400 psi (2.75 MPa) were recorded after 4 hours, and the 2,000 psi (13.8 MPa) level was reached at 8 hours. However, it was noted during the subsequent field trials that the mix temperatures in Georgia were in excess of 90°F (32°C), which would tend to increase early strengths over those recorded in the room temperature lab trial mixes. In all cases cylinders were stored in an insulation box (Burg and Ost 1992) constructed out of expanded polystyrene which was cored to accept individual 4 × 8-in. (100 × 200 mm) cylinders immediately after casting. Each box holds a maximum of 28 such cylinders. This served to simulate casting in a thick section of pavement. After the trial batches had been confirmed a series of correlation batches were produced. From these batches cylinders and beams were cast to establish maturity relationships and relationships between pulse velocity and strength. Mortar cubes were also prepared from which to determine datum temperature and activation energy following ASTM C 1074 procedures. All cement, aggregates, and water were added through weigh hoppers at the ready mix plant. Air-entraining agents (AEA) and conventional water reducer were added through interlocked dispensers. Melment was added by hand to the hopper of the ready mix truck after cement, aggregates, water and AEA had been mixed thoroughly. Accelerators were added by hand to the trucks at the job site immediately prior to final mixing and discharge of the concrete. Each truck was batched with 6 yd^3 (4.5 m^3) of concrete. Haul time to the job sites varied from 10 to 15 minutes. The majority of mixes were very workable and could be finished fairly easily. No major placement problems were encountered.

Field Testing

After the first slab was removed each night the open hole was instrumented to allow in-place testing to be carried out. The instrumentation consisted of a thermocouple tree containing thermocouples placed at depths of 0.5, 5 (mid-depth) and 9 in. (13, 130, and 230 mm) below the surface of the pavement. In addition, expanded polystyrene blocks were attached to the subbase to allow for placement of pulse velocity transducers in the holes left by the blockouts after being removed subsequent to set of the concrete. A single thermocouple used to control the temperature-matched curing (TMC) molds was also placed at mid-depth of the pavement. In-place testing was done to evaluate material properties of fresh and hardened concrete and monitor strength gain at early ages. Slump, unit weight, temperature, and air content were measured on fresh concrete using standard procedures. In-place testing consisted of determination of water content using microwave drying, maturity monitoring (using the embedded thermocouples), pulse velocity testing, temperature-matched curing and retrieval and testing of cores for compressive strengths at opening. Plastic concrete test data are summarized in Table A.18.

Concretes were delivered at relatively high temperatures, from 89 to 96°F (32 to 36°C), to the job site. The daytime temperatures in the area were in excess of 100°F (38°C), and there were no convenient facilities for cooling the mix ingredients. This was of benefit for this particular work, however, as very high early strength gain was desired. As previously noted for the GADOT mix, it is doubtful whether sufficient strength would have been achieved in time for the 4 hour opening if concrete had been delivered at more normal temperatures. While the high temperatures led to some degree of slump loss in the mixes, the site was only 10 to 15 minutes away from the plant, and work was carried out rapidly once concrete was delivered. In most cases, adequate workability was maintained for the time needed to place the repair concrete.

Table A.18. Fresh concrete data determined at I-20 pavement repair site.

Mix	Date	Time (hr)	Slump (in.)	Air Content (%)	Initial Temp. (°F)	Unit Weight (lb/ft^3)
VES	7/15/92	2200	3.0	4.5	94	141.3
VES (rep.)	7/16/92	0230	4.8	4.8	93	140.3
Fast Track	7/16/92	2205	2.2	5.6	93	137.8
Fast Track	7/17/92	0030	3.0	4.5	89	140.0
GADOT	7/13/92	2230	2.8	3.7	96	142.8
GADOT (rep.)	7/14/92	0200	4.0	4.2	91	141.3

Note: 1 in. = 25.4 mm; °F = 1.8 × °C + 32; 1 lb/ft^3 = 16.02 kg/m^3

Determination of Water Content Via Microwave Drying

Water content of selected mixes was determined using the same microwave oven drying technique employed at the earlier bridge deck overlay placements. Measurements were carried out in a field test van using a portable 3000-watt generator set. A battery powered 5000-gm portable balance was used to determine mass loss. Results are presented in Table A.19.

Table A.19. Water contents determined at I-20 pavement repair site.

Mix Type	Reported Total Water (lb/yd^3)	Measured Total Water (lb/yd^3)	Difference (% of reported)
VES	357	358	+0.2
VES (rep.)	359	355	-1.1
GADOT	310	334	+7.7
GADOT (rep.)	309	313	+1.3

Note: 1 yd^3 = 0.7645 m^3

Differences between reported and measured water contents varied between approximately 0.2% and 7% of reported water content. For VES (greatest difference ~1%) this would change actual net w/c by 0.004, a negligible value. For the GADOT mix (greatest difference ~7%) this would increase w/c by 0.025. Typical decrease in compressive strength at 28 days of age to be expected from this magnitude of increase in w/c is about 150 psi (0.7 MPa)

(Kosmatka and Panarese 1988), close to expected within-batch deviations for field control and indicating that this also would have little significant effect on strength of the concrete. Water contents of the Fast Track mixtures could not be determined due to a malfunction in the oven during the final night of testing.

Temperature Development in Test Slabs

In order to estimate strength gain from maturity data, the temperature of the patches throughout the period of cure was required. Temperature was measured at 30-minute intervals using thermocouples embedded at the depths previously noted. Temperatures were also measured in the air adjacent to the slabs, and in a 4 × 8 in. (100 × 200 mm) test cylinder contained within the insulation box. Temperature profiles for the VES slab are shown in Figure A.19. It can be seen that the middle of the slab reaches the highest temperatures at any point in time, as would be expected based on the relatively stable night time air temperature, which allows heat to escape from the top of the slab. The insulated cylinders at first lag the slab, but then gain in temperature and eventually exceed the slab temperature. About 5 hours after placement, the slab temperature stabilizes at about 140°F (60°C) and then begins to trend downward, while the insulated cylinder temperature continues to rise to close to 160°F (70°C). Temperature profiles for the Fast Track slab are shown in Figure A.20. Peak temperatures are about 20°F (11°C) cooler than for the VES mix, which is to be expected considering the lower cement content and absence of accelerating admixtures. Again, the insulation box temperatures continue to rise after the slab has reached peak temperature. Finally, temperature profiles for the GADOT mix are shown in Figure A.21. Peak temperature of 140°F (60°C) at 5 hours is very similar to that of the Fast Track mix. In this case, however, cylinders in the insulation box did not get hotter than the slab itself. Also, the GADOT mix exhibits the greatest temperature differential across the slab of all the mixes studied.

The guide for evaluating thermal effects issued by SHRP (Andersen, Andersen, and Whiting 1992), offers a means of determining whether the high initial concrete temperatures experienced in these placements would result in future cracking or strength loss in the concretes. While cement content of the VES mix was outside the range of the tables, and the GADOT mix contained a chemical accelerator not included in the tables, the table for a cement content of 750 lb/yd^3 (445 kg/m^3) of Type III cement was used to estimate thermal effects in the Fast Track mix. Measured air temperature was 75°F (24°C), and initial concrete temperature was 93°F (34°C) for this mix. The table indicated that for an 8 in. (203 mm) thick pavement at an air temperature of 80°F (27°C) and concrete temperature of 90°F (32°C) no problems are to be expected. Indeed, follow-up examinations indicated no observable cracking in the slab.

Strength Gain Prediction Using Maturity and Pulse Velocity Techniques

Relationships between maturity (both Nurse-Saul and Arrhenius) and pulse velocity were established using laboratory trial batches prepared from material received from the I-20 job.

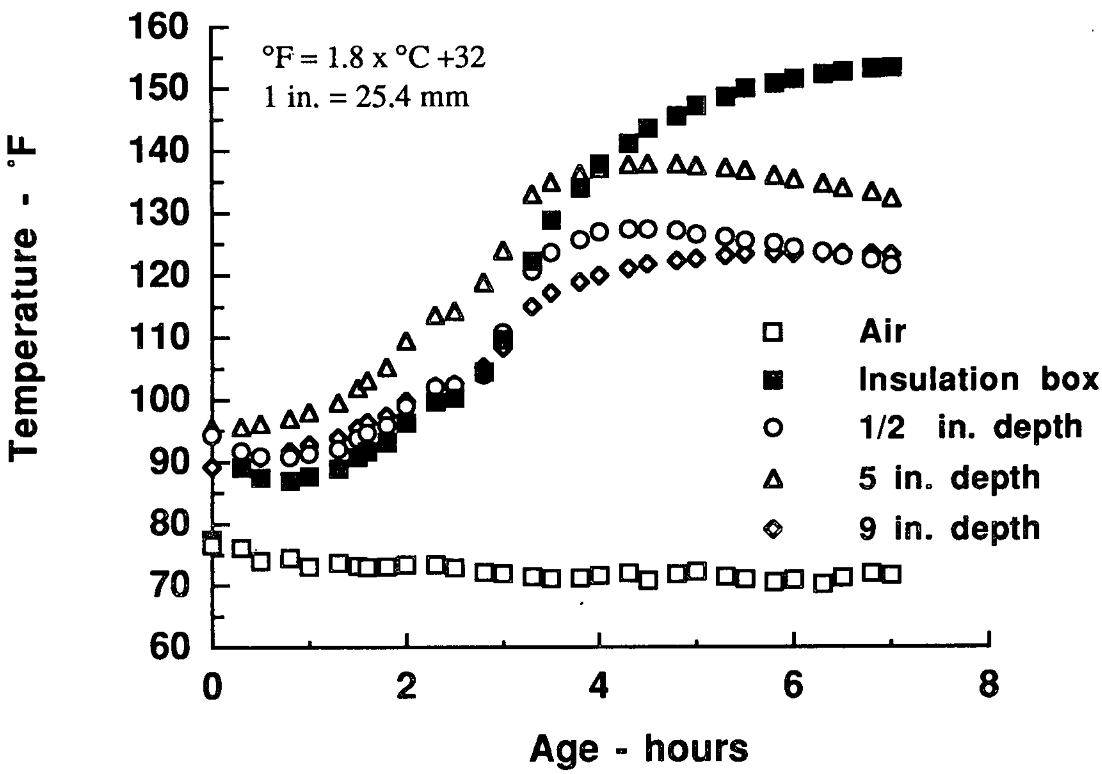

Figure A.19. Temperature profiles for VES mix used on I-20.

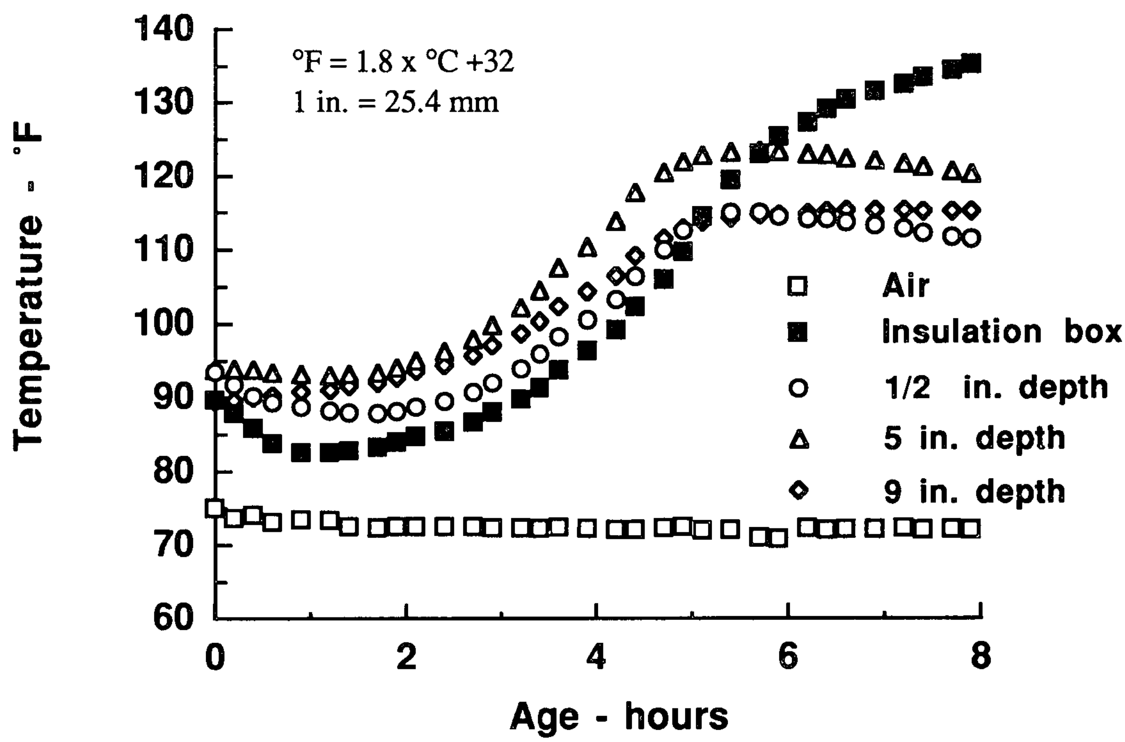

Figure A.20. Temperature profiles for Fast Track mix used on I-20.

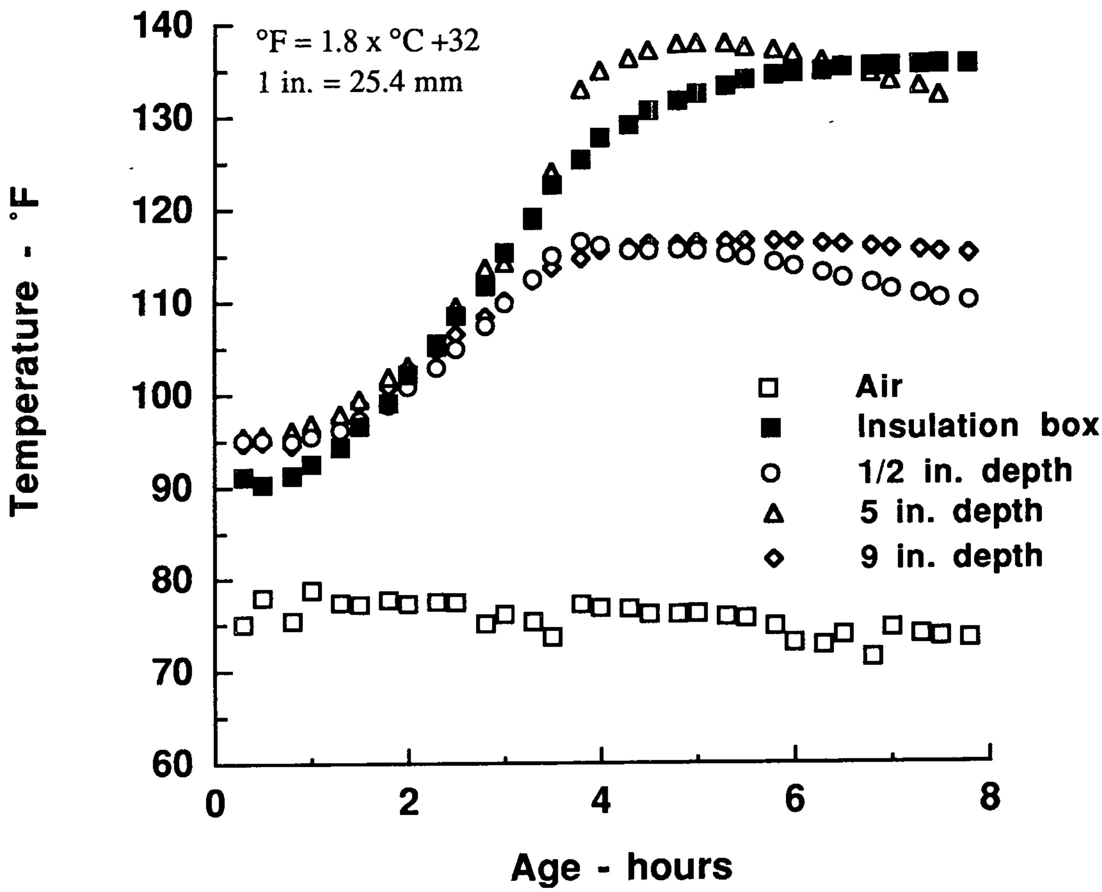

Figure A.21. Temperature profiles for GADOT mix used on I-20.

These relationships are given in Table A.20, and can be used to predict strengths with a fairly good degree of confidence based on the high correlation coefficients established. Temperature data acquired from the test sections were then used to develop plots of time versus both compressive strength and modulus of rupture. It should be noted that the strength which is predicted will be highly dependent on the location within the slab from which the temperature data are taken. As compressive strengths were to be compared with cores taken through the depth of the pavement, an average temperature across the pavement was calculated at each time increment. Predictions of early compressive strength for the three mixes, VES, Fast Track, and GADOT are shown in Figures A.22 through A.24, respectively. In addition to strengths predicted via maturity (Arrhenius approach for all these figures) and pulse velocity, the predictions based on TMC cylinders and cylinders stored in the insulated curing box are also shown.

Due to the large amount of insulation, positioning of the control thermocouple at mid-depth in the slab, and the heat applied in the TMC system, these cylinders invariably were hotter than either the actual pavement or the insulated cylinders in the curing box. For the VES mix (Figure A.22) strengths for the pavement predicted by maturity calculations were lower then TMC strengths by up to 500 psi (3.45 MPa). The insulated cylinders were initially weaker than the TMC cylinders, but the two strengths became closer after about 6 hours. The core strength at opening was quite closely predicted by the maturity function. Pulse velocity predicted a somewhat higher strength. This is reasonable, as pulse velocity transducers are placed near the middle of the pavement, where the higher temperatures would lead to a higher local strength in this layer. Similar data for the Fast Track mix are presented in Figure A.23. Again, highest strengths are obtained from the TMC cylinders. Pulse velocity predictions are close to strengths measured on the cylinders in the insulation box. Again, the best predictor of strength appears to be the maturity function, as the core strength lies exactly on the line of prediction. Figure A.24 shows data developed on the Georgia mix. Here TMC, insulated cylinders, and pulse velocity appear to be relatively equivalent. Maturity predicts somewhat lower strengths at early ages, but after about 5 hours all methods fall within the same general band. Unfortunately, cores suitable for testing could not be retrieved from this slab, as the concrete appeared to ravel as the core barrel cut into the slab. This mix was being used on a routine basis by the contractor, who opened the slabs each morning 4 hours after placing the final repair. Our data indicate that at 4 hours a conservative estimate of strength at opening would be about 1,000 psi (6.90 MPa). In order to obtain equivalent strengths upon opening the final slab, the value of 1,000 psi (6.90 MPa) was established as the target strength for opening all sections each night for the mixes included in the experiment. Figure A.25 and A.26 illustrate the prediction of gain in modulus of rupture with time for the three test sections on I-20. Predictions are shown using the maturity (Arrhenius) approach (Figure A.25) and pulse velocity measurements (Figure A.26).

VES gains flexural strength most rapidly of all three mixes studied. Over the time period 4 to 6 hours, predicted strengths (based on maturity) increase from 250 to 350 psi (1.7 to 2.4 MPa), which would allow early opening to traffic within this time interval. The GADOT mix gains strength more slowly, but becomes essentially equivalent to the VES mix after about 5 hours. The time period between 4 and 5 hours, however, is apparently very critical for the GADOT mix, and the policy of opening the pavement four hours after the last patch is

Table A.20. Developed prediction relationships for I-20 mixes.

Mix	Strength	Predictor	Datum/ Activation Temp.	Equation	Correlation Coefficient
VES	Compressive	Pulse Velocity	—	$\log f'c = 1.27 \times 10^{-4}\ PV + 1.844$	0.98
		Nurse-Saul	61°F	$1/f'c = 0.062/MAT + 1.72 \times 10^{-4}$	0.99
		Arrhenius	5200°K	$1/f'c = 6.06 \times 10^{-3}/MAT + 1.68 \times 10^{-4}$	0.99
	MOR	Pulse Velocity	—	$\sqrt{MOR} = 111.9 \log PV - 439.1$	0.98
		Nurse-Saul	61°F	$\log MOR = -78.3/MAT + 2.82$	0.97
		Arrhenius	5200°K	$\sqrt{MOR} = 10.92 \log MAT + 3.21$	0.98
Fast Track	Compressive	Pulse Velocity	—	$\log f'c = 5.47 \log PV - 19.09$	0.99
		Nurse-Saul	32°F	$f'c = -22.69 \times 10^{3}/MAT = + 5161$	0.99
		Arrhenius	2800°K	$f'c = -59.44 \times 10^{3}/MAT + 5163$	0.99
	MOR	Pulse Velocity	—	$1/MOR = 178.5/PV - 0.0114$	0.94
		Nurse-Saul	32°F	$1/MOR = 1.995/MAT + 7.59\text{x}10^{-4}$	0.97
		Arrhenius	2800°K	$1/MOR = 0.0517/MAT + 7.43 \times 10^{-4}$	0.98
GADOT	Compressive	Pulse Velocity	—	$1/f'c = 30.48PV - 1.91 \times 10^{-3}$	0.99
		Nurse-Saul	41°F	$\log f'c = -203.4/MAT + 3.717$	0.97
		Arrhenius	3500°K	$\log f'c = -6.762/MAT + 3.705$	0.98
	MOR	Pulse Velocity	—	$\log MOR = 1.355 \times 10^{4}/PV + 3.651$	0.99
		Nurse-Saul	41°F	$MOR = -8.933 \times 10^{4}/MAT + 521.2$	0.93
		Arrhenius	3500°K	$MOR = 3.000 \times 10^{3}/MAT + 522.3$	0.93

Note: $°F = 1.8 \times °C + 32$

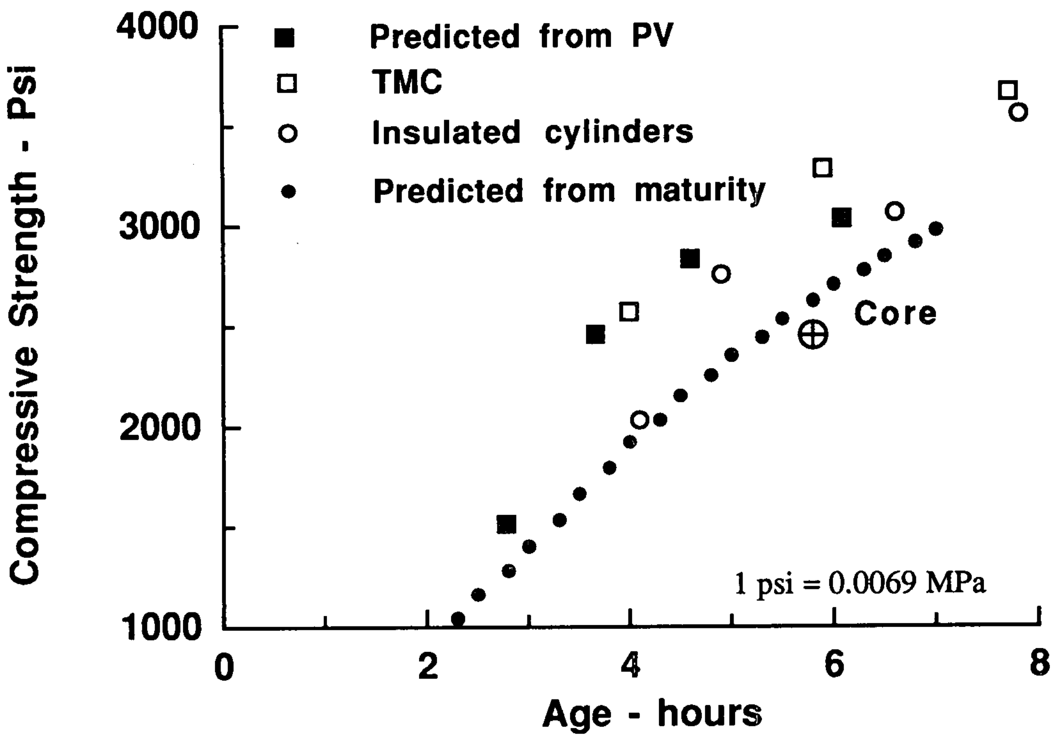

Figure A.22. Compresssive strength predictions for VES mix used on I-20.

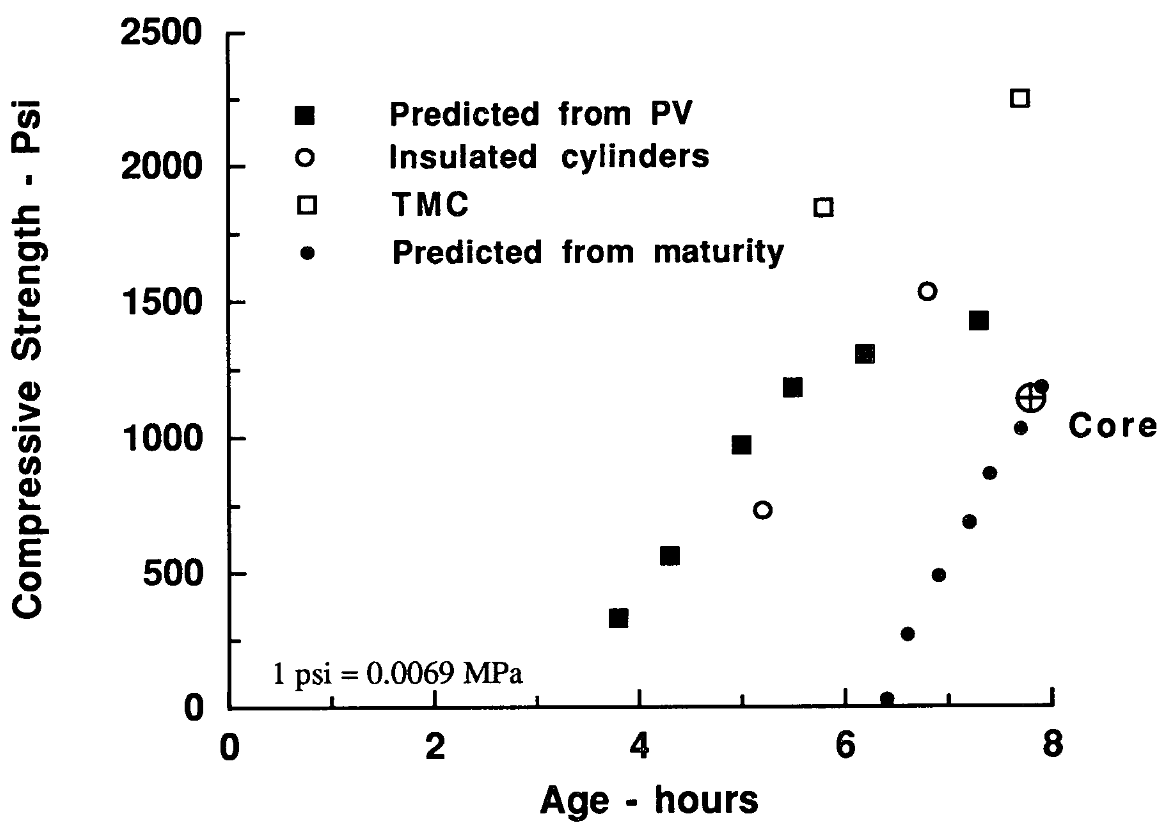

Figure A.23. Compressive strength predictions for Fast Track mix used on I-20.

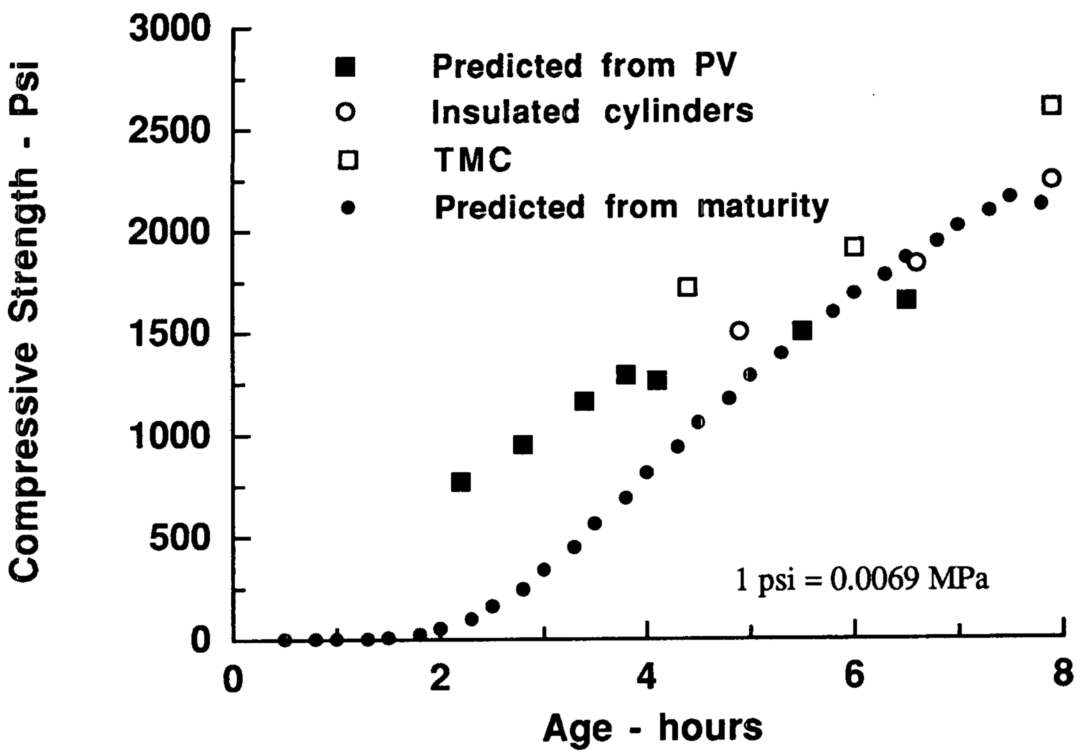

Figure A.24. Compressive strength predictions for GADOT mix used on I-20.

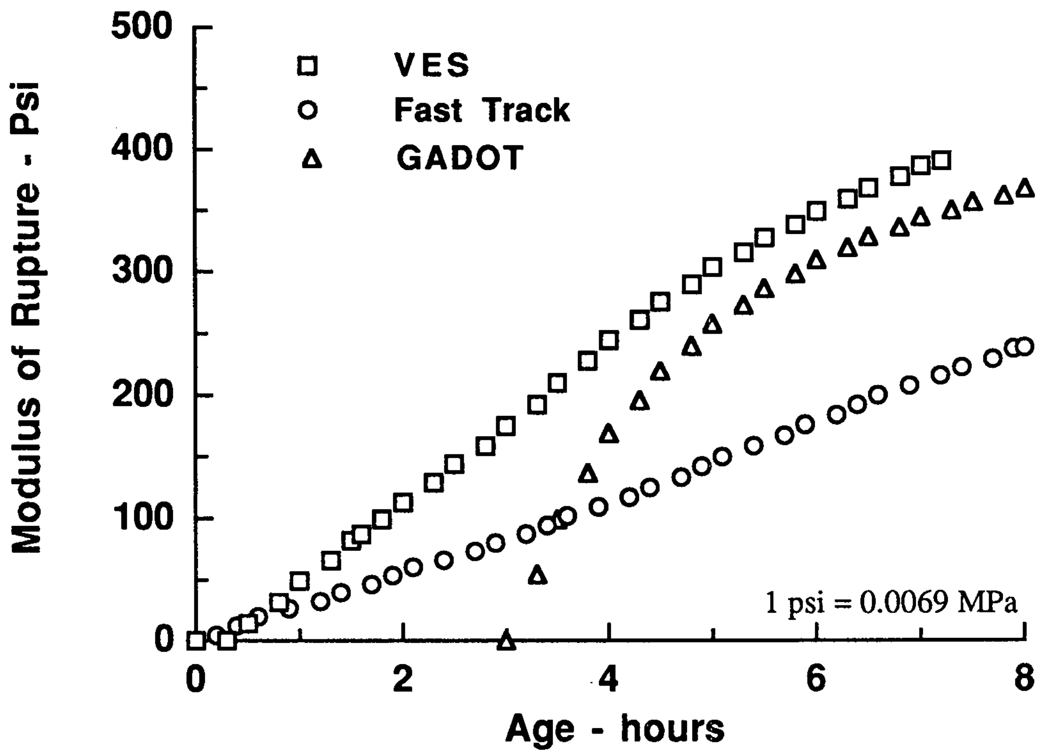

Figure A.25. Modulus of rupture predictions for I-20 mixes using maturity approach.

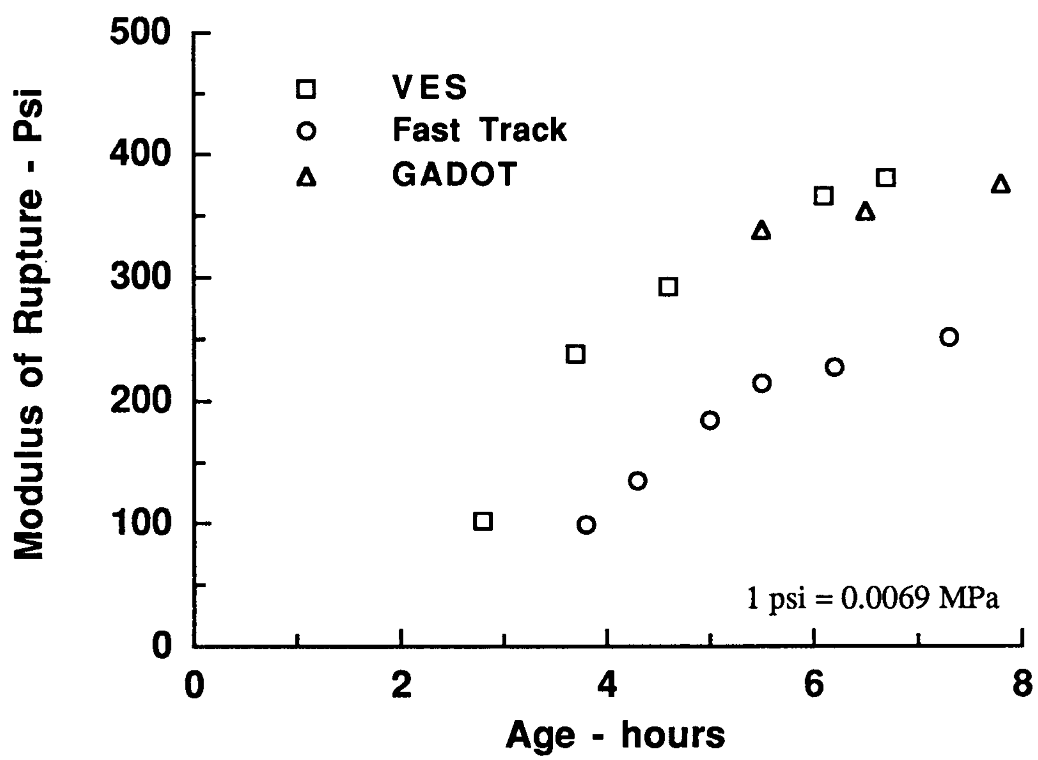

Figure A.26. Modulus of rupture predictions for I-20 mixes using pulse velocity measurements.

poured may result in some patches prepared with this mix having less than desirable strengths at first traffic loadings.

Finally, as would be expected, the Fast Track mix gains strengths more slowly than the other two. This mix was originally planned for an opening time frame of 12 to 24 hours, but because of the particular job restrictions previously noted, closure could not be maintained on the pavement beyond 6 a.m. the following day. This would result in the first patch being opened at about eight hours, the last patch at four hours. Figures A.25 and A.26 indicate a gain in strength from 100 to 250 psi over this time period.

Lab Testing of Specimens Cast From Repair Mixes in the Field

Laboratory testing was carried out to develop materials performance data on materials used at each repair test section. Specimens were prepared in the field and cast from mixes used to place the repairs. Specimens were cured as indicated in the following descriptions and transported back to the laboratory for further curing and testing. All specimens scheduled for testing at one-day age were tested using a field compression testing machine.

Compressive strength testing

Concrete was cast in rigid 4 × 8 in. (100 × 200 mm) plastic cylinder molds at the job site and consolidated by rodding. Cylinders for standard curing conditions were fabricated and cured at a temperature of 60 to 80°F (15.5 to 26.6°C). During each night of testing ambient temperatures were within this range. The following day temperature conditions were maintained by transporting specimens to a controlled temperature area. After all sections had been placed, sand-cushioned cylinders were transported back to the laboratory and stored in a fog room at 73 ±3°F (23 ±1.7°C) until time of test. Cylinders were kept in their plastic molds with lids taped on to minimize moisture loss during transport. Tests were conducted at 1, 3, 7, 28, and 90 days of age. Duplicate cylinders were tested at each age. Replicate sets of cylinders were fabricated, transported, cured, and tested at 28 days to evaluate batch-to-batch differences. Compressive strength testing was done in accordance with AASHTO T 22.

Results through 90 days are presented in Figure A.27. At early ages the VES mix was nearly 1500 psi (10.3 MPa) stronger than the other mixes tested. At 28 days Fast Track and GADOT were essentially equal, yet VES was still almost 1,000 psi (6.90 MPa) stronger in compression. Replicates for VES and GADOT agreed well with the first batch cast; however, the replicate for the Fast Track mix was significantly stronger, essentially equalling the VES mix at 28 days. While this could have been due to a lower water content in the mix, the microwave oven had malfunctioned just prior to casting the Fast Track mixes, and no confirmation of water content in these mixes could be obtained. At 90 days, VES retained its higher strength, followed by GADOT and Fast Track.

Split tensile strength testing

Duplicate cylinders were cast from the same concrete batches as those used for compressive strength cylinders. Standard-cured specimens were fabricated, transported, and cured

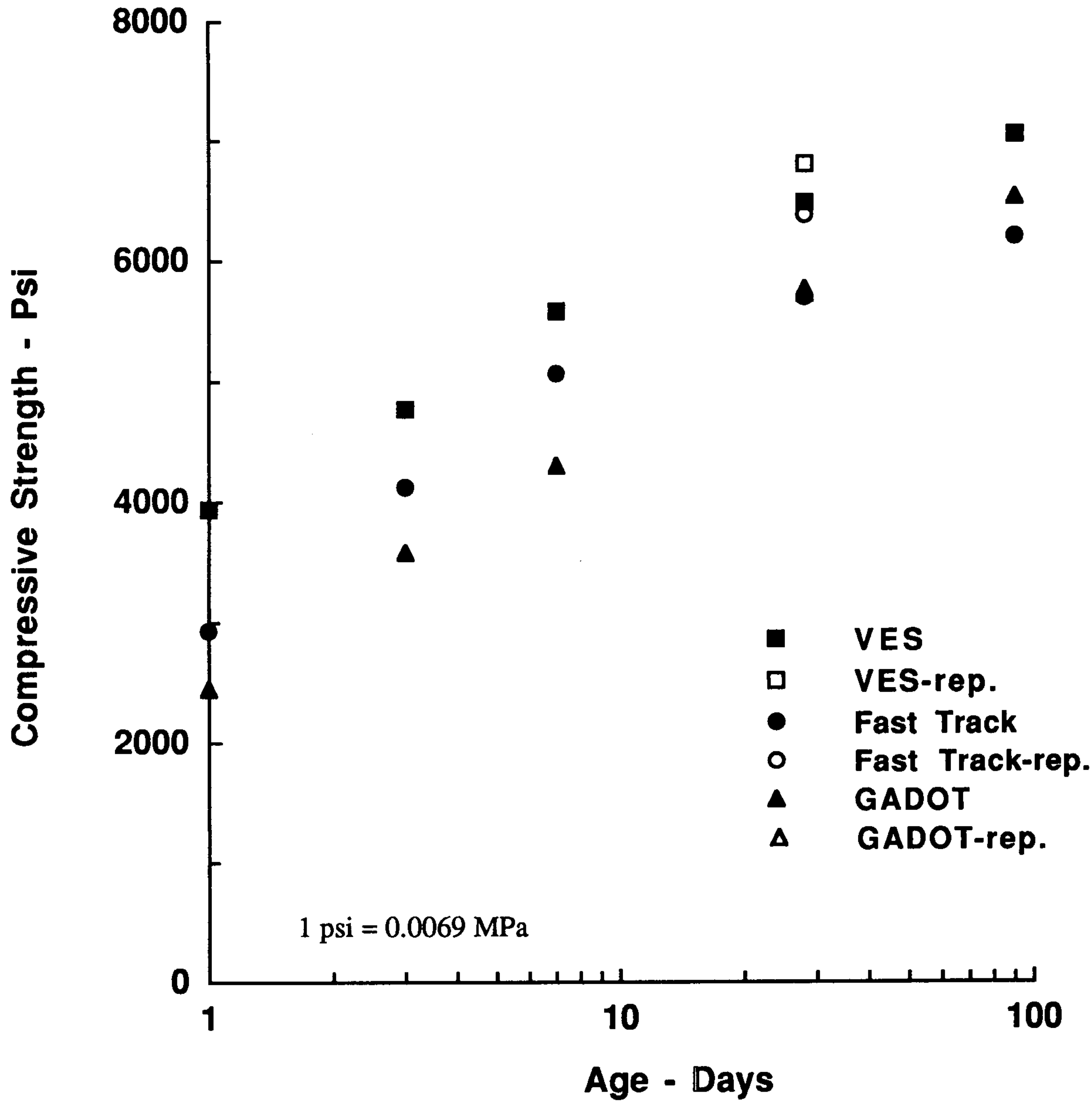

Figure A.27. Compressive strength development for cylinders cast from I-20 repair mixs.

identically to the compressive strength specimens. Split tensile strength was tested at the same intervals as for compressive strength. Split tensile testing was done in accordance with AASHTO T 198. A portable compression testing machine was used to test cylinders on-site at early ages. Splitting tensile strength test data is summarized in Figure A.28 for the three repair mixes. At one-day age the three mixes exhibit the same strength rankings as seen for compressive strength, with VES being highest, Fast Track intermediate, and GADOT having the lowest strength. At intermediate ages trends are not as clear cut and at 28 days all mixes exhibit essentially the same tensile strength. At 90 days VES and GADOT exhibit essentially equivalent tensile strengths, with Fast Track approximately 100 psi (0.7 MPa) lower.

Modulus of elasticity

Static elastic modulus of elasticity was measured at 7 and 28 days on duplicate 4 × 8 in. (100 × 200 mm) cylinders in accordance with ASTM C469 procedures (Table A.21). Modulus of VES is consistently higher than the other mixes at both ages, following the same trends as for compressive strength.

Table A.21. Elastic modulus of specimens cast from repair concretes.

Mix	Age (days)	Elastic modulus (million psi)	Compressive strength (psi)
VES	7	4.40	5,580
	28	4.90	6,480
Fast Track	7	4.00	5,060
	28	4.40	5,690
GADOT	7	4.20	4,300
	28	4.75	5,720

Note: 1 million psi = 6.896 GPa

Resistance to freezing and thawing

Prismatic test specimens 3 × 4 × 16 in. (75 × 100 × 1025 mm) were cast in triplicate, consolidated by rodding, and covered with wet burlap and polyethylene sheeting. They were allowed to stand at ambient temperatures until achieving final set. They were then placed in containers filled with saturated limewater and transported to a controlled temperature environment for the remainder of the field placements. They were then transported back to the laboratory and maintained under saturated limewater until 14 days of age. Results using the modified AASHTO T 161 B procedure described previously are given in Table A.22. Both VES and Fast Track mixes performed very well, with essentially no degradation through 300 cycles. The GADOT mix, however, showed significant deterioration with time. At 300 cycles durability factor had decreased to 75. One of the beams in the set of three had failed totally by 300 cycles. Linear traverse results for the three mixes are shown in Table A.23. The poor performance of the GADOT mix may be explained by the low air content obtained in the concrete, in spite of the fact that all other parameters seem acceptable. The

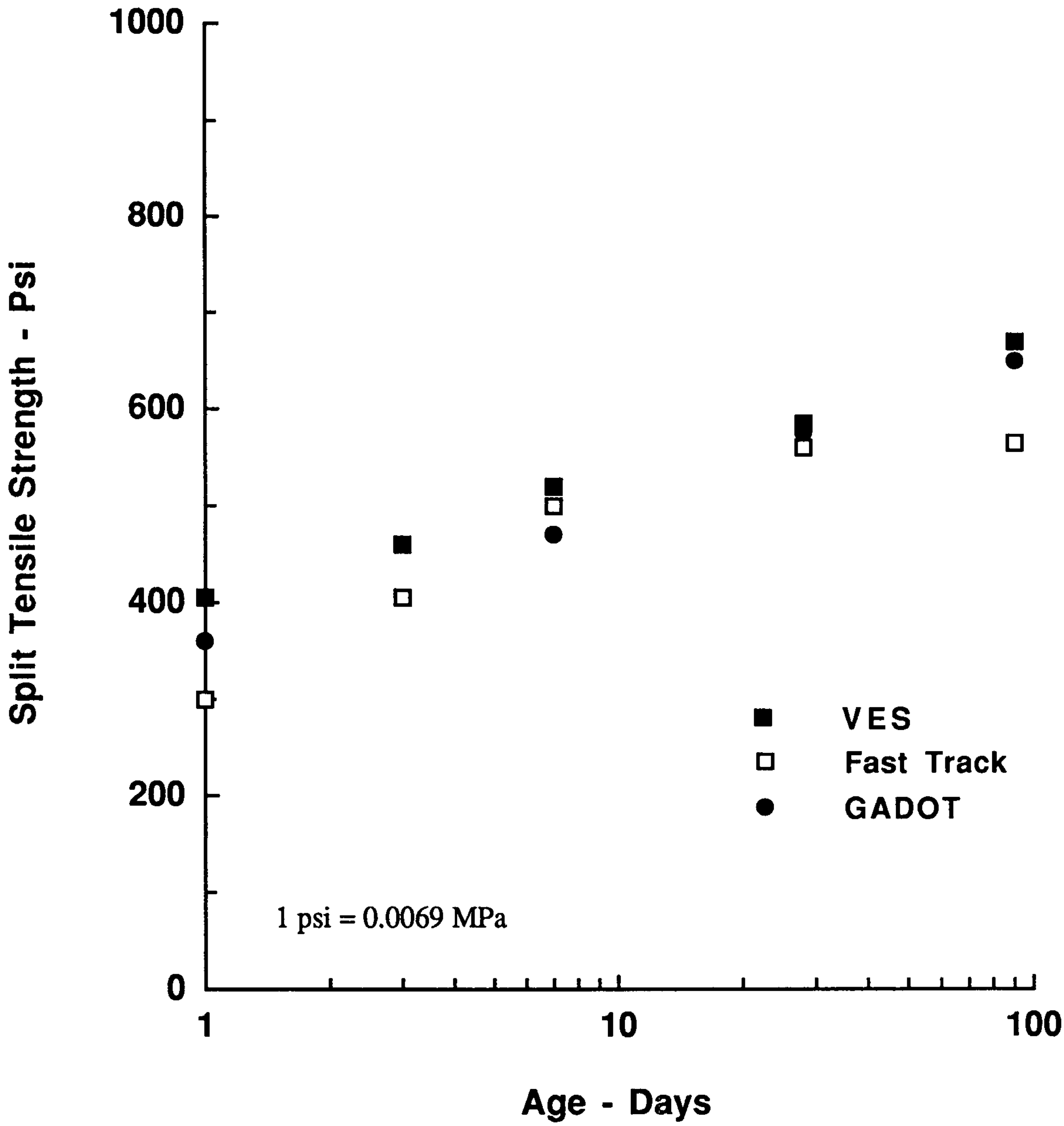

Figure A.28. Split tensile strength development for cylinders cast from I-20 repair mixes.

poor performance of the GADOT mix may also be a result of the use of calcium chloride as an accelerating admixture (Shideler 1952).

Table A.22. Resistance to freezing and thawing of I-20 repair concretes after 300 cycles of test.

Mix	Durability Factor	Expansion (%)	Mass Change (%)
VES	100	0.00	-0.42
Fast Track	100	0.01	-0.16
GADOT	75	0.12	+0.23

Table A.23. Air void parameters for I-20 repair concretes.

Mix	Air Content (%)	Voids Per Inch	Specific Surface (in^2/in^3)	Spacing Factor (in.)
VES	5.2	15.80	1206	0.0037
Fast Track	5.4	9.11	1355	0.0051
GADOT	3.4	18.23	1059	0.0032

Note: 1 void per inch = 0.0394 void per mm; 1 in^2/in^3 = 0.0394 mm^2/mm^3; 1 in. = 25.4 mm

Freeze-thaw testing was also carried out on duplicate 4 × 8 in. (100 × 200 mm) cores removed from the test sections just prior to opening. Cores were cured and conditioned for testing in a manner identical to the beam specimens, with the exception that testing was initiated at an age of 28 days. For these cores testing was carried out using AASHTO T 161A procedures, where specimens are frozen and thawed while immersed in water. Testing was carried out at a rate of approximately 3 cycles per day, as the 4 in. (100 mm) diameter cylindrical specimens respond slowly to temperature change in the particular equipment employed, which is normally used to test prismatic specimens of smaller cross-section. Results are shown in Table A.24. Behavior of VES and Fast Track mixes was similar to that seen in the modified AASHTO T 161 B procedure. While one of the GADOT core specimens performed fairly well, the companion failed at 180 cycles, indicating that questions of durability do exist with respect to this mix.

Table A.24. Resistance to freezing and thawing of I-20 repair concretes after 300 cycles of AASHTO T 161

Mix	Durability Factor	Mass Change (%)
VES	100	0
Fast Track	99	-0.05
GADOT	92*	+0.10

* Note: companion specimen failed at 180 cycles

SR 2 Site—Vermilion, Ohio

The second test site for evaluation of pavement repair mixes was located on State Route (SR) 2 between milepost 35 in Erie County and milepost 3 in Lorain County. SR 2 is a four lane, divided highway built to Interstate standards in 1974 and opened to traffic in early 1975. The test sections used on this project were originally built to determine the factors which influence D-cracking of highway pavements. The original project contained 104 test sections with most having 9 in. (230 mm) of JRCP with 40 ft. (12m) joint spacings. All transverse joints were originally doweled and sealed. Over the years slab cracking has been extensive, the most prevalent distress being mid-slab cracking, with some divided slabs and deteriorated joints. Curiously, there was not a lot of classic D-cracking observable in this stretch of SR 2.

The Ohio Department of Transportation had scheduled an extensive rehabilitation program for this site under State Project No. 6000-92 (sta. 1827+90 Erie to sta. 185+06 Lorain). Various portions of the route were to be totally reconstructed, overlaid, or repaired. Lateral drains and tied shoulders were to be added. The repair areas will be diamond ground after all repairs have been completed. The investigators were able to have a total of 80 6-ft. (1.8 m) repair sections in both EB and WB driving lanes used for the pavement repair experiment.

Repair procedures were almost identical to the I-20 Georgia site, except that the saw cuts for all sections had all been made the week prior to repair, instead of each night of repair. As the temperatures were moderate, all work was carried out during daylight hours. Work started about 7 a.m. each day, and the patches were ready for concrete delivery by 10 a.m. For most of the job temperatures were ideal, within the range of 60 to 80°F (15 to 27°C), allowing concrete delivery temperature to be fairly well controlled. Rain, however, was a problem, and some testing had to be discontinued at times. Opening times were left to the discretion of the investigators; this allowed opening of the sections to be based on prediction of strength gain using NDT methods, and avoided the restrictions on opening encountered on the I-20 job.

Materials selected for the pavement repair mixes at this site consisted of: (1) Type III cement from St. Marys Cement Co.; (2) proprietary rapid strength gain cementitious materials including Pyrament blended cement (extended setting time) (PBC-XT), a blended cement using alkali-activated alumino-silicates and meeting AASHTO M 240 (modified) and Rapid Set cement, a sulfo-aluminate based cement not covered by any AASHTO or ASTM specifications; (3) a sub-angular to rounded natural sand consisting of a mix of siliceous and calcareous minerals having specific gravity of 2.58, absorption of 1.54%, and FM of 2.62; and (4) an angular, somewhat elongated and bladed crushed limestone having a maximum topsize of 1/2 in. (12 mm) with specific gravity of 2.60 and absorption of 2.63%. Admixtures included: (1) Darex Corrosion Inhibitor (DCI), a calcium-nitrite based inhibitor meeting AASHTO Type C requirements for set accelerator; (2) Catexol A.E. 260, a synthetic surfactant based air-entraining agent; (3) Catexol 1000N, a Type A water reducer; (4) Catexol 1000R, a Type B and D set retarding admixture; (5) Catexol 1000SP-MN a Type F high-range water reducer; (6) Melment, a Type F (melamine-based) high-range water reducer; (7) anhydrous fine granular citric acid (a set control agent for Rapid Set cement);

and (8) flake calcium chloride (77 to 80% assay). All admixtures except Melment and DCI were supplied by AXIM Concrete Technologies. Melment was supplied by Cormix Construction Chemicals. DCI was supplied by W.R. Grace & Co.

Mixes used for the repairs are shown in Table A.25. For the 2- to 4-hour openings, the two proprietary products were employed. For 4- to 6-hour openings, the proprietary products were compared with a modification of the VES formulation (see discussion below) and a mix being used by ODOT for fast setting repair requirements. This mix was similar in some respects to the GADOT mix used at the I-20 site, although it had a higher content of Type III cement and used a set retarder to extend working time. For the 12 to 24 hour openings the Fast Track mix and a High Early Strength (HES) mix developed by the C-205 investigators were used.

Prior to carrying out the field experiments materials were shipped to the investigators' laboratories for trial batching and preparation of correlation specimens. All materials were shipped from those manufacturers supplying to the actual job. Considerable difficulties were encountered in reaching the required opening strength for some of the mixes. The VES formulation used at the I-20 site had to be modified by increasing cement content by 45 lb/yd^3 (26.7 kg/m^3) and nearly doubling the dosage of HRWR. Even so, the w/c ratio of 0.45 necessary to obtain the desired workability was significantly higher than that used for the I-20 site. Similarly, mix PC 2, utilizing Pyrament cement, was prepared at a w/c ratio of 0.29, the maximum recommended for this product. The need to utilize higher cement, water, and admixture contents than originally intended in these and other mixes was partly a result of the use of the angular, small size crushed limestone chosen as the coarse aggregate. The small topsize requires use of a higher sand content in the mixes, which increases aggregate surface area and requires more cement paste to achieve proper workability. The need to produce mixes meeting strength criteria resulted in a large expenditure of sample materials, and time limitations prevented further optimization prior to carrying out the actual field work.

In addition to the changes in mix designs, the production equipment available at the job site required institution of some less than optimal practices. The ready mix plant used was a portable job site plant having a 2.5 yd^3 (1.9 m^3) capacity for each mix. It was set up to handle only a single type of cement, sand, coarse aggregate, water, and an air-entraining and two other liquid admixtures. The admixture supplier was Axim Chemicals, which produces a variety of liquid admixtures under the trade name Catexol. Therefore, the AE 260, 1000R, 1000N, and 1000SP-MN were added via dispensers at the plant. All other admixtures had to be manually added to the ready mix truck. The HRWR's and citric acid were added at the plant, and accelerators were added at the job site. Additionally, because only one cement silo was available for automatic batching of the Type III cement, the Pyrament and Rapid Set cements had to be delivered in bags and manually added to the ready mix truck; this was a slow and tedious operation which significantly increased the slump loss of the concretes. This procedure, which is typically employed for production of small patching mixes using job site mixers, is not recommended for typical ready mix concrete operations. However, it was the only way to introduce these products to the ready mix trucks at this site. Fortunately, haul time to the job was less than 10 minutes, which helped to minimize the slump loss problem in most cases.

Table A.25. Mixes used at SR 2 full-depth pavement repair site.

	2 to 4 Hour Opening			4 to 6 Hour Opening			12 to 24 Hour Opening	
Material Cubic Yard Basis	PC 1 (Pyrament)	RSC 1 (Rapid Set)	VES (Type III)	PC 2 (Pyrament)	RSC 2 (Rapid Set)	ODOT (Type III)	Fast Track (Type III)	HES (Type III)
Cement (lb.)	900	750	915	850	650	900	750	870
Fine Aggregate (lb.)	1,430	1,390	1,180	1,450	1,400	1,000	1,360	775
Coarse Aggregate (lb.)	1,370	1,290	1,090	1,400	1,300	1,420	1,200	1,645
Water (lb.)	240	302	409	246	325	360	369	357
Water reducer (oz.)	—	—	—	—	—	—	45	—
Accelerator (DCI) (gal.)	—	—	6.0	—	—	—	—	4.0
Accelerator ($CaCl_2$) (lb.)	—	—	—	—	—	18	—	—
Set Retarder (oz.)	—	—	—	—	—	13.5	—	—
Citric Acid (lb.)	—	11.25	—	—	7.8	—	—	—
HRWR (Melment) (oz.)	—	—	45.75	—	—	—	—	—
HRWR (Catexol 1000SP-MN) (oz.)	—	—	—	—	—	—	—	209
AEA (oz.)	—	22.5	73	—	26	90	52.5	69.6
w/c ratio	0.27	0.40	0.45	0.29	0.50	0.40	0.49	0.41

Note: 1 yd^3 = 0.7645 m^3; 1 lb. = 0.4535 kg.; 1 gal. = 3.78 L; 1 oz. = 29.57 mL

For most of the mixes some workability problems were encountered in the field. Both state and contractor personnel had to be educated to the fact that these were not normal concrete mixes, and that they were very time sensitive. There was no time to adjust mixes at the plant, or to add water or HRWR incrementally at the job site. One had to add materials as rapidly as possible at the plant, quickly get the truck to the job (fortunately haul times were no more than 10 minutes), add admixtures if required, and discharge, consolidate, and finish these concretes as quickly as possible. The contractor also wished to save material by batching larger amounts and filling more sections, but holding the truck until the first section was done sometimes resulted in unacceptable slump loss. In most cases, once the procedure for each mix was learned by trial and error (usually taking the first two truckloads) operations generally went fairly smoothly. No one mix or material could be singled out as being any more problem-prone than another. While the manual batching of bagged product did appear to lead to some slump loss problems with the proprietary cements, this was not an ongoing problem. For instance, while many problems in finishing and slump loss were encountered with the first day's production using Pyrament cement (PC 1), the second day's production (PC 2) went much more smoothly. Conversely, the first day's production using Rapid Set cement (RSC 1) was virtually trouble free; however, many problems in erratic behavior of mixes were encountered during the RSC 2 placements. Some of these inconsistenciés were explained by fluctuations in aggregate moisture and batching errors, others had no apparent cause. The investigators do believe that if the concrete had been produced by a well-controlled fixed-site ready mix facility (rather than at a temporary batch plant), many of these problems could have been avoided.

Table A.26. Fresh concrete data determined at SR 2 pavement repair site.

Mix	Date	Time (hr)	Slump (in.)	Air Content (%)	Initial Temp. (°F)	Unit Weight (lb/ft^3)
HES	9/1/92	1450	5.8	9.7	85	132.8
HES (rep.)	9/1/92	2015	4.6	9.7	85	131.5
Fast Track	9/2/92	1100	0.5	5.5	83	139.9
Fast Track (rep.)	9/2/92	1710	4.5	10.0	82	130.2
PC1	9/3/92	1040	1.2	2.8	84	148.4
PC1 (rep.)	9/3/92	1515	1.2	2.6	87	142.0
RSC1	9/4/92	1055	7.0	3.4	80	141.0
RSC1 (rep.)	9/4/92	1500	5.2	7.7	83	135.4
VES	9/8/92	1115	3.5	7.0	93	129.6
VES (rep.)	9/8/92	1440	4.0	7.5	92	132.5
PC2	9/9/92	1030	4.4	1.8	77	145.0
PC2 (rep.)	9/9/92	1415	3.5	3.5	85	150.0
RSC2	9/10/92	1300	1.5	3.3	82	142.4
RSC2 (rep.)	9/10/92	1500	3.5	4.2	86	139.6
RSC2 (rep.2)	9/9/92	1445	3.5	6.5	—	137.4
ODOT	9/11/92	1130	1.2	5.7	85	137.6
ODOT (rep.)	9/11/92	1715	2.6	4.5	86	135.4

Note: 1 in. = 25.4 mm ; °F = 1.8 × °C + 32 ; 1 lb/ft^3 = 16.02 kg/m^3

Temperature development in test slabs

In order to estimate strength gain from maturity data, the temperature of the sections throughout the period of cure was required. Temperature was measured at 30 minute intervals in the slabs using thermocouples embedded at the same depths as at the I-20 site. Temperatures were also measured in the air adjacent to the slabs and in a 4 × 8 in. (100 × 200 mm) test cylinder contained within the insulated field curing box. Temperature profiles for the two to four hour opening mixes, i.e., PC 1 and RSC 1, are shown in Figures A.29 and A.30, respectively. In contrast to the temperature profiles for the mixes used at the I-2o site, mix PC 1 shows relatively little heat developed over time. With the exception of a rapid jump in temperature, approximately 20°F (11°C) during the first hour after placement, slab temperatures for the PC 1 section stay relatively constant over time. Mid-slab temperature reaches a peak of 108°F (42°C) at 1-½ hours after placement. As for the I-20 sections, the highest temperatures at opening were generated in the insulated cylinder. Profiles for the RSC 1 section show a much slower development of temperature during the first 1-¼ hours. Temperature at mid-slab rises rapidly beginning at about 1-¼ hours to a peak of 125°F (52°C) at 2.5 hours after placement.

Temperature profiles for the two proprietary mixes used in the four to six hour opening range are shown in Figures A.31 and A.32. Again, mid-slab temperatures for the Pyrament cement mix (PC 2) rise to about 105°F (41°C) after 1.5 to 2 hours. Temperature for the Rapid Set cement mix (RSC 2) peaks close to 130°F (54°C) after four hours. The VES and

Table A.27. Water contents determined at SR 2 pavement repair site.

Mix Type	Reported Total Water (lb/yd^3)	Measured Total Water (lb/yd^3)	Difference (% of reported)
HES	394	396	+0.5
HES (rep.)	393	391	-0.5
Fast Track	399	382	-4.3
Fast Track (rep.)	380	364	-4.2
PC 1	298	297	-0.3
PC 1 (rep.)	298	310	+4.0
RSC 1	383	400	+4.4
RSC 1 (rep.)	349	364	+4.3
VES	424	449	+5.9
VES (rep.)	408	444	+8.8
PC 2	307	337	+9.7
PC 2 (rep.)	303	321	+5.9
RSC 2	343	338	-1.4
RSC 2 (rep.)	350	352	+0.6
ODOT	418	402	-3.8
ODOT (rep.)	429	419	-2.3

Note: 1 yd^3 = 0.7645 m^3

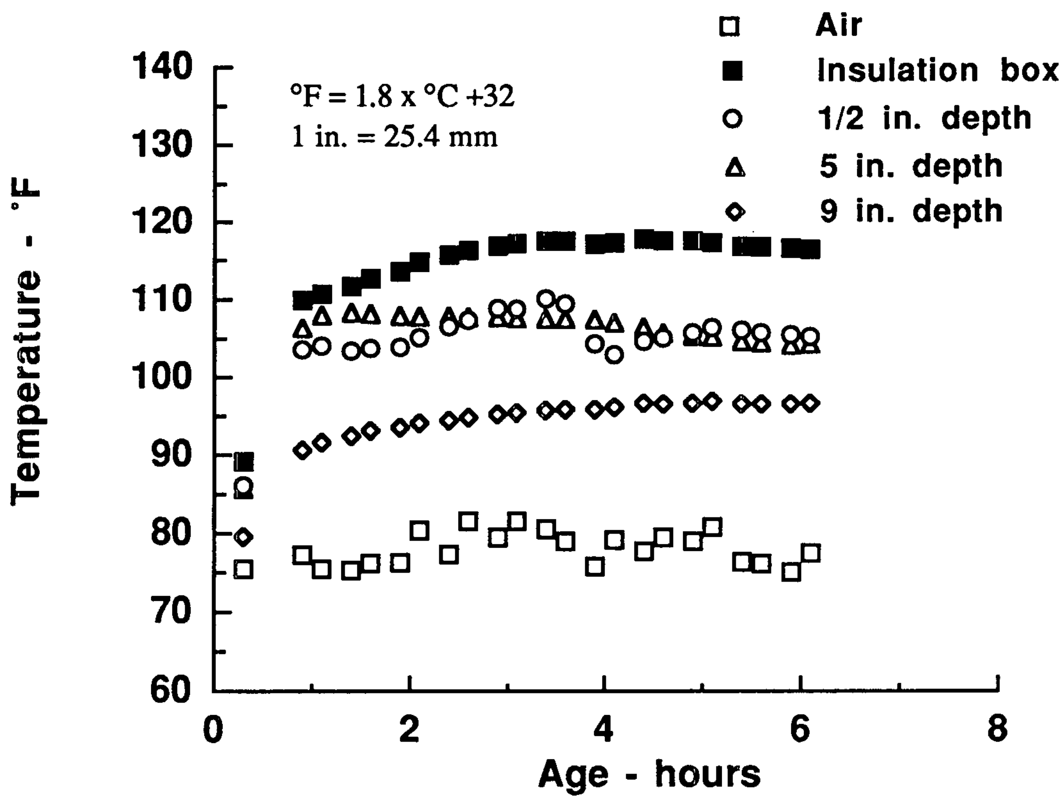

Figure A.29. Temperature profiles for PC 1 mix used on SR 2.

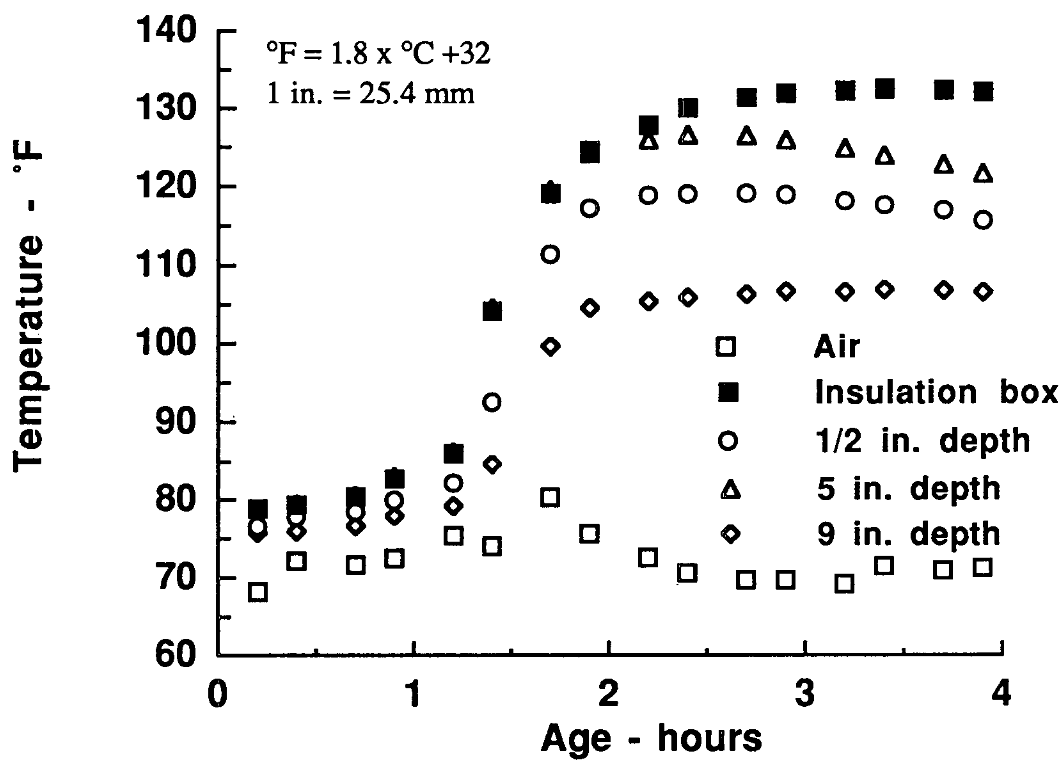

Figure A.30. Temperature profiles for RSC 1 mix used on SR 2.

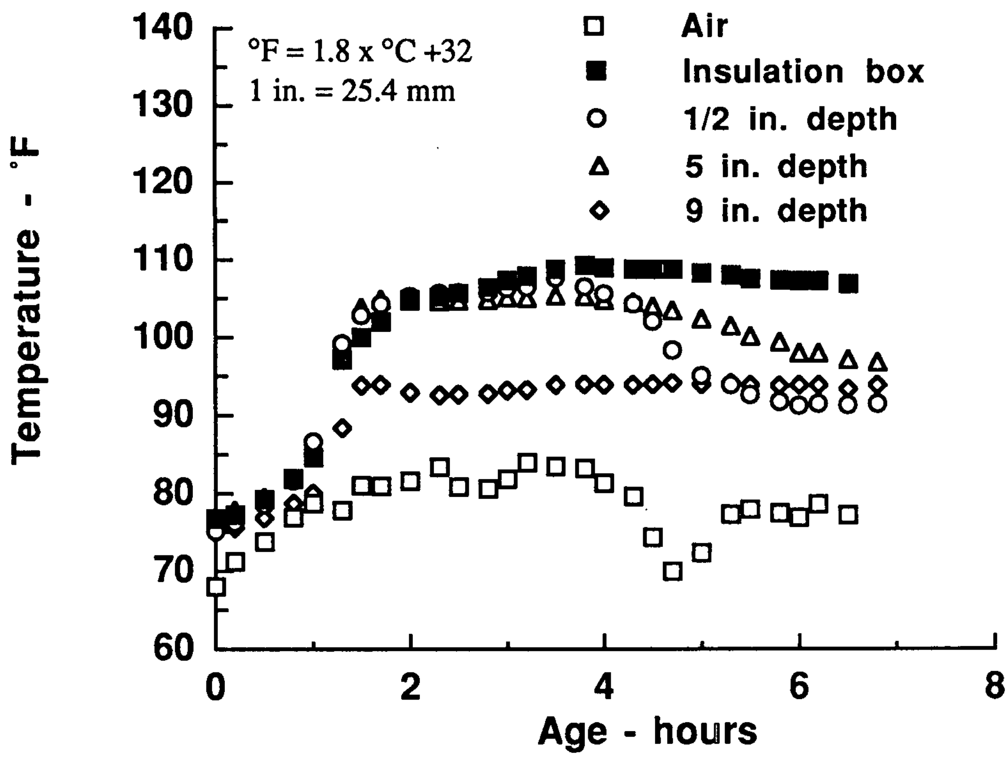

Figure A.31. Temperature profiles for PC 2 mix used on SR 2.

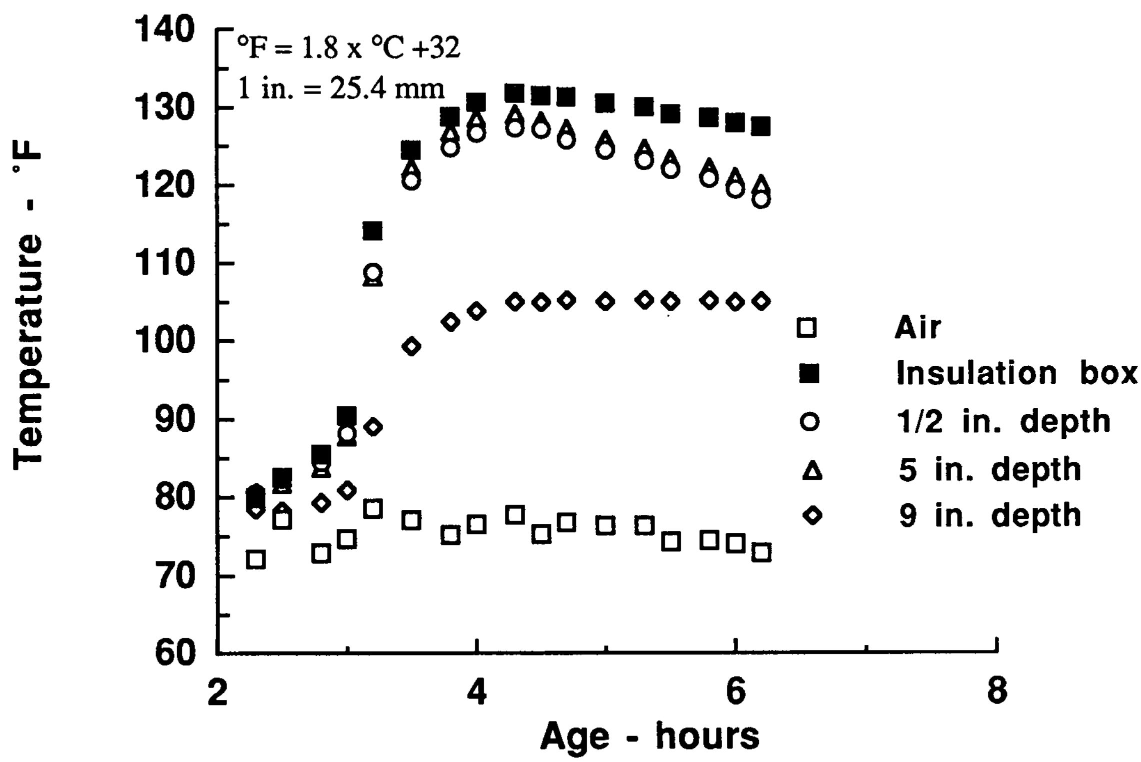

Figure A.32. Temperature profiles for RSC 2 mix used on SR 2.

ODOT mixes (Figures A.33 and A.34), also targeted for four to six hour opening times, generated considerably more heat than the proprietary products. VES mid-slab temperature peaks at 160°F (71°C) at 4-1/2 hours, while the ODOT mix peaks at nearly 165°F (74°C) after 4 hours. The heat peaks for VES and ODOT mixes used on SR 2 are about 20°F (11°C) greater than corresponding peaks for VES and GADOT mixes at the I-20 site. This is most likely attributable to the increased cement content at the SR 2 location, about 45 lb/yd^3 (27 kg/m^3) more Type III cement was used in the VES mix at SR 2 than in the mix at I-20. The ODOT mix, in addition to being formulated with Type III as opposed to Type I cement in the GADOT mix, had 148 lb/yd^3 (88 kg/m^3) more cement in the mix at the SR 2 site.

Finally, the temperature profiles for the mixes selected for 12 to 24 hour opening times are shown in Figures A.35 and A.36. Mid-slab temperature peaks at five hours for HES at about 145°F (63°C). Considering the heat generated and the rapid strength gains exhibited by this mix (see subsequent discussion), HES could more properly be classified as a four to six hour opening mix rather than the 12 to 24 hours chosen by the C-205 developers. Fast Track concrete is somewhat slower to reach its heat peak, approaching a temperature of 130°F (54°C) at mid-slab approximately 7-1/2 hours after placement. Studies of fast track pavement mixes in Iowa (Grove et.al. 1990) showed mid-slab temperatures on a 10.5 in. (267 mm) thick pavement rising to a peak also near 130°F (54°C) after 10 hours.

Strength gain prediction using maturity and pulse velocity techniques

Relationships between maturity and pulse velocity were established using laboratory trial batches prepared from material received from the SR 2 job. Correlations were carried out in an identical manner to the I-20 site, and predictive equations were developed. Temperature data acquired from the test sections were then used to developed plots of time versus both compressive strength and modulus of rupture. Predictions of early compressive strength for all eight mixes are shown in Figures A.37 through A.44. In addition to strengths predicted via maturity and pulse velocity, the strengths based on TMC cylinders and cylinders stored in the insulated curing box are also shown.

Strength gain versus time for the 2- to 4-hour opening mixes at SR 2 are shown in Figures A.37 and A.38. Results for the PC 1 mix (Figure A.37) are similar to those for the I-20 sections. Best predictor of core strength was the maturity function, although in this case pulse velocity and TMC cylinders were also fairly close to the actual value at opening. It should be noted, that similar to the laboratory trial batch experience with this mix, the PC 1 concrete failed to reach the desired value of 2,000 psi (13.8 MPa) within four hours after placement. In contrast, the RSC 1 mix (Figure A.38) gained strength much more rapidly. Both pulse velocity and maturity predict that this mix exceeds 2,000 psi (13.8 MPa) within two hours after placement. Cores taken at 2-1/2 hours exhibited a strength of 3230 psi (22.3 MPa). Prediction of core strength was excellent by all the NDT techniques employed at this section.

Figures A.39 and A.40 show strength gain for the proprietary cements used for four to six hour opening times. PC 2 gains strength relatively slowly, and again failed to meet the established criterion of 2,000 psi (13.8 MPa) within the six hour window. Due to impending severe weather conditions on this day, the core was removed three hours after placement

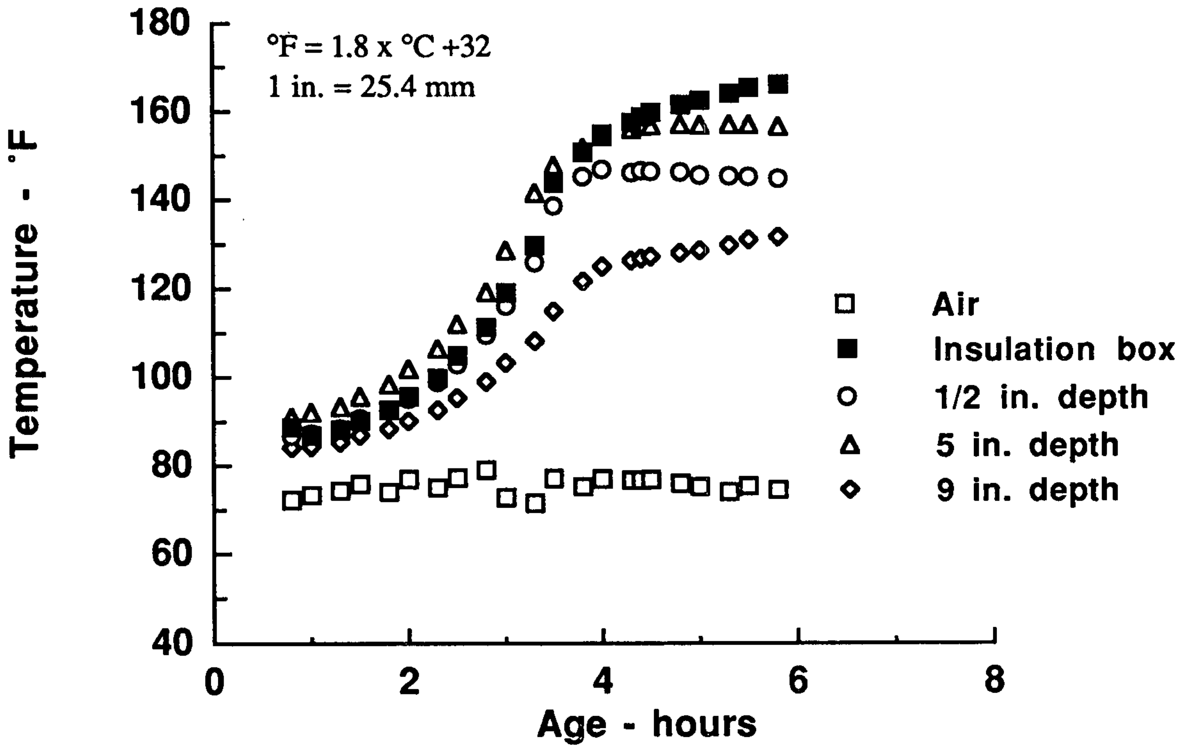

Figure A.33. Temperature profiles for VES mix used on SR 2.

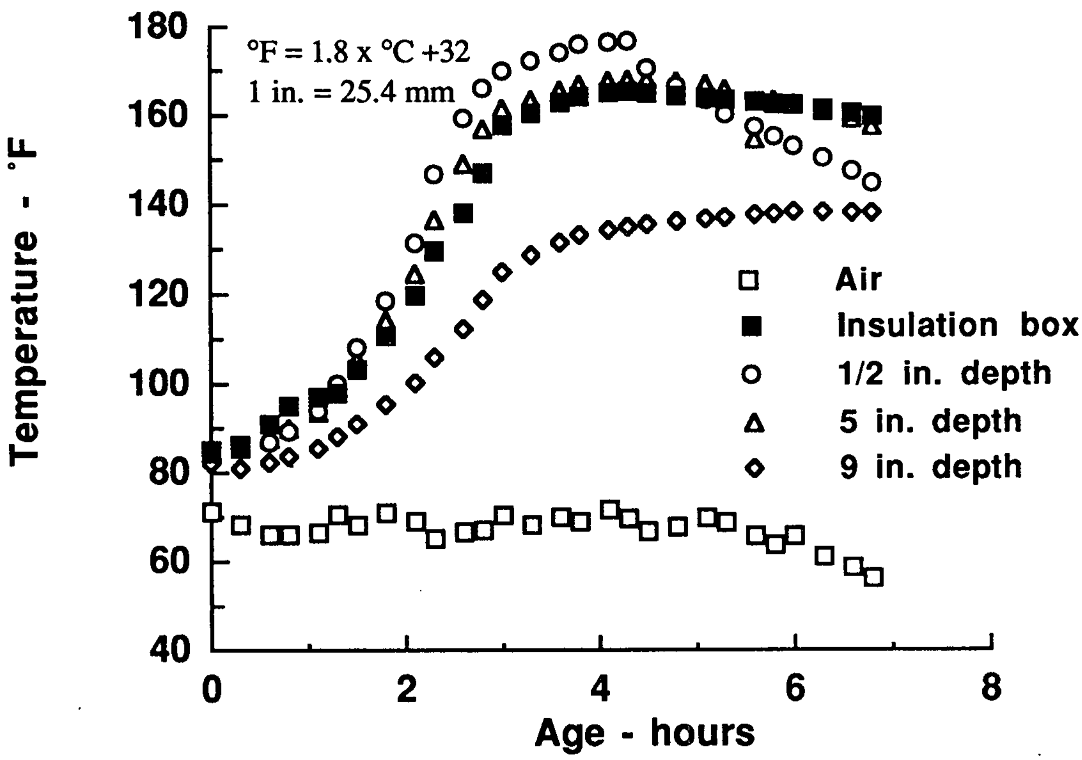

Figure A.34. Temperature profiles for ODOT mix used on SR 2.

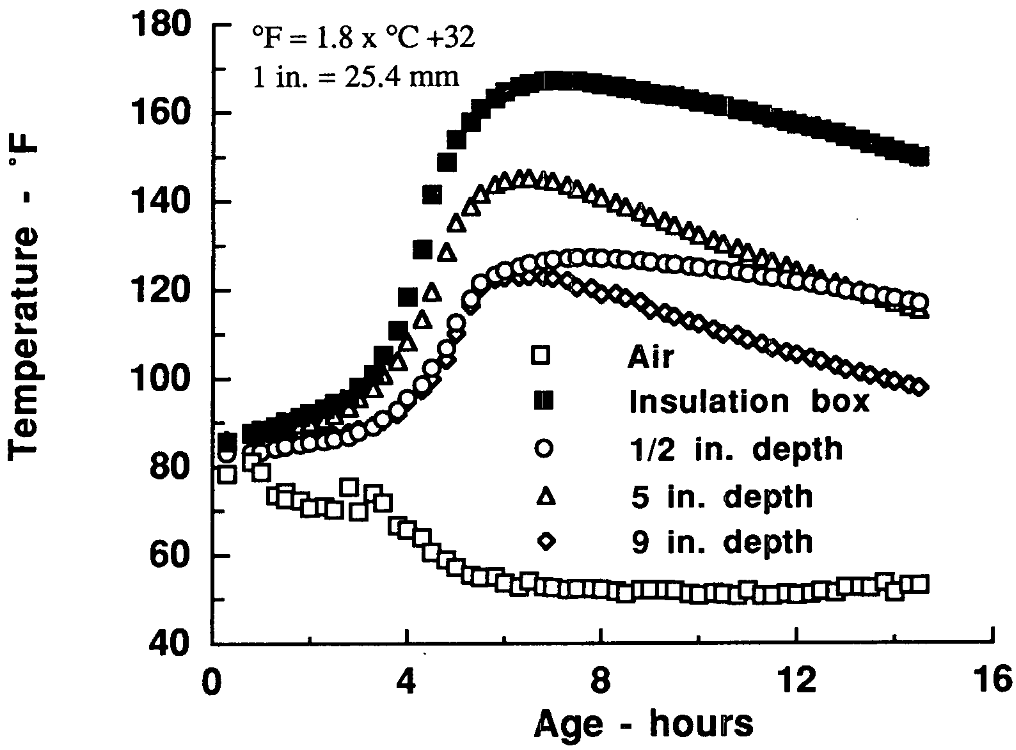

Figure A.35. Temperature profiles for HES mix used on SR 2.

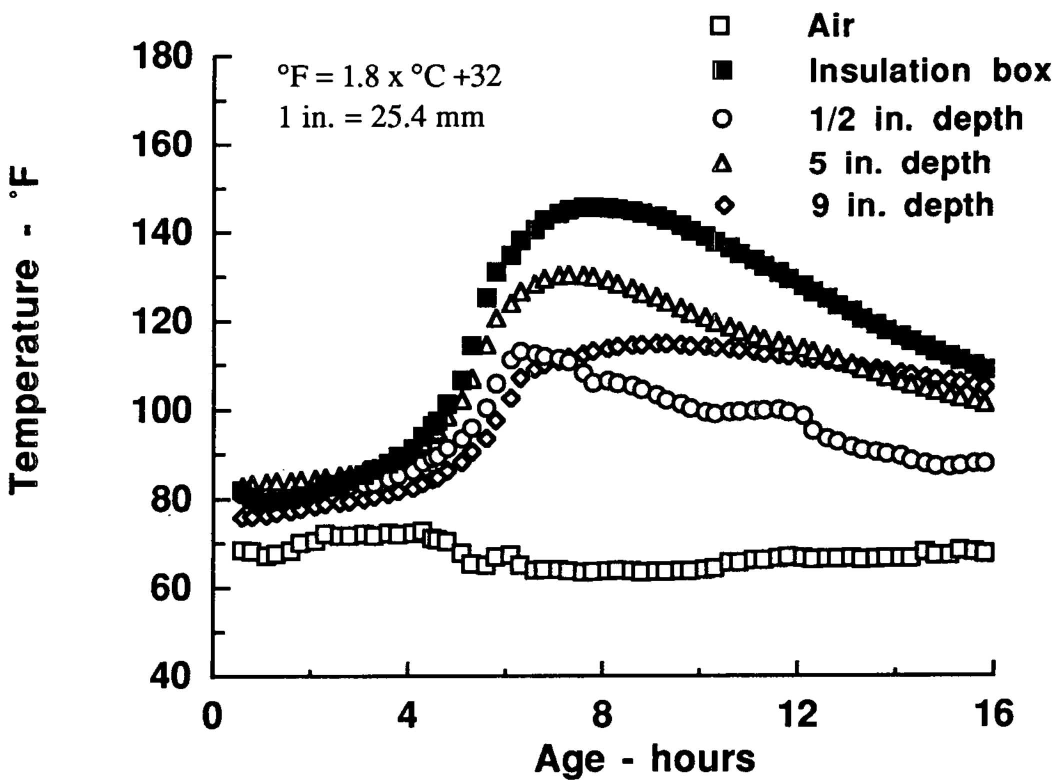

Figure A.36. Temperature profiles for Fast Track mix used on SR 2.

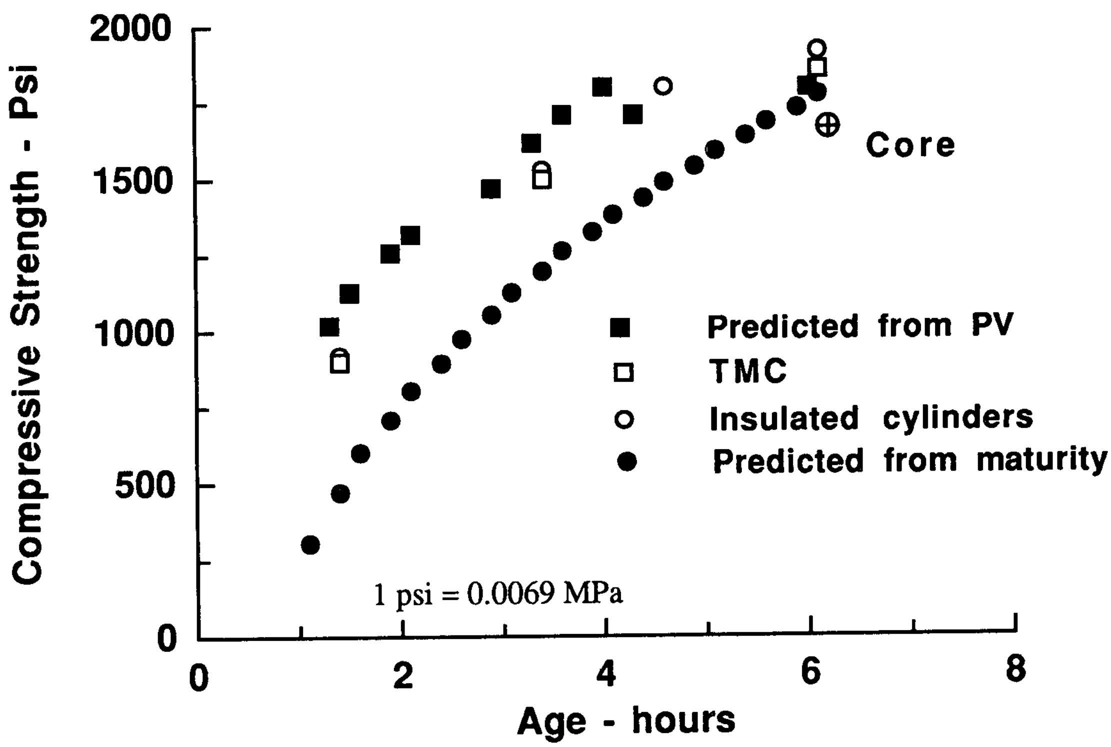

Figure A.37. Compressive strength predictions for PC 1 mix used on SR 2.

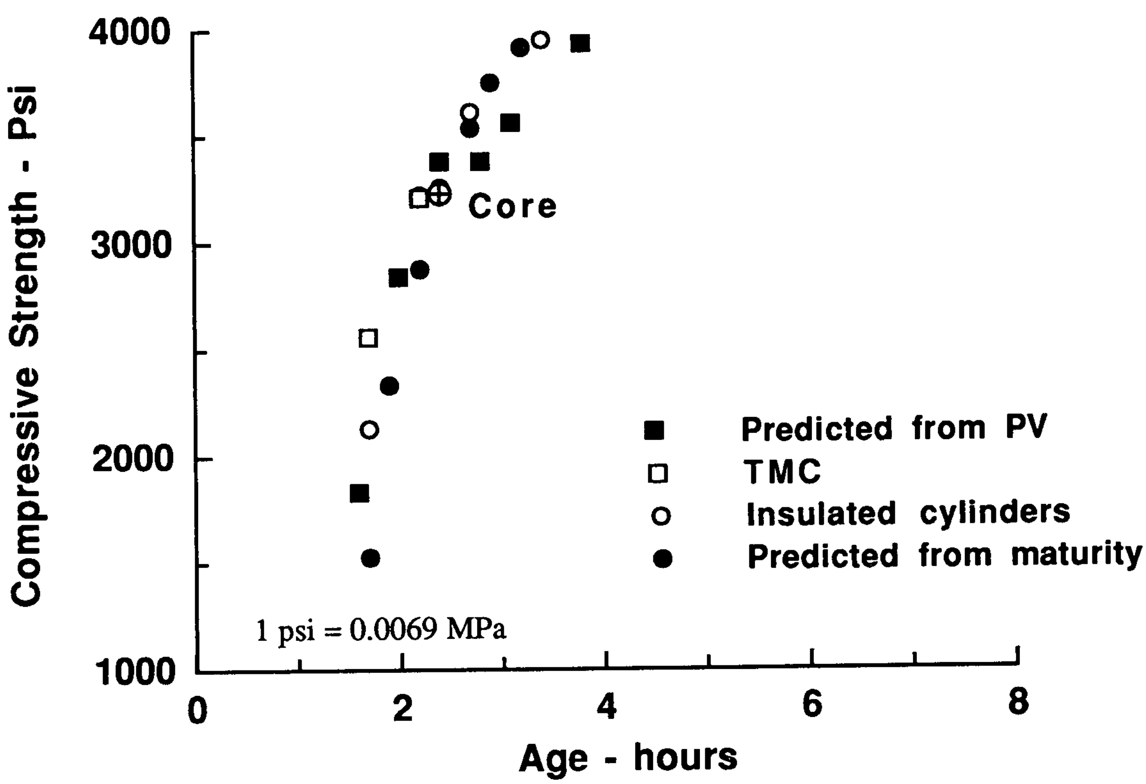

Figure A.38. Compressive strength predictions for RSC 1 mix used on SR 2.

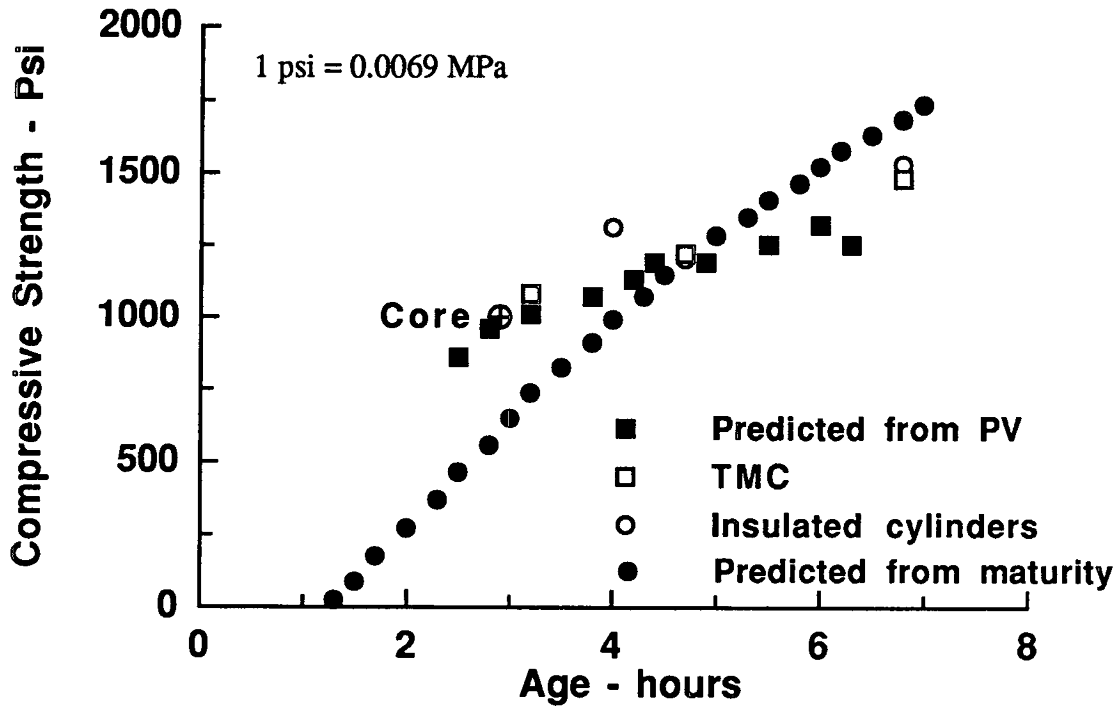

Figure A.39. Compressive strength predictions for PC 2 mix used on SR 2.

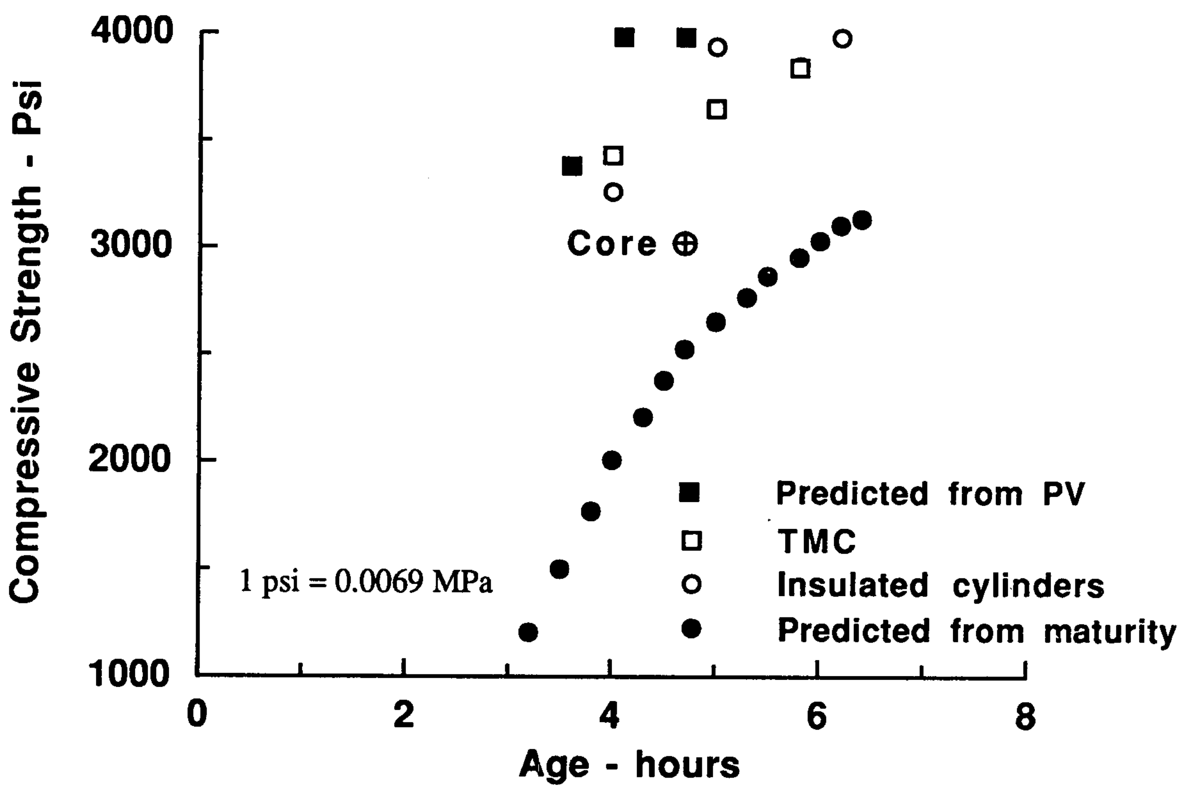

Figure A.40. Compressive strength predictions for RSC 2 mix used on SR 2.

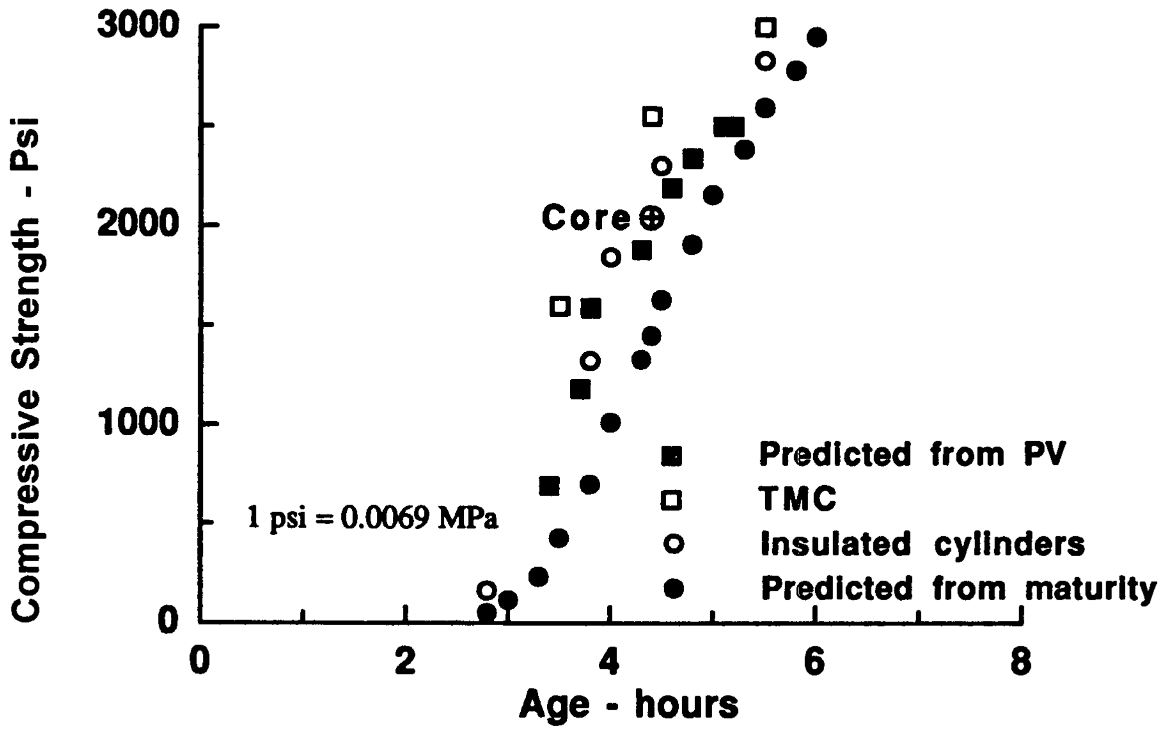

Figure A.41. Compressive strength predictions for VES mix used on SR 2.

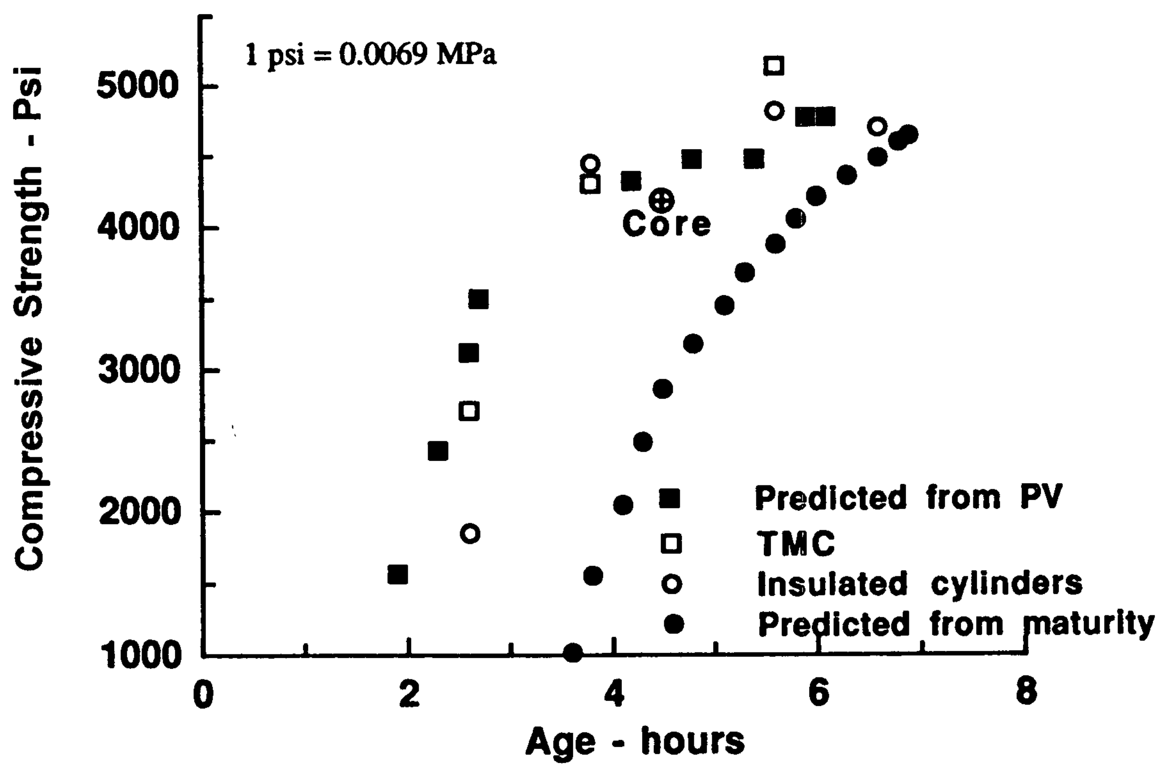

Figure 4.2. Compressive strength predictions for ODOT mix used on SR 2.

rather than at the nominal opening time. Prediction via pulse velocity, TMC and insulated cylinders was excellent. Prediction via maturity was conservative by approximately 350 psi (2.4 MPa). RSC 2 (Figure A.40) gained strength much more rapidly than its counterpart PC 2. Strengths at four hours were far in excess of 2,000 psi (13.8 MPa). At the time at which the core was taken (4-3/4 hours) pulse velocity, TMC and insulated cylinders overestimated the strength by 500 to 1,000 psi (3.5 to 6.9 MPa). The maturity function underestimated the strength by about 500 psi (3.5 MPa).

Strength gain for the two other 4- to 6-hour opening mixes, VES and ODOT, are shown in Figures A.41 and A.42, respectively. At 4 hours, the VES mix just meets the 2,000 psi (13.8 MPa) criterion. Core strength is well predicted by pulse velocity and insulated cylinders. TMC results are high, and maturity again is a conservative predictor by approximately 600 psi (4.1 MPa). The ODOT mix shows a more rapid gain of strength than VES. By 4 hours strengths are in the vicinity of 4,000 psi (25.6 MPa), with a measured core strength of 4,180 psi (28.9 MPa) at 4-1/2 hours. The maturity function is very conservative in this instance, predicting strengths lower by 700 psi (4.8 MPa). It should be noted that this mix exhibited much higher peak slab temperatures than the other sections (see Figure A.34) which may result in more rapid rates of strength gain than predicted by the maturity function.

The final two figures in this series (A.43 and A.44) show strength gain for the HES and Fast Track mixes. For its nominal classification as a "12- to 24-hour opening" mix, HES does quite well, exceeding 2,000 psi (13.8 MPa) within as little as 6 hours. Core strength measured at 6 3/4 hours was 2980 psi (20.6 MPa). All NDT techniques appeared to be fairly good predictors of in-place strength for this section. The Fast Track concrete section gained strength more slowly than the HES. Approximately six hours after placement a steady rain began, preventing any further pulse velocity measurements and cancelling the scheduled coring on this section. Testing of insulated cylinders and TMC cylinders continued, as these operations could be carried out sheltered from the weather. The large difference between strengths predicted from maturity versus TMC and insulated cylinders after eight hours for this mix (Figure A.44) reflects the cooling effect of the rain on the slab, while the well-insulated cylinders and the TMC molds continued to maintain high temperatures in these sets of specimens. Because of the lack of a core test specimen, a cylinder stored in the vicinity of the slab was used to obtain a measure of strength after 12 hours of test. This gave a value of 3500 psi (24.1 MPa).

Modulus of rupture (MOR) as predicted via the maturity function (Arrhenius) is shown in Figure A.45 for two to four hour opening mixes and Figure A.46 for the remaining SR 2 mixes. Mix RSC 1 gains strength most rapidly with time, exceeding a value of 300 psi (2 MPa) within only two hours after placement. The companion PC 1 mixture reaches the same value after four hours. For the four to six hour opening mixes both exceeded 300 psi (2 MPa) within five hours.

Development of flexural strengths for the remaining mixes is shown in Figure A.46. The two four to six hour opening mixes gain MOR very rapidly. Both VES and ODOT mixes reach 300 psi (2 MPa) within a period of four hours. This is a more rapid gain of flexural strength than seen for the companion mixes (PC 2 and RSC 2) prepared with proprietary rapid setting products. Finally, HES, a nominal "12- to 24-hour opening" mix, gains MOR very rapidly,

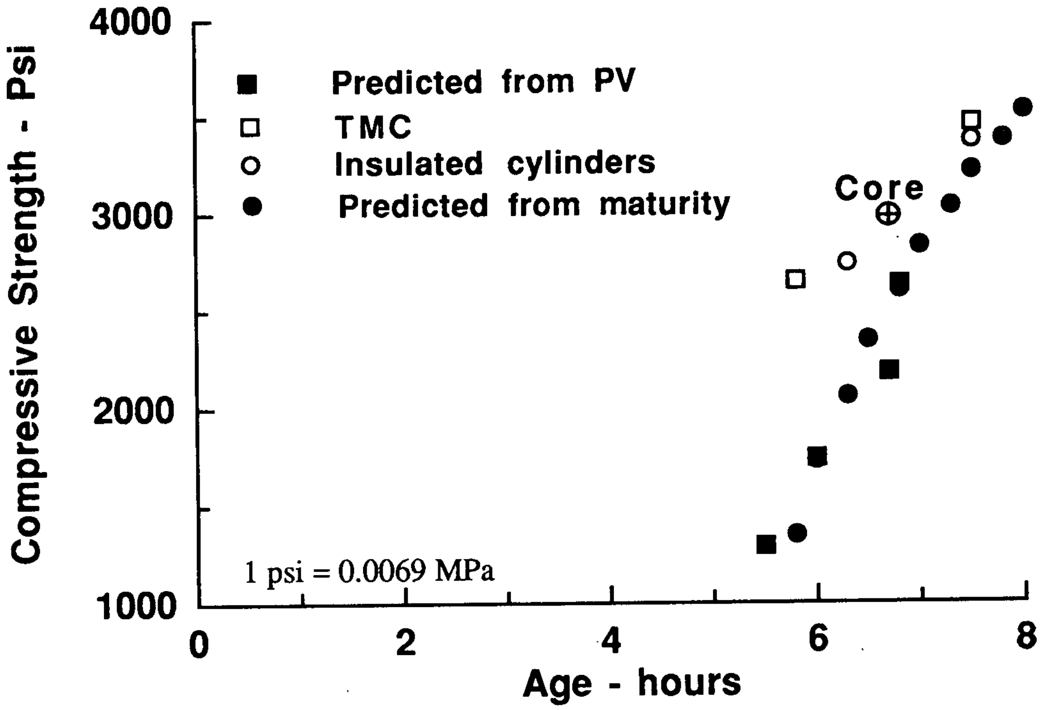

Figure A.43. Compressive strength predictions for HES mix used on SR 2.

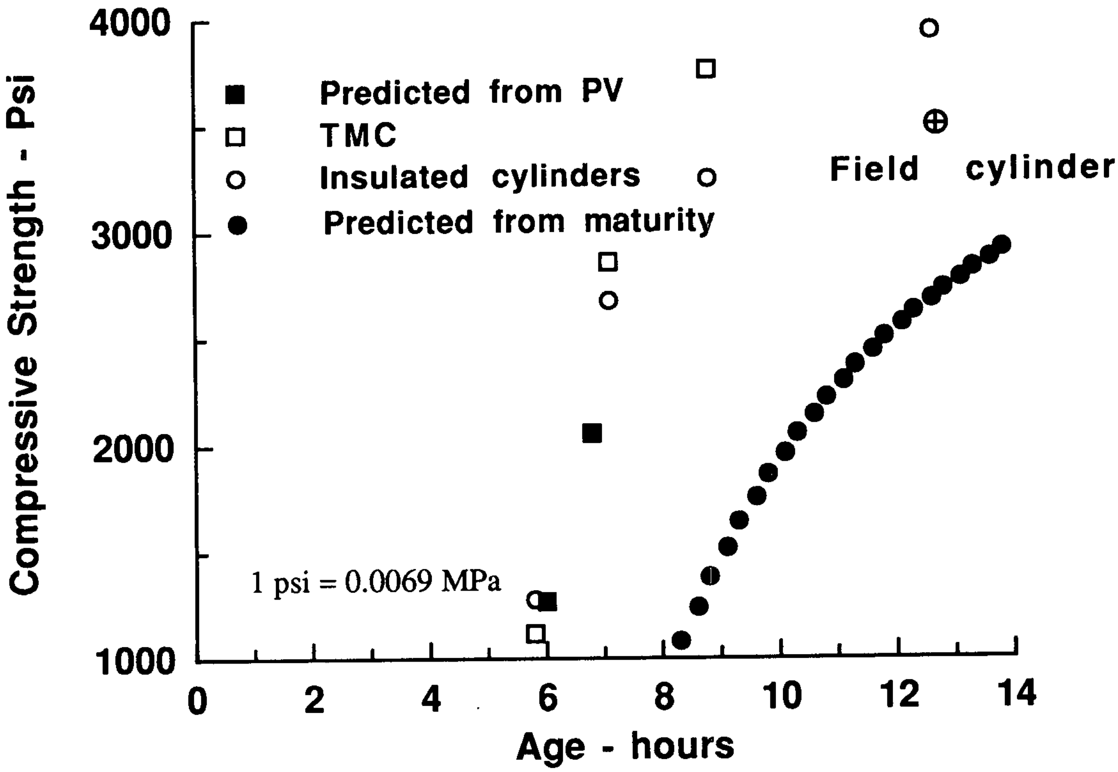

Figure A.44. Compressive strength predictions for Fast Track mix used on SR 2.

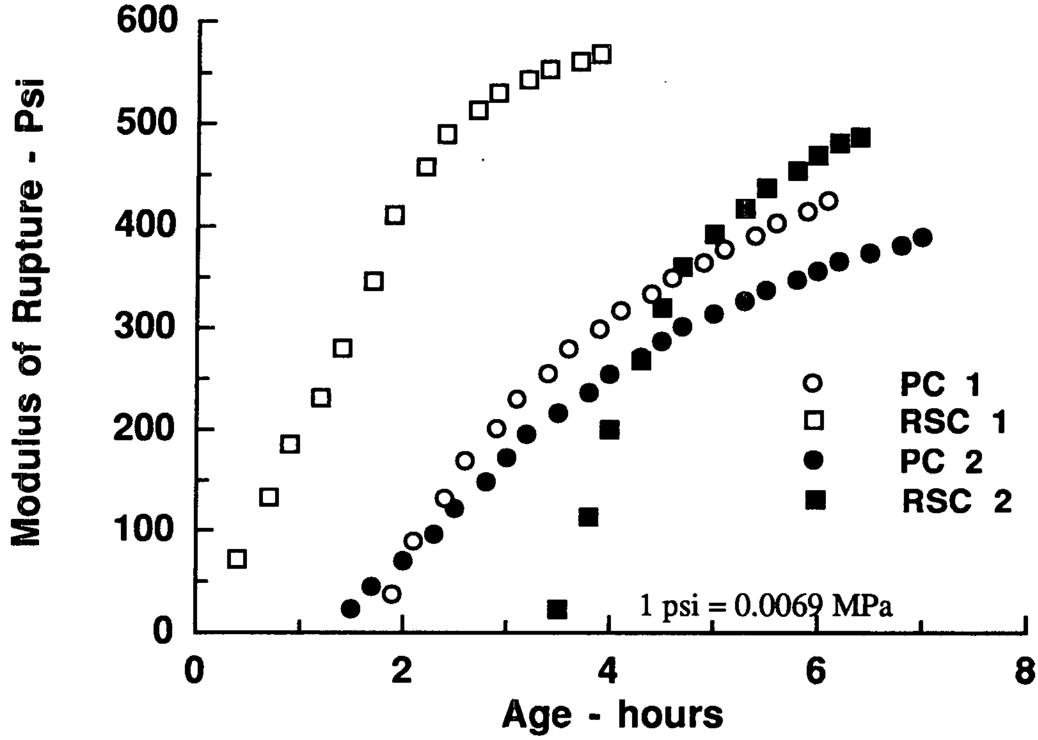

Figure A.45. Modulus of rupture predictions for 2 to 4 hour opening SR 2 mixes using maturity approach.

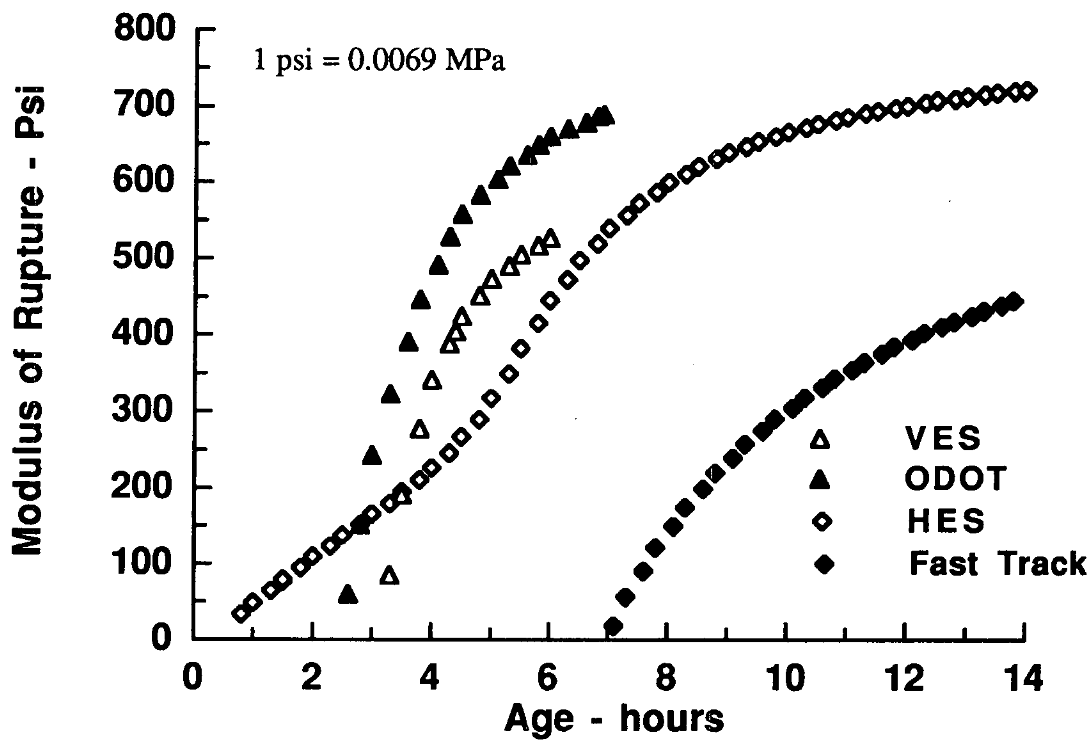

Figure A.46. Modulus of rupture predictions for remaining SR 2 mixes using maturity approach.

reaching over 400 psi (2.8 MPa) in only six hours. This mix, therefore, should more properly be classified in the four to six hour opening range. The Fast Track mix gains strength more slowly than HES, yet by 12 hours reaches strengths in excess of 350 psi (2.4 MPa).

Pulse velocity was also used to predict rate of gain of modulus of rupture with time. Figure A.47 shows predictions for the mixes prepared with proprietary products. Curves are distinctively "flatter" than those using the maturity function. For the two to four hour opening mixes, the time at which the mixes reach a MOR of 300 psi (2 MPa) is approximately the same as is predicted using the maturity function. For the four to six hour mixes PC 2 reaches a plateau at about 250 psi (1.7 MPa), but RSC 2 reaches a very high level of strength, over 500 psi (3.4 MPa) in only four hours. Relationships between MOR and pulse velocity shown in Figure A.48 are generally similar to those using the maturity approach. Pulse velocity indicates that the ODOT mix gains strength more rapidly than VES. This most likely reflects the use of the mid-slab measurement position for pulse velocity readings, versus the use of average temperature through the slab for maturity. The major difference between pulse velocity and maturity for these mixes occurs for the Fast Track mix. Pulse velocity predicts MOR to exceed 300 psi (2 MPa) at about six hours, while maturity (Figure A.46) indicated that it would take almost 10 hours to reach the same strength. Again, pulse velocity is measured at the warmest point on the slab, where strength gain would be most rapid.

Miniature disposable maturity meters ("COMA" meters) were positioned approximately 3 ft (0.9 m) in from the edge of each slab. The COMA meters extend approximately 2 in. (50 mm) into the surface of the concrete. Therefore, to compare against readings obtained using thermocouples it is necessary to calculate maturities (as equivalent age at 20°C) using temperatures measured near the top of the slab. For all slabs, temperatures measured at 0.5 in. (13 mm) and 5 in. (130 mm) were very close during the period of time from placement to nominal opening. Therefore, the temperatures measured at the 0.5 in (13 mm) level were used to compute equivalent age using the Arrhenius approach for comparison with COMA meter readings on the same concrete. Results are presented in Table A.28.

Table A.28. Comparison of maturities using COMA meters and Arrhenius calculations.

		Maturity (days at 68°F (20°C))	
Mix	**Time (hours)**	**COMA meter**	**Arrhenius**
PC 1	6.4	0.750	1.070
RSC 1	4.1	0.375	1.070
PC 2	7.0	0.625	0.590
RSC 2	6.4	0.375	0.640
VES	6.0	1.100	1.420
ODOT	3.3	0.750	0.775
HES	8.0	1.037	2.070
Fast Track	12.3	0.750	0.730

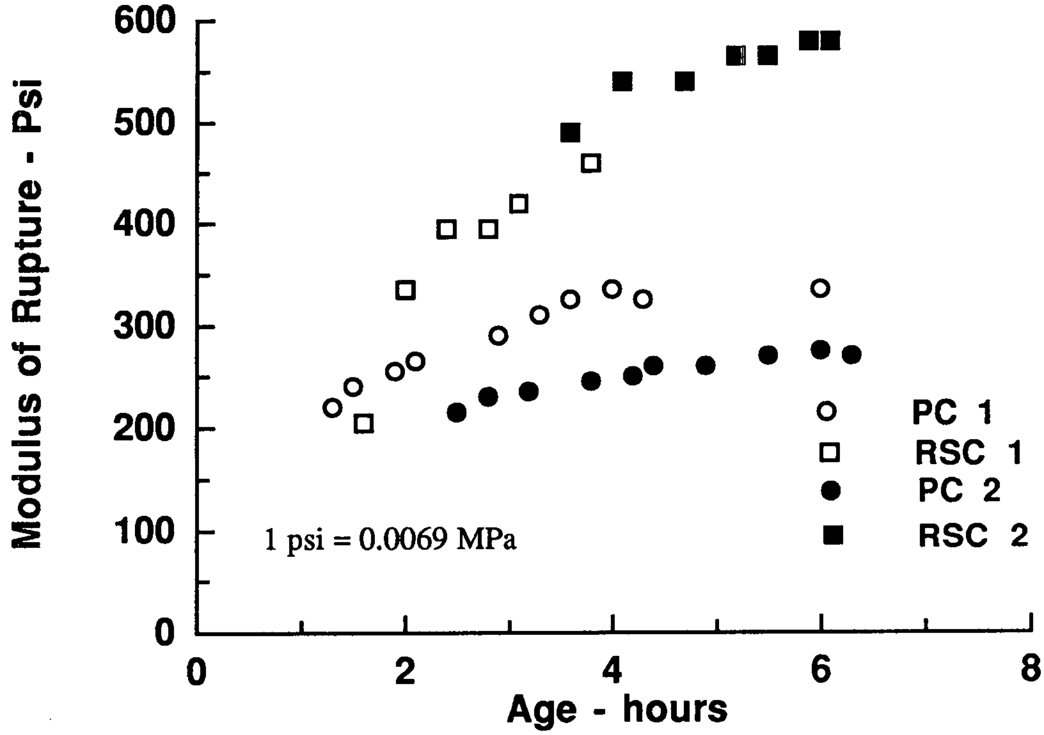

Figure A.47. Modulus of rupture predictions for 2 to 4 hour opening SR 2 mixes using pulse velocity.

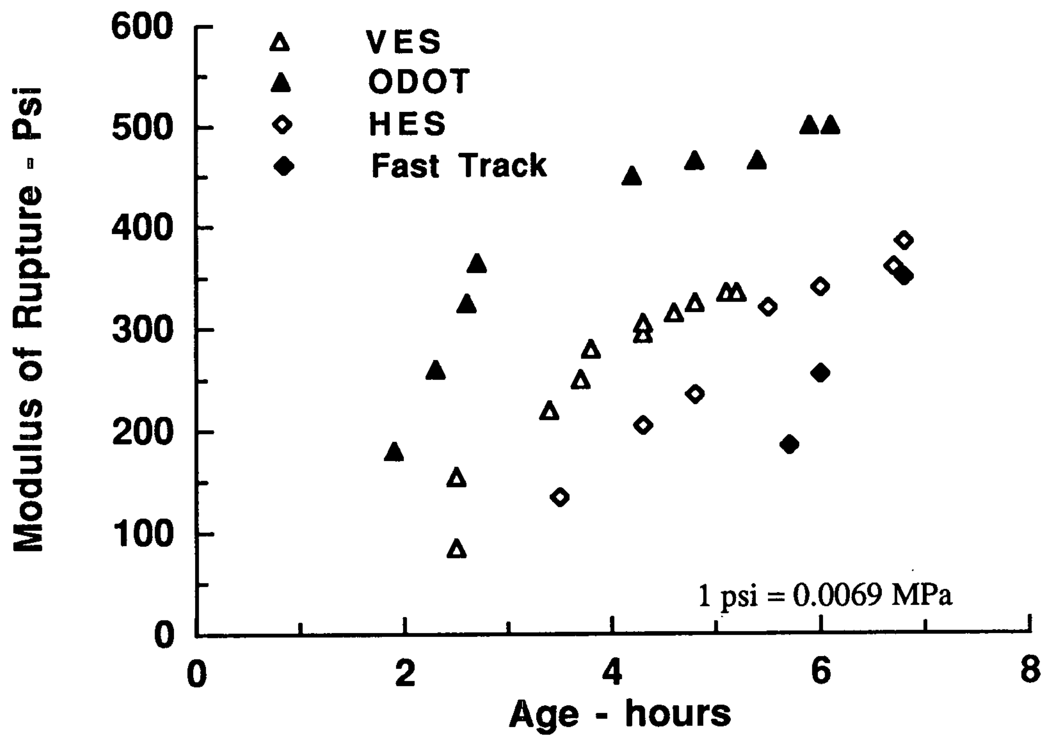

Figure A.48. Modulus of rupture predictions for remaining SR 2 mixes using pulse velocity.

In a few cases (PC 2, ODOT, and Fast Track), equivalent ages for COMA meters and as calculated from thermocouple measurements were within 5% of each other. However, for the other mixes wide differences occurred. For these mixes, equivalent ages as measured using the COMA meters were from 30 to 60% lower at time of measurement than were the maturities as calculated from measured temperatures in the slabs. Literature supplied with the embeddable meters stated than an activation energy of 40 kJ/mol was assumed in construction of the scale on the device. Activation energies as actually measured for the mixes ranged from 19 to 62 kJ/mol. However, when maturities were recomputed using the 40 kJ/mol figure, agreement was still no better between the COMA meters and the calculated maturities. The disposable meters, therefore, do not appear to be an accurate means of measuring maturities in such early opening applications.

Laboratory Testing of Specimens Cast in the Field

As at the previous field location, laboratory testing was carried out to develop performance data on materials used at each repair test section. Specimens were prepared in the field and cast from mixes used to place the repairs. Specimens were cured as indicated in the following descriptions and transported back to the laboratory for further curing and testing. All specimens scheduled for testing at one day age were tested using a field compression testing machine.

Compressive strength testing

Concrete was cast in rigid 4 × 8 in. (100 × 200 mm) plastic cylinder molds at the job site and consolidated using an external vibrating table. Test specimens were cured and transported in a manner identical to those specimens prepared at the I-20 site. Tests were conducted at 1, 3, 7, 28, and 90 days of age. Duplicate cylinders were tested at each age. Replicate sets of cylinders were tested at 28 days. Compressive strength testing was done in accordance with AASHTO T 22.

Results for the proprietary cement mixes are presented in Figure A.49. Mixtures prepared with Pyrament cement (PC 1 and PC 2) exhibit a rapid rate of strength gain over the period, indicating significant hydration potential is left in the cement after its initial early rapid strength gain. Ultimate strengths for PC 1 and PC 2 mixes are over 10,000 psi (70 MPa). The concretes produced with Rapid Set cement, however, appear to have little potential for increased strength gain after the initial rapid strength rise during the first few hours after placement. Final strengths are in the 5,000 to 6,000 psi (34 to 41 MPa) range for these mixes. Replicate specimens agreed well with initial batches for PC1 and PC2 mixes, but strengths of replicates for RSC and RSC 2 were lower than for the initial batches. This may be partly explained by the higher air contents in the replicate batches prepared with Rapid Set cement, which were used for the durability testing. The much lower strength of the RSC 2 replicate may be further explained by the higher water content in this mix (see Table A.27).

Strength gain over time for the remaining mixes is shown in Figure A.50. Most of the mixes show a continuing increase in strength over the time period. Highest strengths are obtained

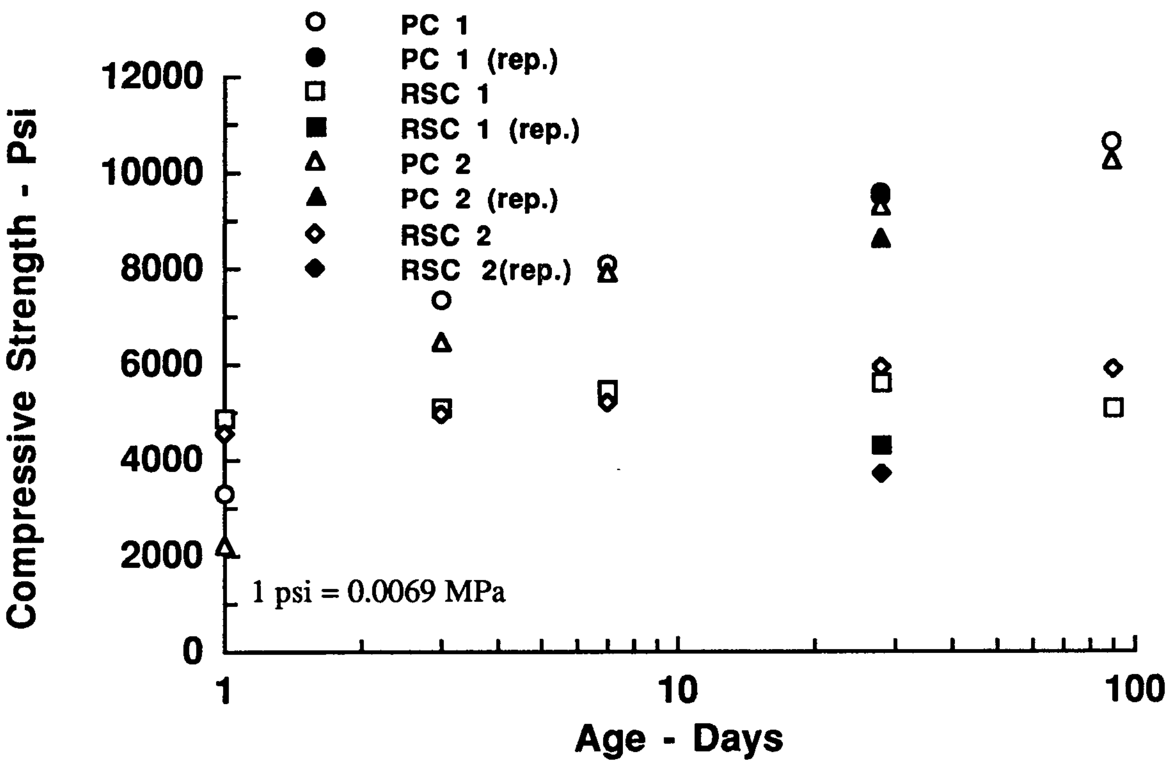

Figure A.49. Compressive strength development for cylinders cast from proprietary cement mixes on SR 2.

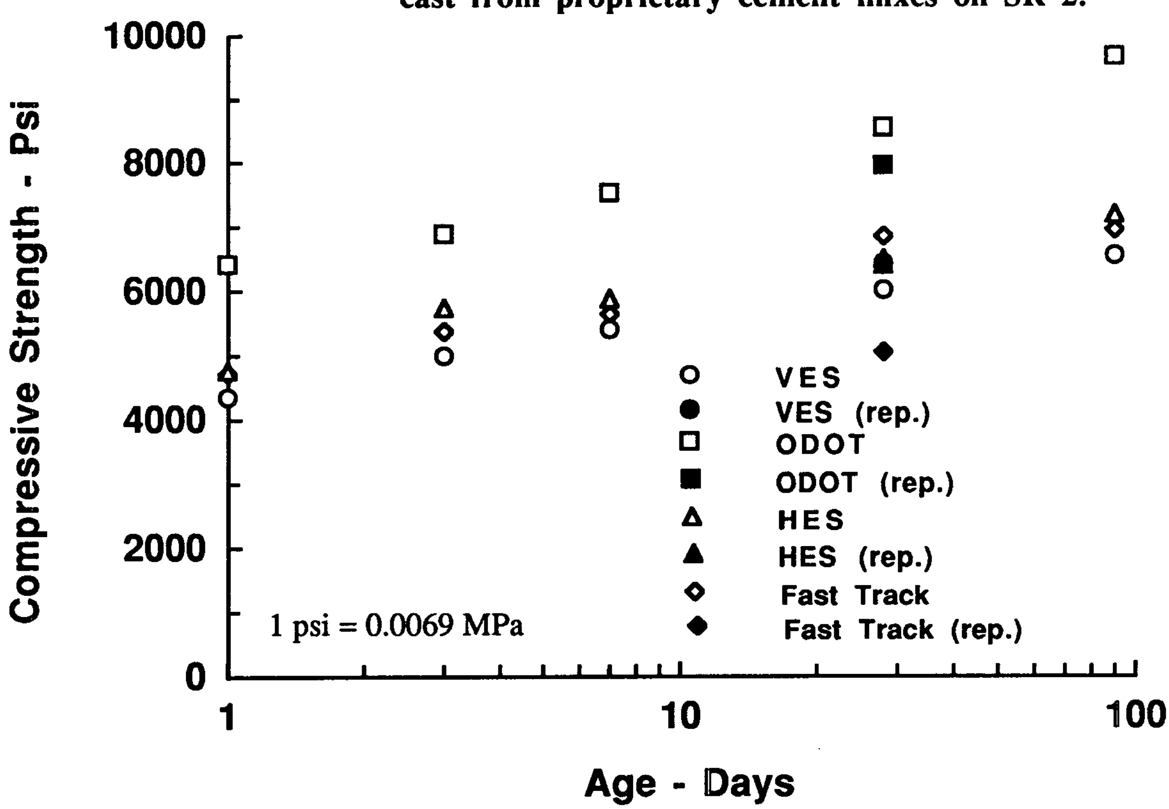

Figure A.50. Compressive strength development for cylinders cast from remaining mixes on SR 2.

for the ODOT mix, reaching 9,700 psi (66.9 MPa) at 90 days. The remaining three mixes (VES, HES, and Fast Track) are remarkably similar in their strength profile over time. Replicate agreement is good, with the exception of the Fast Track mix, where the replicate tested 1,800 psi (12.4 MPa) below the initial batch. This can be primarily assigned to the very high air content (10.0%) measured in the replicate batch.

Split tensile strength testing

Duplicate cylinders were cast from the same concrete batches as those used for compressive strength cylinders. Standard cured specimens were fabricated, transported, and cured identically to the compressive strength specimens. Split tensile strength was tested at the same intervals as for compressive strength in accordance with AASHTO T 198. Splitting tensile strength test data is summarized in Figure A.51 for the proprietary products mixes. Behavior is much the same as for compressive strengths. Highest split tensile strengths are exhibited by the PC 1 mix. Tensile strength development for the Rapid Set mixes is essentially flat across the time period. The remaining mixes (Figure A.52) show modest gains in split tensile strengths across the period monitored.

Modulus of elasticity

Static elastic modulus of elasticity was measured at 7 and 28 days on duplicate 4 × 8 in. (100 × 200 mm) cylinders in accordance with ASTM C 469 procedures (Table A.29).

Table A.29. Elastic modulus of specimens cast from SR 2 concretes.

Mix	Age (days)	Elastic modulus (million psi)	Compressive strength (psi)
PC 1	7	4.15	8,090
	28	4.25	9,490
RSC 1	7	2.85	5,460
	28	4.25	5,610
PC 2	7	3.85	7,920
	28	4.10	9,290
RSC 2	7	2.95	5,210
	28	3.20	5,940
VES	7	2.40	5,420
	28	3.85	6,030
ODOT	7	3.10	7,540
	28	4.45	8,570
HES	7	2.85	5,890
	28	3.10	6,510
Fast Track	7	2.10	5,650
	28	3.15	6,860

Note: 1 million psi = 6.896 GPa

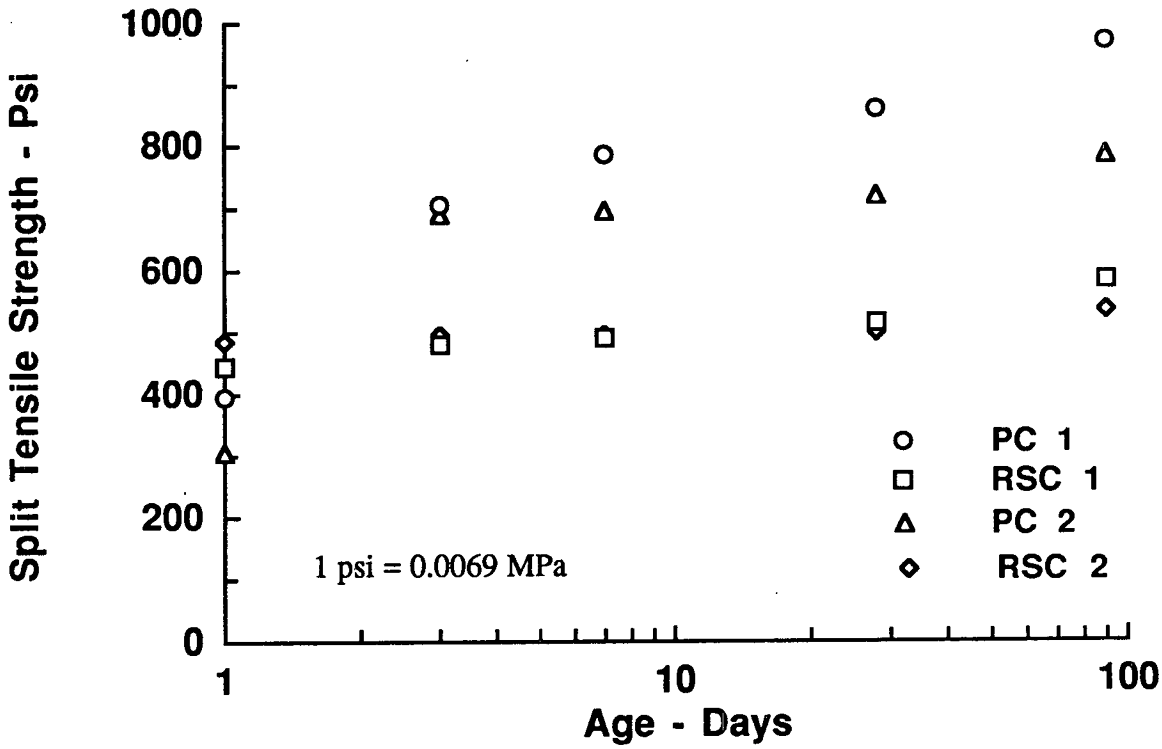

Figure A.51. Split tensile strength development for cylinders cast from proprietary cement mixes on SR 2.

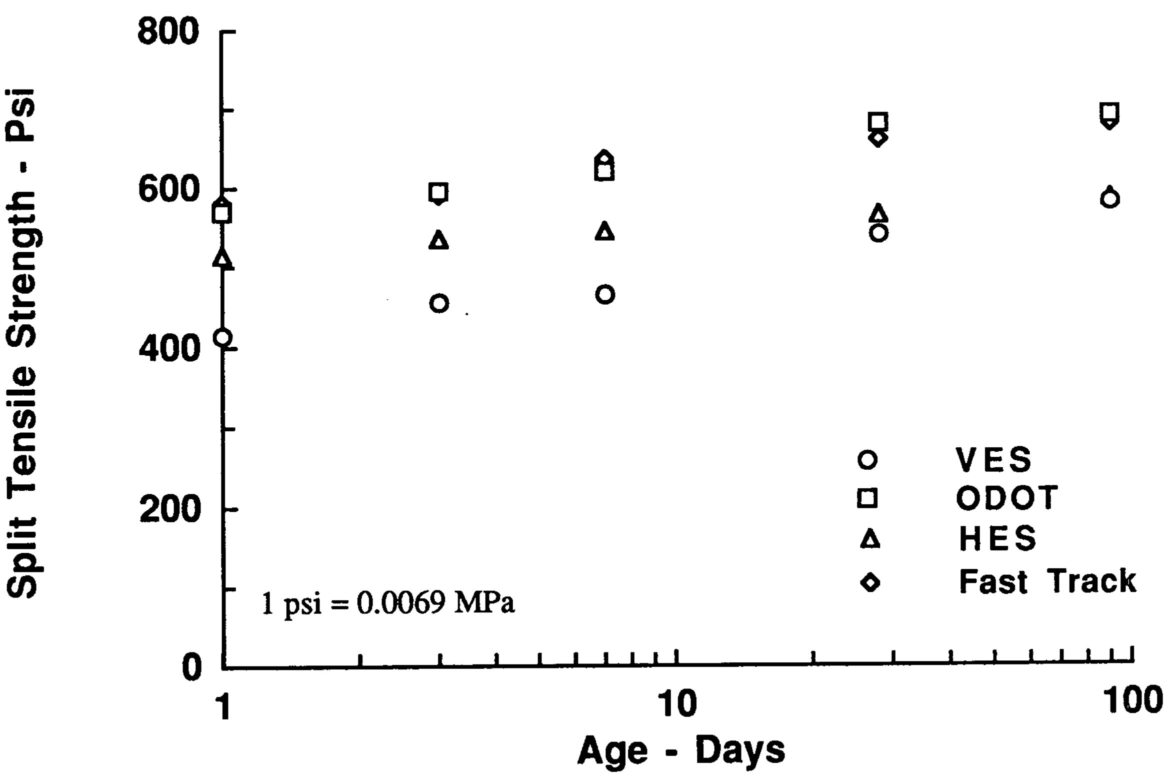

Figure A.52. Split tensile strength development for cylinders cast from remaining mixes on SR 2.

At seven days of age elastic moduli range from a low of 2.10 × 10^6 psi (14.5 GPa) for Fast Track to a high of 4.15 × 10^6 (28.6 GPa) for PC 1. At 28 days elastic moduli range from a low of 3.10 × 10^6 psi (21.4 GPa) for HES to a high of 4.45 × 10^6 psi (30.7 GPa) for ODOT. There is a general association between compressive strengths and elastic moduli for most of the mixes, the mixes exhibiting highest strengths (i.e. PC 1, PC 2, and ODOT) also showing higher moduli values. However, at 28 days there are one or two exceptions. For instance, mix RSC 1 exhibits the lowest 28 day strength at 5,610 psi (38.7 MPa) yet its modulus is 4.25 million psi (29.3 GPa), one of the highest at this age.

Resistance to freezing and thawing

Prismatic test specimens 3 × 4 × 16 in. (75 × 100 × 1025 mm) were cast in triplicate, consolidated by rodding, and covered with wet burlap and polyethylene sheeting. They were allowed to stand at ambient temperatures until achieving final set. They were then placed in containers filled with saturated limewater and transported to a controlled temperature environment for the remainder of the field placements. They were then transported back to the laboratory and maintained under saturated limewater until 14 days of age. Results using the modified AASHTO T 161 B procedure described previously are given in Table A.30. With the exception of HES and RSC 2 mixes, most of the concretes tested performed rather poorly. Durability factors were very low, and expansions for many of the concretes exceeded 0.1% at 300 cycles. This performance can only partly be explained by air contents and air void system parameters (Table A.31). The lack of air entrainment in mixes PC1 and PC 2 would appear to explain their poor behavior. While PC 2 had a total air content of 6.0%, most of this was entrapped air, as reflected in the low specific surface and high spacing factor. The mixes that performed well (HES and RSC 2) had more than adequate air content and very low spacing factors. Mix RSC 1 had a low air content and high spacing factor, which would help to explain its poor performance. The ODOT mix, while having an acceptable air void system, did have a somewhat low air content, and did contain calcium chloride, which, in common with the GADOT mix used on I-20, appears to degrade freeze-thaw performance.

Table A.30. Resistance to freezing and thawing of SR 2 repair concretes after 300 cycles of test (modified AASHTO T 161B).

Mix	Durability Factor (DF)	Expansion (%)	Mass Change (%)
PC 1	15	0.19	+0.82
RSC 1	64	0.26	-3.27
PC 2	14	0.15	-1.78
RSC 2	101	-0.02	-0.04
VES	15	0.13	-1.77
ODOT	13	0.09	+0.86
HES	80	0.07	-0.65
Fast Track	25	0.09	+0.46

The two mixes whose performance cannot be explained by the air void systems are Fast Track and VES. Both performed extremely well in their I-20 counterparts. At SR 2 however, while air contents for these mixes were higher than for the I-20 versions of the same designs, they did not pass the freeze-thaw testing. While performing the linear traverse analyses for air content, varying degrees of microcracking were observed in most of the companion specimens (with the exception of RSC 1 and RSC 2 and HES). This observation may also help to explain the poor freeze-thaw resistance of some of these mixes, as freezing of water within the microcracks may exert additional stresses on the specimens during the freeze-thaw cycling.

Freeze-thaw testing was also carried out on duplicate 4 × 8 in. (100 × 200 mm) cores removed from the test sections just prior to opening. An exception was the Fast Track section, where rain interrupted coring and cores for freeze-thaw testing could not be obtained. A set of cores was obtained from the Fast Track section during the condition survey carried out two months after installation. Cores were cured and conditioned for testing in a manner identical to the beam specimens, with the exception that testing was initiated at an age of 28 days. As for the cores removed from I-20, testing was carried out by freezing and thawing in water (AASHTO T 161 A). After approximately six weeks of test, a malfunction was noted in the freezer, which had resulted in accumulation of only 48 cycles in the specimens to that point in time. The specimens were then transferred to another freezer, where cycles then began to accumulate at a rate of about five cycles per day. Data through 300 cycles are reported. Results are shown in Table A.32.

Table A.31. Air void parameters for SR 2 repair concretes.

Mix	Air Content (%)	Voids Per Inch	Specific Surface (in^2/in^3)	Spacing Factor (in.)
PC 1	3.4	3.80	443	0.0122
RSC 1	3.2	3.51	441	0.0127
PC 2	6.0	6.26	420	0.0096
RSC 2	7.5	12.02	638	0.0050
VES	10.0	49.84	1991	0.0012
ODOT	4.3	11.21	1034	0.0047
HES	11.4	41.19	1441	0.0015
Fast Track	7.2	27.22	1513	0.0022

Note: 1 void per in. = 0.0394 void per mm; 1 in^2/in^3 = 0.0394 mm^2/mm^3; 1 in. = 25.4 mm

Performance of the core specimens was better than for the corresponding beams cast from the concretes and tested using the modified AASHTO T 161B procedure. As in the beam testing, mixes RSC 2 and HES performed the best, although mass loss was greater for the core specimens. The cores taken from the Fast Track section also performed quite well, although these had been in-place prior to test for a considerably longer period of time than cores from the other sections. While the remainder of the cores did not perform as well as these, accrued damage during test was nowhere near as great as experienced for the beam specimens. As the cores experienced only a limited number of cycles during the first six weeks of test, it is possible that increased strength and further hydration of the cores over that time improved their freeze-thaw durability over the prisms which experienced rapid cycling starting at 14 days of age. The increased severity of the modified Procedure B, in addition, may also contribute to the differences observed between prism and core testing.

Table A.32. Resistance to freezing and thawing of SR 2 repair concretes after indicated cycles of AASHTO T 161 A.

Mix	DF	Mass Change (%)
PC 1	90	+0.41
RSC 1	87	-4.67
PC 2	72	-0.56
RSC 2	100	-1.09
VES	70	-12.99
ODOT	88	-4.19
HES	94	-4.24
Fast Track	94	+0.28

Appendix B

Early Opening Criteria for Full-Depth Pavement Repairs

Introduction

This experiment was conducted to evaluate various materials and processes for the early opening of full-depth concrete repairs under actual field conditions. The purposes of this experiment were to demonstrate and validate the technologies that allow the early opening of concrete repairs to traffic and also to document the information needed to apply this technology.

The scope of field testing and validation performed under this project not only included the evaluation of material properties and testing procedures, but also the collection of performance data. As such, the full-depth repair experiment is a multi-purpose experiment designed to address all aspects of field evaluation. The principal elements of the experiment were:

- Evaluation of 11 different concrete mixes that would allow the opening of the concrete pavement repairs to traffic from 4 hours to 24 hours.

- Evaluation of various testing procedures for determining the in-place concrete strength, including pulse velocity, maturity, and temperature-matched curing.

- Assessment of fatigue damage attributable to early opening of concrete repairs and determination of the minimum strength needed for early opening to ensure adequate long-term performance.

The last item on the above list is of critical importance for early opening projects. While several highway agencies specify a minimum strength for opening concrete pavement repairs to traffic, information on the effects of early opening on repair performance is scarce.

This experiment was conducted to provide answers to some of the key questions facing design engineers in planning and conducting an early opening project. Although the testing platform for this experiment was full-depth repairs, the results of this experiment should be generally applicable to early opening of other highway concrete applications, such as partial-depth repairs, reconstruction, and concrete overlays.

Research Plan

Objective and Scope

The objective of this experiment is to evaluate the materials and processes for the early opening of concrete repairs to traffic. The performance of materials that will allow opening of full-depth repairs of concrete pavement to traffic in 2 to 4 hours, 4 to 6 hours, and 12 to 24 hours of placement was evaluated. The effects of early opening on the performance of concrete repairs made with these materials was investigated. Various methods of determining in-place strength of concrete were also evaluated.

One of the goals of this experiment was to establish guidelines for the minimum strength required at opening time to ensure adequate performance of concrete pavement repairs. Since it is not possible to obtain the long-term performance data needed to establish such guidelines within the time frame of this project, long-term monitoring of the experimental sections is considered an essential part of the experimental design. The monitoring should consist of annual distress surveys and traffic monitoring. Within the time frame of this project, the guidelines for early opening were developed based on analytical results. When sufficient amount of performance data becomes available, it should be possible to validate these guidelines. The performance data would allow a comparison of analytical predictions to actual performance.

The scope of Fast Track paving, or "Accelerated Rigid Paving Techniques", is not limited to simply using rapid-setting mixes, but includes any and all techniques that would reduce the duration of lane closures (Ferragut 1990). This means that innovative repair design, materials, testing procedures, construction techniques, as well as work planning, are all topics covered under Fast Track paving; however, the scope of this experiment was limited to the investigation of repair materials and testing procedures only. The only exception made was at the Georgia site, where the effect of slab length on repair performance was also investigated. At both test sites, the host state's standard design and construction procedures were used.

Experimental Design

Experimental Factors

The main variables of this experiment were material type, concrete strength at opening, and repair length. These are important factors for early opening considerations that influence how fast concrete repairs may be opened to traffic without compromising long-term performance. The material type determines the rate of strength development and the ultimate strength the repair will achieve. The strength at opening is the most sensitive parameter affecting the amount of fatigue damage the repair will experience during the first few hours after opening to traffic. The repair length affects the critical stresses that develop in the repair for fatigue; thus, the repair length is an important factor affecting the strength needed for opening.

Careful consideration was given to the other factors that are known to affect the performance of concrete repairs. These factors include environmental factors, repair techniques, repair design, and condition of the surrounding pavement. Rigorous treatment of all of the factors affecting the performance of full-depth repairs was not possible because of resource limitations; therefore, the scope of the experiment was appropriately limited.

While it would have been desirable to repeat the experiment in all four climatic regions, preferably with replicate sections, only two sections could be placed: one in Ohio (Wet-Freeze) and the other in Georgia (Wet-Nonfreeze). Since these are the two climatic regions where full-depth repairs are most frequently made, this is not a serious limitation; however, the lack of replicate sections does limit the statistical significance of the performance data.

The other compromises made in the scope of this experiment are not expected to seriously limit the usefulness of the results. For example, temperature at placement has a significant effect on the rate of strength gain. Although the ambient temperature is not directly considered in the experiment, it is not expected to adversely affect the results of the experiment because the strength at opening, which was closely monitored, was related to performance. The same repair techniques and repair design were used at each site, and the existing pavement at both test sites has an expected service life of 10 years or more.

Opening Times

The materials selected for the study are described in Appendix A. Table B.1 shows the nominal opening times selected for each of the materials. These times were achieved by placing the repairs at different times, and then opening the entire section to traffic when the last repair reached the minimum strength for opening. With each material, the times were selected to give opening times ranging from 1 to 4 hours less than the nominal opening time for the material to 1 to 4 hours more (depending on the material). This gives strengths at opening ranging from very low to very high, and should allow a comparison of the effects of the strength at opening on repair performance.

Table B.1. Nominal opening times of concrete repairs.

Mix	Number of Repairs for Each Opening Time (in hours)								
	1	3	5	6	8	10	18	26	Total
PC 1	3	4		3					10
RSC 1	3	4		3					10
VES		3	4		3				10
PC 2		3	4		3				10
RSC 2		3	4		3				10
Fast Track						3	4	3	10
HES						3	4	3	10
TOTAL	6	17	12	6	9	6	8	6	70

Analysis

The data collected from the experiment was analyzed to determine the amount of fatigue damage attributable to early opening, and the minimum strength required for opening concrete pavement repairs to traffic. The effect of concrete strength at opening on dowel bearing stress was also examined to study the effects of early opening on faulting of the repairs. A detailed discussion of how the analysis was performed is given in the Data Analysis section.

Georgia Site

Plan of Sections

The project site is located on eastbound I-20, near Augusta, between mile posts 189 and 192. At this site, the number of materials tested was limited to three, but twenty repairs were planned with each material rather than ten, as prescribed in the experimental design. The three materials tested were Georgia Department of Transportation's 4-hr mix (GADOT), Fast Track I (Fast Track), and Very Early Strength (VES). Each section contained ten repairs of one size and ten of another, except for the Fast Track section. The Fast Track section contained only 17 repairs, because of constraints at the test site. The repairs were 6 ft (1.8 m), 8 ft (2.4 m), 10 ft (3.0 m), 12 ft (3.7 m), and 15 ft (4.6 m) long. The location and size of the repairs are summarized in Table B.2. Table B.2 also summarizes the time of placement, estimated strength at opening time, estimated fatigue damage attributed to early opening, and any cracking observed during the follow-up survey.

At this site the repairs were placed and opened to traffic according to the state's standard practice. Georgia DOT has considerable experience in placing Fast Track concrete repairs, and the SHRP C-206 repairs were included as a part of an on-going repair project in which Fast Track, full-depth repairs were being placed. The early opening of concrete repairs to traffic in Georgia is based on curing time. Using the DOT's own early-opening mix, the minimum specified curing time on early opening projects is 4 hours. On this project, a typical day's work started at 6:30 p.m. with the closing of the work area to traffic. As soon as enough of the area was blocked off, concrete removal crews started their work. The concrete placement typically started at about 10:00 p.m. and continued until 2:00 a.m. The contractor was required to open the repairs to traffic at 6:00 a.m. and the last repair placed had to be given at least 4 hours of curing.

During the preliminary analysis, the minimum strength at which the repairs would be opened to traffic was determined for each material. The minimum strengths were established to give the repairs a minimum of 3 years of service life. The analysis showed that the minimum strengths for the opening of the test sections should be a modulus of rupture (third-point loading) of 150 psi to 200 psi (1.0 MPa to 1.4 MPa), depending on the material type, and a compressive strength of 1,500 psi (10.3 MPa). The minimum compressive strength was established based on the maximum expected dowel bearing stress determined using the modified Friberg analysis (Heinrichs 1989). The estimated dowel bearing stresses were between 1,500 psi and 2,000 psi (10.3 and 13.7 MPa), depending on the dowel diameter.

Table B.2. Summary of Georgia sections.

No.	MP	Distance to Next Repair, ft	Repair Length, ft	Time of Placement	Age at Opening	Temp oF	f'c	Modulus of Rupture, psi	Fatigue Damage Due to Early Opening	Cracking
Section 1, GA DOT				7/13/92						
1-1	189.1	110	8 ft	10:12 pm	7.8	78	2,250	365	0.001	none
1-2		162	15 ft	10:40 pm	7.3		2,100	355	0.036	none
1-3	189.2	288	8 ft	10:50 pm	7.2		2,050	350	0.001	none
1-4		570	8 ft	10:55 pm	7.1		2,050	350	0.001	none
1-5	189.3	30	8 ft	11:12 pm	6.8	77	1,950	340	0.001	none
1-6		270	8 ft	11:15 pm	6.8		1,950	340	0.001	none
1-7	189.4	90	15 ft	11:25 pm	6.6	77	1,900	330	0.038	none
1-8		90	15 ft	10:10 pm	7.8	81	2,250	365	0.036	none
1-9		90	8 ft	10:20 pm	7.7		2,250	360	0.001	none
1-10		120	8 ft	10:30 pm	7.5	79	2,150	360	0.001	none
1-11		30	15 ft	11:40 pm	6.3	76	1,800	320	0.040	none
1-12	189.5	30	8 ft	12:00 am	6.0		1,700	310	0.002	none
1-13		30	8 ft	12:05 am	5.9	75	1,650	305	0.003	none
1-14		30	15 ft	12:20 am	5.7		1,550	295	0.047	none
1-15		60	8 ft	12:20 am	5.7		1,550	295	0.003	none
1-16		60	16 ft	12:50 am	5.2		1,350	270	0.057	none
1-17		390	15 ft	1:10 am	4.8		1,200	245	0.059	none
1-18	189.6	450	15 ft	1:15 am	4.8	79	1,200	245	0.059	none
1-19	189.7	630	15 ft	1:45 am	4.3		1,000	200	0.067	none
1-20	189.8		15 ft	2:00 am	4.0		900	170	0.103	none
Sec Length: 3,545 ft			Opening:	7/14/92 6:00 am		4.0 Hr Opening				
Section 2, FT I				7/16/92						
2-3A	190.7	420	7 ft	10:10 pm	8.8	75	1,300	250	0.006	none
2-4	190.8	42	10 ft	10:30 pm	8.5		1,300	245	0.011	none
2-5	190.8	18	6 ft	10:40 pm	8.3		1,250	245	0.003	none
2-6	190.8	90	12 ft	10:45 pm	8.3		1,250	245	0.048	none
2-7	190.8	30	10 ft	10:45 pm	8.3		1,250	245	0.011	none
2-8	190.8	60	10 ft	10:55 pm	8.1		1,250	240	0.011	none
2-9	190.8	30	10 ft	11:10 pm	7.8	74	1,200	230	0.012	none
2-10	190.8	30	10 ft	11:15 pm	7.8		1,200	230	0.012	none
2-11	190.8	69	10 ft	11:25 pm	7.6		1,150	230	0.012	none
2-12	190.8	261	6 ft	11:30 pm	7.5		1,150	225	0.004	none
2-13	190.9	102	11 ft	11:30 pm	7.5		1,150	225	0.027	none
2-14	190.9	34	6 ft	11:45 pm	7.3		1,100	220	0.004	none
2-15	190.9	267	6 ft	11:50 pm	7.2		1,100	215	0.005	none
2-16	190.9	588	6 ft	11:55 pm	7.1		1,100	215	0.005	none
2-17	191.1	229	6 ft	12:10 am	6.8	73	1,000	205	0.006	none
2-18	191.1		7 ft	12:15 am	6.8		1,000	205	0.012	none
Sec Length: 2,277 ft			Opening:	7/17/92 7:00 am		6.8 Hr Opening				

Note: 1 ft = .305 m; oF = oC * 1.8 + 32; 1psi = 0.0069 MPa.

Table B.2. Summary of Georgia sections (continued).

No.	MP	Distance to Next Repair, ft	Repair Length, ft	Time of Placement	Age at Opening	Temp oF	f'c	Modulus of Rupture, psi	Fatigue Damage Due to Early Opening	Cracking
Section 3, VES				7/15/92						
3-1	191.1	210	12 ft	10:20 pm	6.7	77	2,700	380	0.003	none
3-2	191.2	60	12 ft	10:40 pm	6.3	74	2,600	365	0.003	none
3-3	191.2	90	12 ft	11:00 pm	6.0		2,500	360	0.003	none
3-4	191.2	30	12 ft	11:15 pm	5.8		2,450	350	0.003	none
3-5	191.2	180	12 ft	11:25 pm	5.6		2,350	335	0.003	none
3-6	191.2	30	8 ft	11:30 pm	5.5	74	2,350	335	0.001	none
3-7	191.2	30	12 ft	11:50 pm	5.2		2,200	320	0.003	none
3-8	191.2	90	8 ft	12:05 am	4.9		2,100	305	0.001	none
3-9	191.3	60	8 ft	12:10 am	4.8		2,050	300	0.001	none
3-10	191.3	45	12 ft	12:20 am	4.7		2,000	295	0.004	none
3-11	191.3	54	12 ft	12:25 am	4.6	73	1,950	290	0.004	none
3-12	191.3	21	8 ft	12:40 am	4.3		1,800	270	0.002	none
3-13	191.3	240	8 ft	12:40 am	4.3		1,800	270	0.002	none
3-14	191.3	120	12 ft	12:50 am	4.2		1,700	260	0.014	none
3-15	191.4	270	9 ft	12:55 am	4.1		1,650	255	0.005	none
3-16	191.4	150	12 ft	1:10 am	3.8	72	1,550	230	0.015	none
3-17	191.4	540	8 ft	1:20 am	3.7		1,500	225	0.008	none
3-18	191.5	600	8 ft	1:20 am	3.7		1,500	225	0.008	none
3-19	191.6	780	8 ft	2:15 am	2.8		1,050	165	0.073	none
3-20	191.8		8 ft	2:20 am	2.7	72	1,000	155	0.122	none
Sec Length: 3,600 ft			Opening:	7/16/92 6:00 am		2.7 Hr Opening				

Note: 1 ft = .305 m; oF = oC * 1.8 + 32; 1 psi = 0.0069 MPa.

It was also determined that if the state's normal practice gives strengths less than those established in the preliminary analysis, the lower strengths from the state's normal practice would be used as the criteria for opening. Hence, the GADOT mix was placed first, following the state's normal placement schedule. This mix achieved a compressive strength of 1,000 psi (6.9 MPa) in 4 hours. The modulus of rupture was about 180 psi (1.2 MPa). Since the compressive strength of the GADOT material at normal opening time is much less than the minimum compressive strength established in the preliminary analysis, and Georgia has had much success with their standard practice, the compressive strength of 1,000 psi (6.9 MPa) was adopted as the criteria for the opening of all of the sections.

The compressive strength of 1,000 psi (6.9 MPa) is only half of the calculated dowel bearing stress for the repair design used in Georgia. The consequence of subjecting concrete repairs to a very high bearing stresses at early ages is not well known. In reality, the high dowel bearing stresses are not likely to develop. The concrete is more likely to yield and cause socketing of the dowels. Whether or not the high dowel bearing stress develops, it appears that inadequate compressive strength in the repairs at opening is likely to result in increased faulting of the repairs.

One of the factors that may be working in favor of the dowel bearing capacity of concrete slabs at early ages is that the compressive strength of the concrete around the dowels may be higher than what the cylinder tests would indicate because of the higher temperatures at the middle of the slab. Higher temperatures mean a greater maturity, indicating higher strength.

Traffic Data Collection

The traffic data needed for fatigue analysis include the traffic count, traffic distribution, truck weights, and lateral traffic wander. These data were collected during the installation of the test sections. Using the AASHTO procedure (AASHTO 1986) to convert the traffic passes to the number of 18-kip (80 kN) equivalent single axle loads (ESAL) applications, the first three data items together give the number of ESAL loads placed on the pavement. The lateral traffic wander gives information regarding the number of ESAL loads placed at the critical location for fatigue.

Traffic Count and Distribution

The daily traffic count and traffic distribution data were provided by the Planning Data Services Division of GADOT. The data were collected from the first day of repair placement through 2 days past the last day (from July 13 through July 18, 1992). An automated traffic counter/classifier was run continuously during this time period to collect the data.

Figure B.1 shows the traffic pattern observed during this time period. The data shown represent two-way average daily traffic (ADT) of 17,140 vehicles, of which 17.4% were heavy trucks (class 5 and greater). The annual average at this location is 25,750 ADT and 19.5% trucks. Figure B.1 shows that while there is a daily cycle of heavy and low traffic hours, the truck traffic remains fairly constant throughout the day at this location; however, the figure shows a definite weekly cycle in truck traffic.

Truck Weights

The truck weight data were collected manually at a nearby weigh station. The weigh station is located only about 2 mi (3.2 km) west of the test site, and there is only one intersection to a small county road between the station and the test site. The station is open 24 hours a day, 7 days a week, and equipped with weigh-in-motion (WIM) located at the approach ramp as well as static scales. If the WIM shows that the approaching truck is overweight, it is pulled over for static weighing; otherwise, it is waved through. The WIM data feed directly into a computer, and the computer automatically directs the traffic.

While the computer system at the weigh station provided all of the information needed for the study, no means of storing this information was available because of equipment problems at the time. Hence, the truck weight data were recorded manually off of the computer monitor. Because of the requirement for manual data collection, the amount of data that could be collected was somewhat limited. Efforts were made to collect the truck weight data for 4 hours each morning, starting at 6:00 a.m. when the repairs were opened to traffic. Additionally, the data were collected for one 24-hr period.

Using the AASHTO load equivalency factors and the truck weight data, the truck factor (the average number of ESALs/truck) was determined on an hourly basis. Figure B.2 shows the results. The figure shows that there is a daily cycle in the truck factor. The hourly truck factors were applied to the daily truck traffic to obtain the hourly ESAL passes at the test site during installation. This is shown in Figure B.3. It was assumed that the truck factors shown in Figure B.2 are representative of the truck factors during the installation period. The overall average truck factor was 1.37.

In the fatigue analysis, the fatigue damage was determined on a hourly basis for the first 3 days and then daily for the next 11 days to determine the fatigue damage attributable to early opening. The daily totals from Figure B.3 were used to estimate the daily traffic for the days following the last day the actual traffic data were collected. The assumed traffic for the entire analysis period (7/13/92 through 7/31/92) is shown in Figure B.4.

Lateral Traffic Wander

The lateral traffic wander data were collected by painting distance markers on the pavement and visually observing the location of the truck wheels as they passed over the markers from a bridge overpass. The distance marking consisted of five 3 × 4 in. (76 × 102 mm) blocks with 4 in. (102 mm) spacing placed at the pavement edge to cover the outer 40 in. (1.0 m) of the driving lane. They were spray-painted on the pavement using a cardboard mask. With these markings, it was possible to determine the wheel location with an accuracy of ±1 in. (25 mm).

The average location of the truck wheels at this site was 22.8 in. (580 mm) from the outer edge. This value is about 4 in. (102 mm) greater than the typical average of 18 in. (460 mm). One possible explanation for this difference is that on this particular stretch of EB I-20, approximately the outer 4 in. (102 mm) of the driving lane is covered with asphalt/tar, giving the impression that the outer 4 in. (100 mm) of the driving lane is part of the asphalt shoulder. This could have caused the movement of the average truck wheel path toward the center. The average wheel location was determined based on 184 observations, so more data are needed to confirm this finding.

Follow-up Survey

The test sections were evaluated 2 months after construction to obtain the in-place, material properties and to document any failures. The testing consisted of coring, falling weight deflectometer (FWD) testing, and a visual distress survey. The cores were tested to determine the in-place compressive strength and elastic modulus. The FWD test results were used to determine the modulus of subgrade reaction, dynamic modulus of concrete, and load-transfer efficiency at the transverse joints.

The results of the core testing are given in Table B.3 and the FWD testing in Table B.4. The average deflection load-transfer at the transverse joints ranged from 62% to 84%. The repair leave-joints generally showed somewhat higher load-transfer efficiency than the approach-joints. However, FWD testing did not show any apparent relationship between the

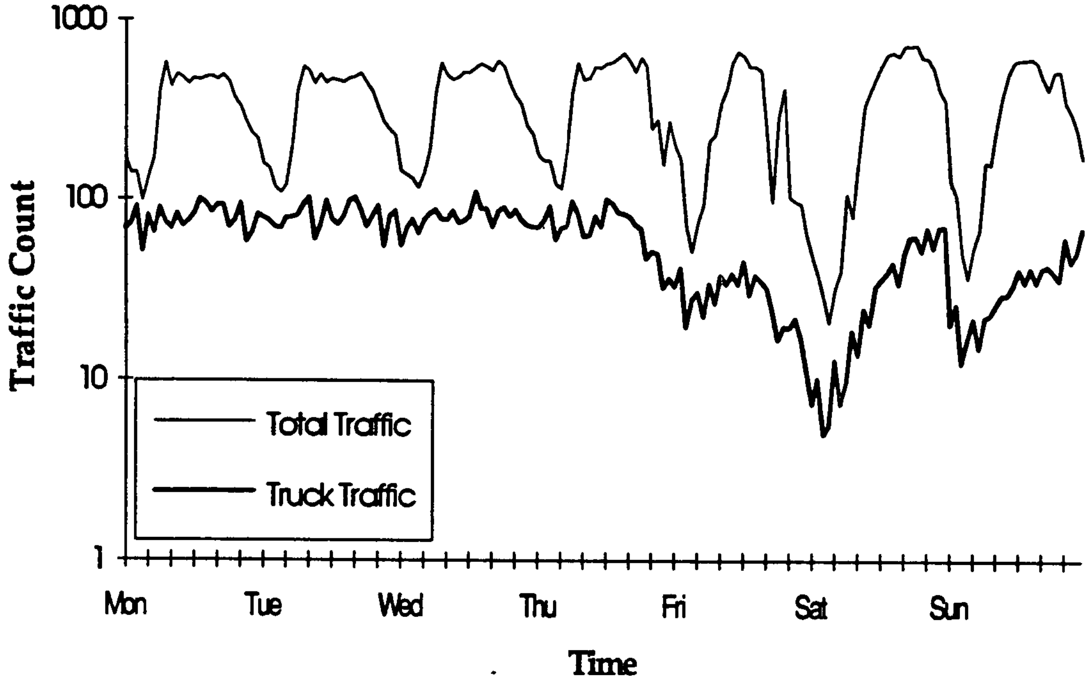

Figure B.1 Traffic on EB I-20, Augusta, GA, during the week of 7/13/92.

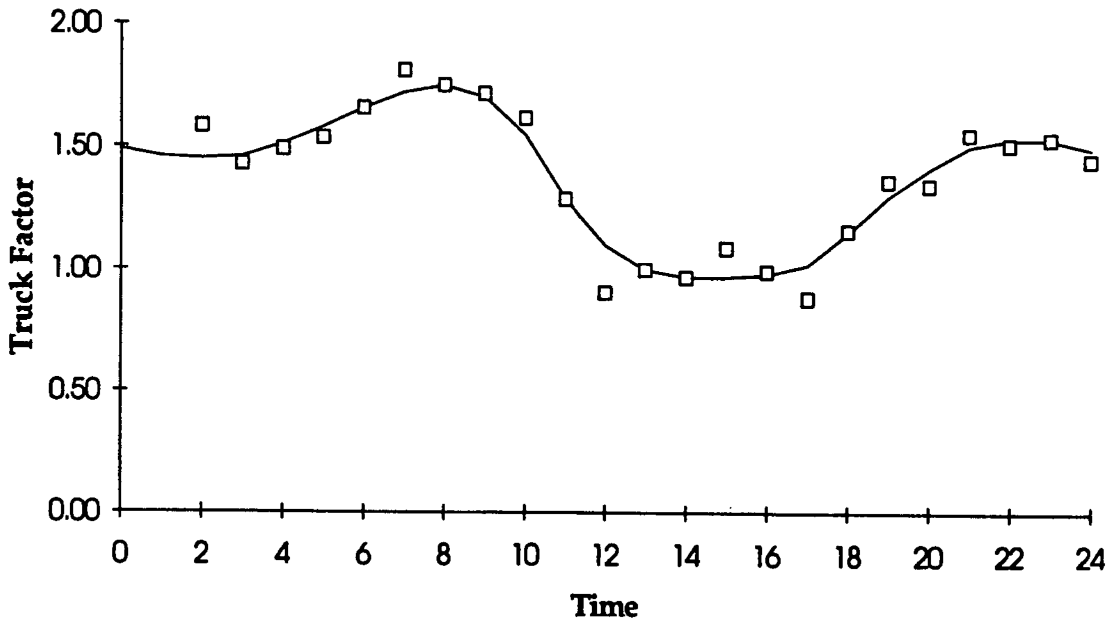

Figure B.2 Hourly Truck Factor Variation, I-20, Augusta, GA.

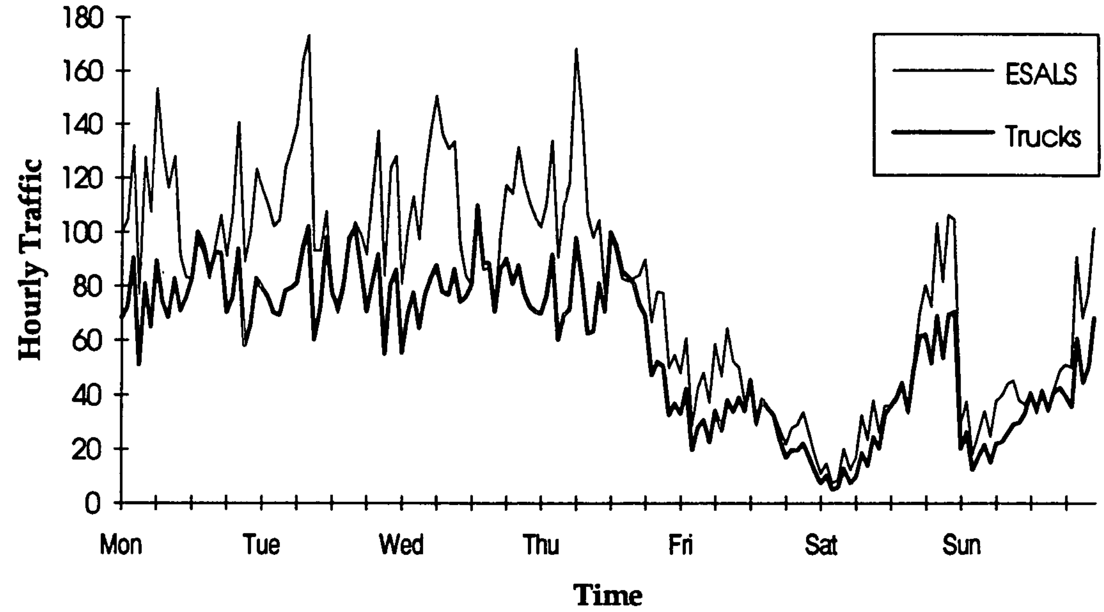

Figure B.3 Truck traffic on EB I-20, Augusta, GA, during the week of 7/13/92.

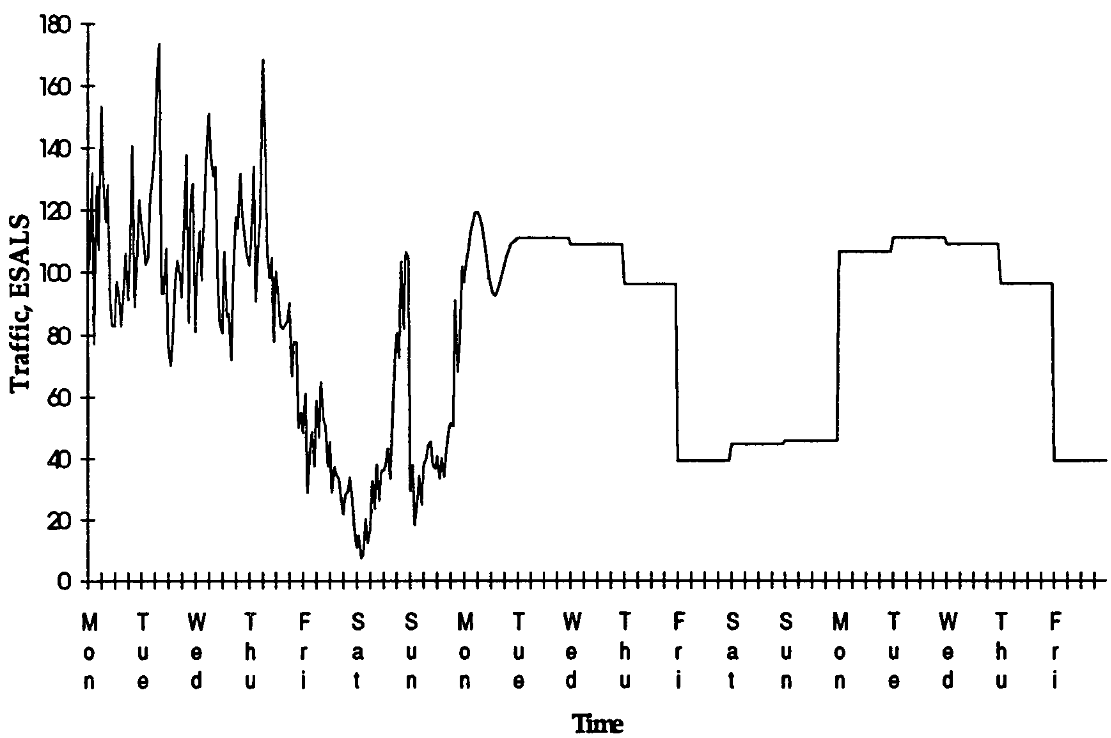

Figure B.4 Assumed traffic on EB I-20, Augusta, GA (7/13/92 - 7/31/92).

compressive strength at opening time and joint load-transfer efficiency (Table B.2 shows the strength of each repair at opening time). The average modulus of subgrade reaction (k) was 240 pci (65.2 kPa/mm). The visual distress survey showed no cracking or other distresses in the Georgia sections after 2 months of service.

Table B.3. Core testing results, Georgia sections.

Mix	Compressive Strength, psi	Split Tensile Strength, psi	Modulus of Rupture, psi	E_c, Mpsi
GADOT	4,790	580	570	3.75
Fast Track	4,540	505	575	3.55
VES	5,790	590	535	3.85

Note: 1 psi = 0.0069 MPa

Table B.4. Falling weight deflectometer testing results from Georgia sections.

Material Type			Temperature, oC		Load Transfer Efficiency				
	Repair No.	Station	Air	Slab	Approach	Leave	Average	K, pci	Ec, psi
GA DOT	1-3	0+00	15	13	75.9%	86.0%	81.0%	170	4.33E+06
	1-4	2+88	15	13	69.7%	84.4%	77.1%	189	5.62E+06
	1-7	11+59	16	14	66.8%	83.5%	75.1%	154	5.94E+06
	1-8	12+49	16	14	50.9%	84.9%	67.9%	168	6.66E+06
	1-9A	13+69	16	14	54.4%	84.2%	69.3%	192	7.31E+06
	1-18	21+54	17	14	63.5%	78.8%	71.2%	132	7.32E+06
	1-19	26+03	17	14	51.1%	82.9%	67.0%	337	5.21E+06
	1-20	32+34	17	15	66.6%	81.3%	73.9%	250	6.55E+06
	Average				*62.4%*	*83.3%*	*72.8%*	*199*	*6.12E+06*
FT I	2-4	0+46	22	19	73.0%	83.3%	78.2%	213	6.90E+06
	2-5	0+83	22	18	70.6%	80.1%	75.3%		
	2-6	1+05	22	19	71.7%	83.8%	77.7%	256	5.61E+06
	2-10	3+14	22	18	75.2%	81.0%	78.1%	173	6.84E+06
	2-11	3+46	23	20	75.0%	84.4%	79.7%	197	5.57E+06
	2-12	4+16	25	19	62.2%	72.0%	67.1%		
	2-17	16+63	22	18	84.1%	77.8%	81.0%		
	2-18	18+92	23	19	68.1%	84.0%	76.0%	249	3.28E+06
	Average				*72.5%*	*80.8%*	*76.6%*	*218*	*5.64E+06*
VES	3-2	21+67	22	20	65.1%	82.8%	73.9%	169	7.99E+06
	3-3	22+25	23	19	59.2%	83.6%	71.4%	182	6.53E+06
	3-8	25+85	22	18	85.0%	86.8%	85.9%	253	5.44E+06
	3-9	26+77	23	19	68.8%	83.7%	76.3%	247	5.51E+06
	3-10	27+38	23	19	74.6%	81.8%	78.2%	230	6.38E+06
	3-11	27+83	22	19	71.0%	84.7%	77.9%	267	6.15E+06
	3-17	36+38	23	20	72.0%	83.2%	77.6%	419	4.71E+06
	3-18	41+90	23	19	54.8%	82.7%	68.8%	505	3.61E+06
	3-20	55+68	24	21	83.7%	84.0%	83.8%	289	3.78E+06
	Average				*71.2%*	*83.8%*	*77.5%*	*299*	*5.27E+06*

Note: °C = (°F - 32) / 1.8; 1 pci = 0.2715 kPa/mm; 1 psi = 0.0069 MPa

Ohio Site

Plan of Section

The project site is located in both the eastbound and westbound lanes of S.R. 2, near Vermilion, between the Baumhart Road and Vermilion Road exits. A total of eight different materials were tested at this site: all of the materials listed in Table B.1 plus the early opening mix from Ohio Department of Transportation (ODOT). Ten 6-ft (1.8 m) repairs were made with each of the eight materials tested. As in Georgia, one material was placed per day, and each test section consisted of repairs made with one material. The eight test sections were evenly divided between the eastbound and westbound directions. The repair locations are summarized in Tables B.5 and B.6, along with the time of placement, strength at the opening time, and estimated fatigue damage attributed to early opening.

Because of short repair lengths (6 ft [1.8 m]), fatigue was not a critical factor controlling the opening of the repair to traffic at this site. The compressive strength was much more critical than the modulus of rupture. Hence, the compressive strength of 1,000 psi (6.9 MPa) was again used as the criteria for opening the repairs to traffic.

Traffic Data Collection

As in Georgia, the traffic data collected during the installation of the Ohio sections included traffic count, traffic distribution, truck weights, and lateral traffic wander. At the Ohio site, all of the traffic data, except for lateral traffic wander, were provided by the ODOT Bureau of Technical Services.

Traffic Count and Distribution

The traffic count and distribution data were collected continuously from September 1 through September 20, 1992, and then again from September 26 through October 4, 1992 on both the eastbound and westbound lanes. Figure B.5 shows the traffic pattern observed during the period when the test sections were installed. This represents a two-way ADT of 18,300, of which 13% were trucks. The annual average taken in 1990 was 14,400 ADT with 22% trucks. The traffic was evenly distributed between the two directions, and about 90% of trucks were using the driving lane.

Truck Weights

The truck weight data were collected using a WIM device. The WIM data were collected from September 22 through September 25, 1992 in the eastbound direction, and from October 6 through October 8, 1992 in the westbound direction. The hourly truck factors were determined based on the WIM data using the AASHTO procedure and are shown in Figure B.6. The data from midnight through 4:00 a.m. are missing, but the truck count during this period is very low. Hence, the use of the average value of the truck factor is not likely to adversely affect the analysis results. The average overall truck factor was 1.94.

Table B.5. Summary of Ohio sections—eastbound sections.

No.	MP	Station	Repair Length, ft	Time of Placement	Age at Opening	Temp oF	f'c, psi	Modulus of Rupture, psi	Fatigue Damage Due to Early Opening	Cracking (LT)
HES - 10 to 26 hr Opening				9/1/92						
A-1	2.8	147+97	6 ft	2:50 pm	14.8	74	4,450	505	0.000	n/a
A-2		148+17	6 ft	12:50 pm	16.8	73	4,650	520	0.000	L
A-3		148+57	6 ft	1:55 pm	15.8	77	4,600	520	0.000	
A-4		148+82	6 ft	3:00 pm	14.7	74	4,450	510	0.000	L
A-5		149+00	6 ft	3:35 pm	14.1		4,350	500	0.000	
A-6		149+21	6 ft	7:20 pm	10.3	67	3,800	455	0.000	
A-7		149+40	6 ft	7:30 pm	10.2		3,750	455	0.000	
A-8		149+53	6 ft	7:35 pm	10.1	63	3,750	455	0.000	
A-9		149+65	6 ft	8:05 pm	9.6	58	3,700	450	0.000	L
A-10		150+05	6 ft	8:10 pm	9.5	53	3,700	450	0.000	
Sec Length:		208 ft	Opening:	9/2/92 5:40 am		9.5	Hr Opening			
FT I - 10 to 26 hr Opening				9/2/92						
B-1	2.1	113+12	6 ft	11:15 am	18.3	68	4,300	580	0.000	n/a
B-2	2.2	113+57	6 ft	10:35 am	19.0	66	4,350	585	0.000	
B-3		113+92	6 ft	11:25 am	18.2		4,300	580	0.000	
B-4		114+56	6 ft	11:35 am	18.0		4,300	575	0.000	
B-5		114+96	6 ft	2:10 pm	15.4	73	4,100	560	0.000	
B-6		115+11	6 ft	2:20 pm	15.3		4,100	560	0.000	
B-7		115+37	6 ft	2:25 pm	15.2		4,100	555	0.000	
B-8		115+52	6 ft	5:10 pm	12.4	68	3,900	530	0.000	L
B-9		115+77	6 ft	5:15 pm	12.3		3,850	530	0.000	L
B-10		115+94	6 ft	5:20 pm	12.3		3,850	530	0.000	
Sec Length:		282 ft	Opening:	9/3/92 5:35 am		12.3	Hr Opening			
PC 1 - 1 to 6 hr Opening				9/3/92						
C-1	1.1	58+26	6 ft	10:50 am	6.6	77	2,150	350	0.000	n/a
C-2		58+56	6 ft	11:35 am	5.8		1,900	325	0.000	L
C-3		58+96	6 ft	12:15 pm	5.2		1,850	320	0.000	
C-4		59+12	6 ft	12:40 pm	4.8		1,800	315	0.000	
C-5		59+36	6 ft	1:15 pm	4.2	84	1,750	305	0.001	
C-6		59+54	6 ft	1:30 pm	3.9		1,650	300	0.001	
C-7	1.2	61+88	6 ft	2:05 pm	3.3		1,500	280	0.002	
C-8		62+18	6 ft	2:25 pm	3.0		1,500	275	0.002	
C-9		62+29	6 ft	2:55 pm	2.5		1,300	255	0.005	
C-10		63+00	6 ft	3:25 pm	2.0		1,150	230	0.010	
Sec Length:		474 ft	Opening:	5:25 pm	2.0	Hr Opening				
RSC 1 - 1 to 6 hr Opening				9/4/92						
D-1		1828+26	6 ft	11:05 am	4.8		4,050	475	0.000	n/a
D-2		1828+72	6 ft	10:20 am	5.5	69	4,100	480	0.000	L
D-3		1829+11	6 ft	12:10 pm	3.7	72	3,950	460	0.000	L
D-4		1829+83	6 ft	12:15 pm	3.6		3,900	455	0.000	L
D-5		1830+25	6 ft	12:45 pm	3.1	80	3,750	440	0.000	
D-6		1830+38	6 ft	12:50 pm	3.0	76	3,700	435	0.000	L
D-7		1831+84	6 ft	1:30 pm	2.3	71	3,200	380	0.001	L
D-8		1832+20	6 ft	1:35 pm	2.3		3,200	380	0.001	L
D-9		1832+34	6 ft	1:55 pm	1.9		2,150	245	0.004	L
D-10		1832+61	6 ft	2:00 pm	1.8	70	2,150	245	0.004	L
Sec Length:		435 ft	Opening:	3:50 pm	1.8	Hr Opening				

Note: 1 ft = .305 m; 1 psi = 0.0069 MPa; HL = Hairline crack; L = Low severity crack

Table B.6. Summary of Ohio sections—westbound sections

No.	MP	Station	Repair Length, ft	Time of Placement	Age at Opening	Temp oF	f'c, psi	Modulus of Rupture, psi	Fatigue Damage Due to Early Opening	Cracking (LT)
VES - 3 to 8 hr Opening				9/8/92						
E-1	2.0	107+08	6 ft	12:25 pm	5.4		2,750	355	0.000	n/a
E-2		106+58	6 ft	11:20 am	6.5	75	3,100	375	0.000	L
E-3		106+28	6 ft	11:50 am	6.0		3,050	370	0.000	HL
E-4		105+82	6 ft	12:50 pm	5.0		2,550	340	0.000	
E-5		105+44	6 ft	1:35 pm	4.3	74	2,050	310	0.001	HL
E-6		105+04	6 ft	1:35 pm	4.3		2,050	310	0.001	L
E-7		104+65	6 ft	2:00 pm	3.8		1,300	260	0.002	
E-8		104+20	6 ft	2:05 pm	3.8		1,300	260	0.002	L
E-9		103+87	6 ft	2:35 pm	3.3	75	1,000	240	0.004	
E-10		103+30	6 ft	2:35 pm	3.3		1,000	240	0.004	
Sec Length:		378 ft	Opening:	5:50 pm	3.3	Hr Opening				
PC 2 - 3 to 8 hr Opening				9/9/92						
F-1	2.1	112+44	6 ft	10:35 am	7.3	68	1,600	305	0.001	n/a
F-2		112+02	6 ft	11:15 am	6.6	74	1,500	290	0.002	
F-3		111+77	6 ft	11:55 am	5.9	79	1,400	275	0.003	L
F-4		111+65	6 ft	12:10 pm	5.7	81	1,400	270	0.004	
F-5		111+45	6 ft	12:55 pm	4.9	82	1,350	260	0.005	
F-6		111+24	6 ft	1:05 pm	4.8	83	1,350	255	0.010	
F-7		111+07	6 ft	1:35 pm	4.3	81	1,300	250	0.011	
F-8		110+29	6 ft	1:45 pm	4.1	82	1,250	250	0.011	
F-9		109+91	6 ft	2:10 pm	3.7	84	1,200	235	0.011	
F-10		109+40	6 ft	2:20 pm	3.5	84	1,150	220	0.011	
Sec Length:		304 ft	Opening:	5:50 pm	3.5	Hr Opening				
RSC 2 - 3 to 8 hr Opening				9/10/92						
G-1	2.6	136+63	6 ft	1:00 pm	4.7		3,800	520	0.000	n/a
G-2		135+70	6 ft	12:10 pm	5.5	77	3,950	535	0.000	
G-3		134+73	6 ft	1:20 pm	4.3		3,350	500	0.000	
G-4	2.5	133+69	6 ft	1:45 pm	3.9	77	3,150	465	0.000	
G-5		133+42	6 ft	1:50 pm	3.8		3,150	465	0.000	
G-6		132+67	6 ft	2:20 pm	3.3		2,600	420	0.000	
G-7		131+44	6 ft	2:30 pm	3.2		2,600	420	0.000	
G-8		131+11	6 ft	2:55 pm	2.8		2,150	375	0.000	
G-9		129+94	6 ft	3:05 pm	2.6	78	1,950	350	0.000	
G-10		129+43	6 ft	3:15 pm	2.4		1,850	310	0.001	
Sec Length:		720 ft	Opening:	5:40 pm	2.4	Hr Opening				
FS - 3 to 8 hr Opening				9/11/92						
H-1	2.9	152+94	6 ft	12:40 pm	7.1	68	5,300	550	0.000	n/a
H-2		152+53	6 ft	1:00 pm	6.8		5,250	540	0.000	L
H-3		151+11	6 ft	12:00 pm	7.8	71	5,500	565	0.000	L
H-4	2.8	149+48	6 ft	1:15 pm	6.5		5,150	535	0.000	L
H-5		148+66	6 ft	1:30 pm	6.3	66	5,100	530	0.000	L
H-6		147+57	6 ft	3:40 pm	4.1		4,550	470	0.000	L
H-7		147+03	6 ft	3:50 pm	3.9		4,450	465	0.000	L
H-8		146+68	6 ft	5:25 pm	2.3		1,100	135	0.150	L
H-9	2.7	145+04	6 ft	5:35 pm	2.2		1,100	135	0.150	L
H-10		144+25	6 ft	5:45 pm	2.0		1,100	135	0.150	L
Sec Length:		869 ft	Opening:	7:45 pm	2.0	Hr Opening				

Note: 1 ft = .305 m; 1 psi = 0.0069 MPa; HL = Hairline crack; L = Low severity crack

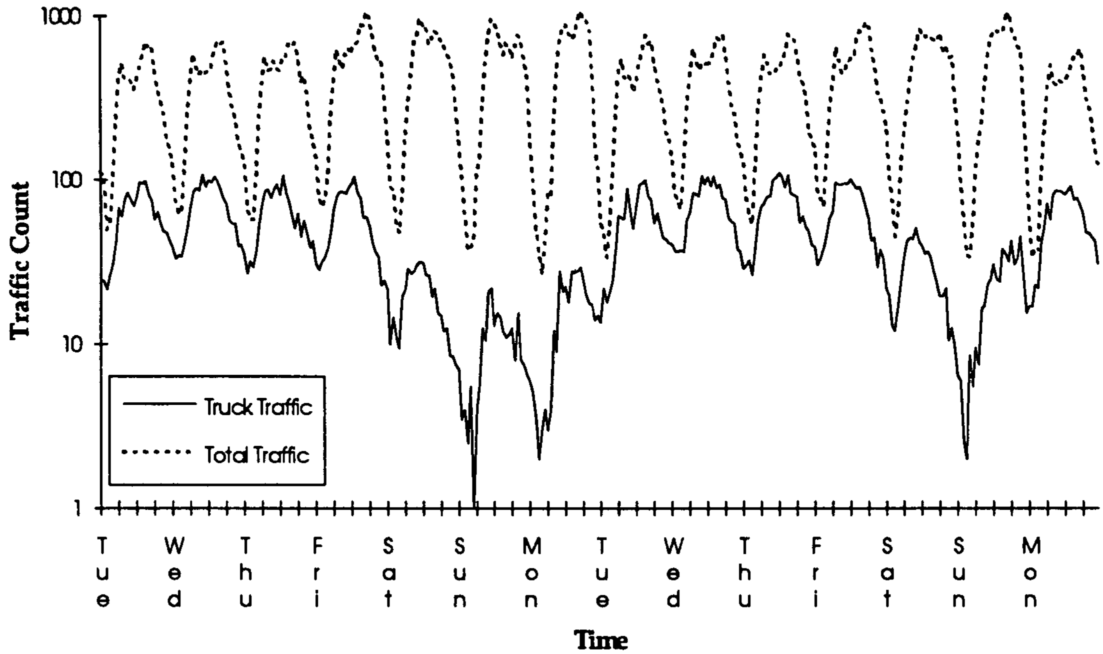

Figure B.5 One way traffic on SR2, Vermillion Ohio (9/1/92 - 9/14/92).

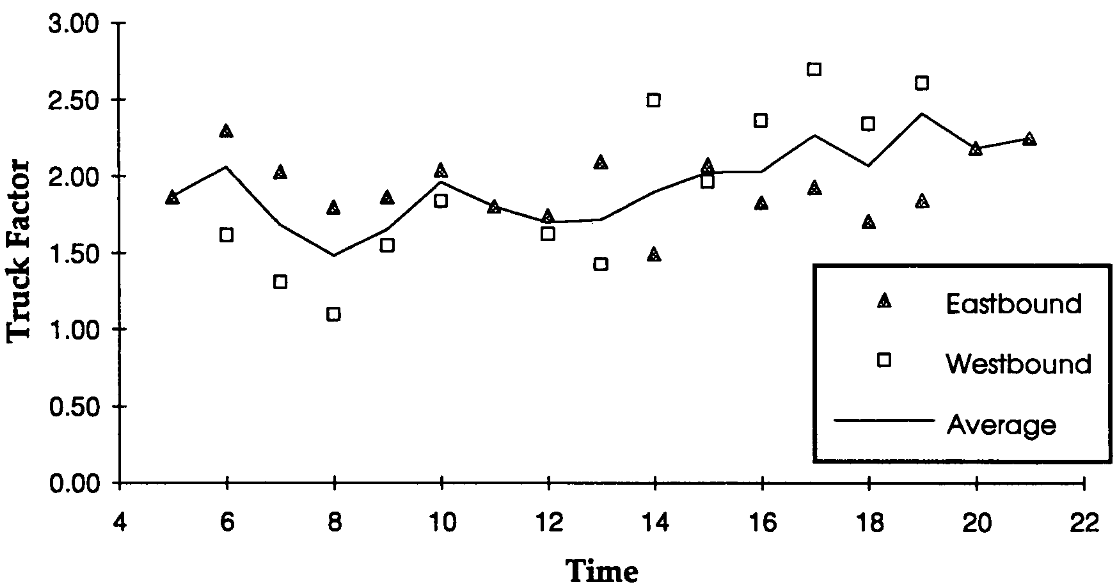

Figure B.6 Hourly truck factor variation, SR2, Vermillion, OH.

Figure B.7 shows the truck traffic during the installation in terms of both the truck count and ESALs.

As in Georgia, the truck traffic on S.R. 2 also showed a weekly cycle. Although considerably more traffic data were collected from this site, there were some gaps in the data. Therefore, the average of all of the available data were used to estimate the average daily truck traffic for each day of the week and this average traffic was used to represent the traffic during the periods when the actual data were not available. The assumed traffic for the Ohio sections is shown in Figure B.8.

Lateral Traffic Wander

The same method used in Georgia was used to obtain the lateral traffic wander data. Since the duration of construction was much longer in Ohio, it was possible to obtain more data. The average truck wheel location based on 630 observations was 18.9 in. (480 mm), measured from the outer slab edge.

Follow-up Survey

The Ohio sections were also evaluated 2 months after construction to obtain the in-place, material properties and document any failures. The testing consisted of coring, FWD testing, and a visual distress survey. As with the Georgia sections, the cores were tested to determine the in-place compressive strength and elastic modulus. The core testing results are summarized in Table B.7.

The FWD testing at the Ohio site was conducted primarily to determine and document load-transfer efficiency at the transverse joints. Because of the short repair length (6 ft [1.8 m]), reliable values of subgrade modulus of reaction (k) could not be established by testing the repairs. Table B.8 shows a considerable variation in the back-calculated values of k and elastic modulus of concrete. The subgrade k for the Ohio sections was assumed to be 200 pci

Table B.7. Core testing results, Ohio sections.

Mix	f'_c (psi)	E_c (Mpsi)
HES	6,090	2.80
Fast Track	5,720	2.90
PC 1	6,360	3.85
RSC 1	6,010	3.05
VES	5,600	3.85
PC 2	8,040	3.60
RSC 2	5,620	3.00
ODOT	7,520	3.15

Note: 1 psi = 0.0069 MPa

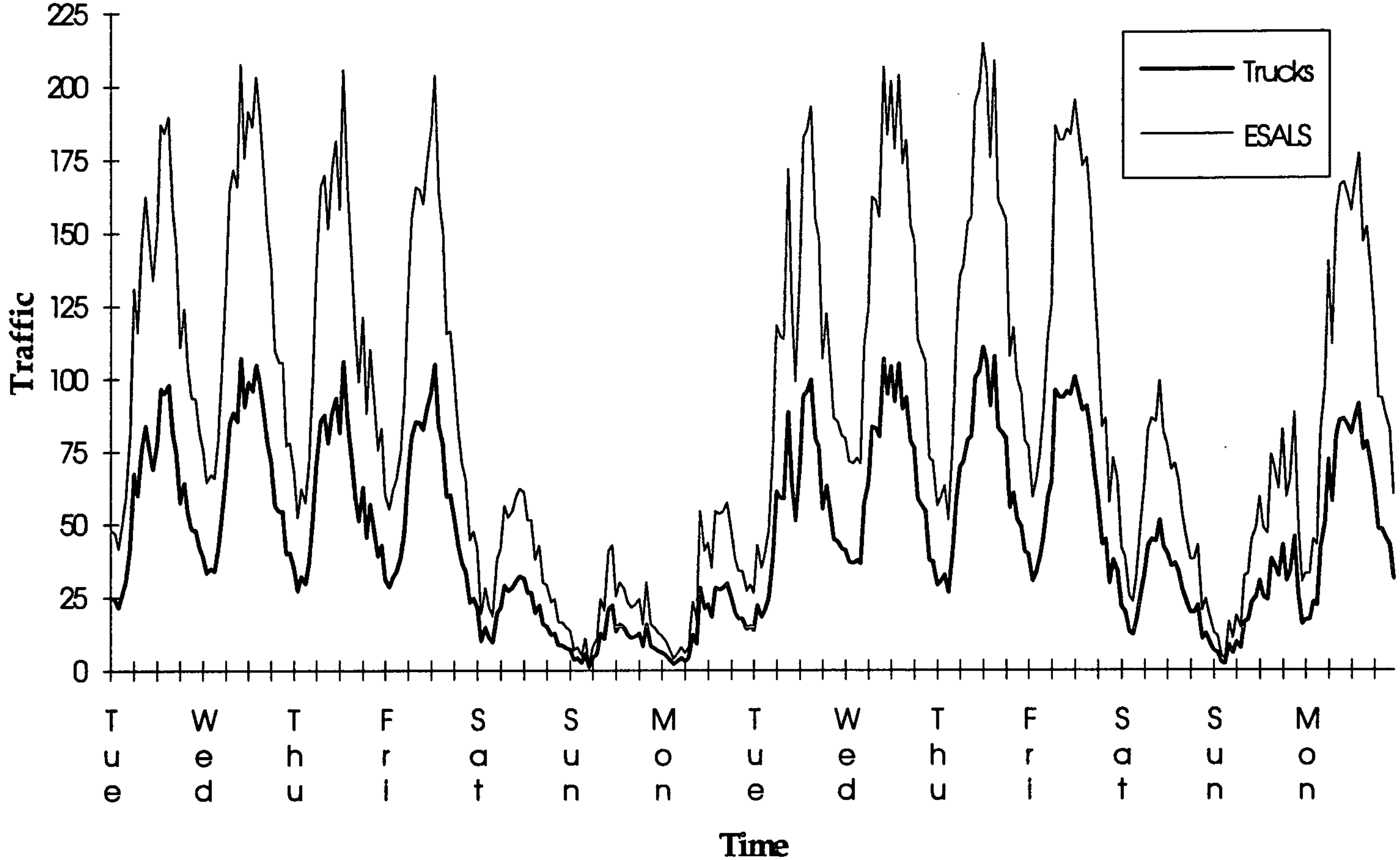

Figure B.7 One way truck traffic on SR2, Vermillion Ohio (9/1/92 - 9/14/92).

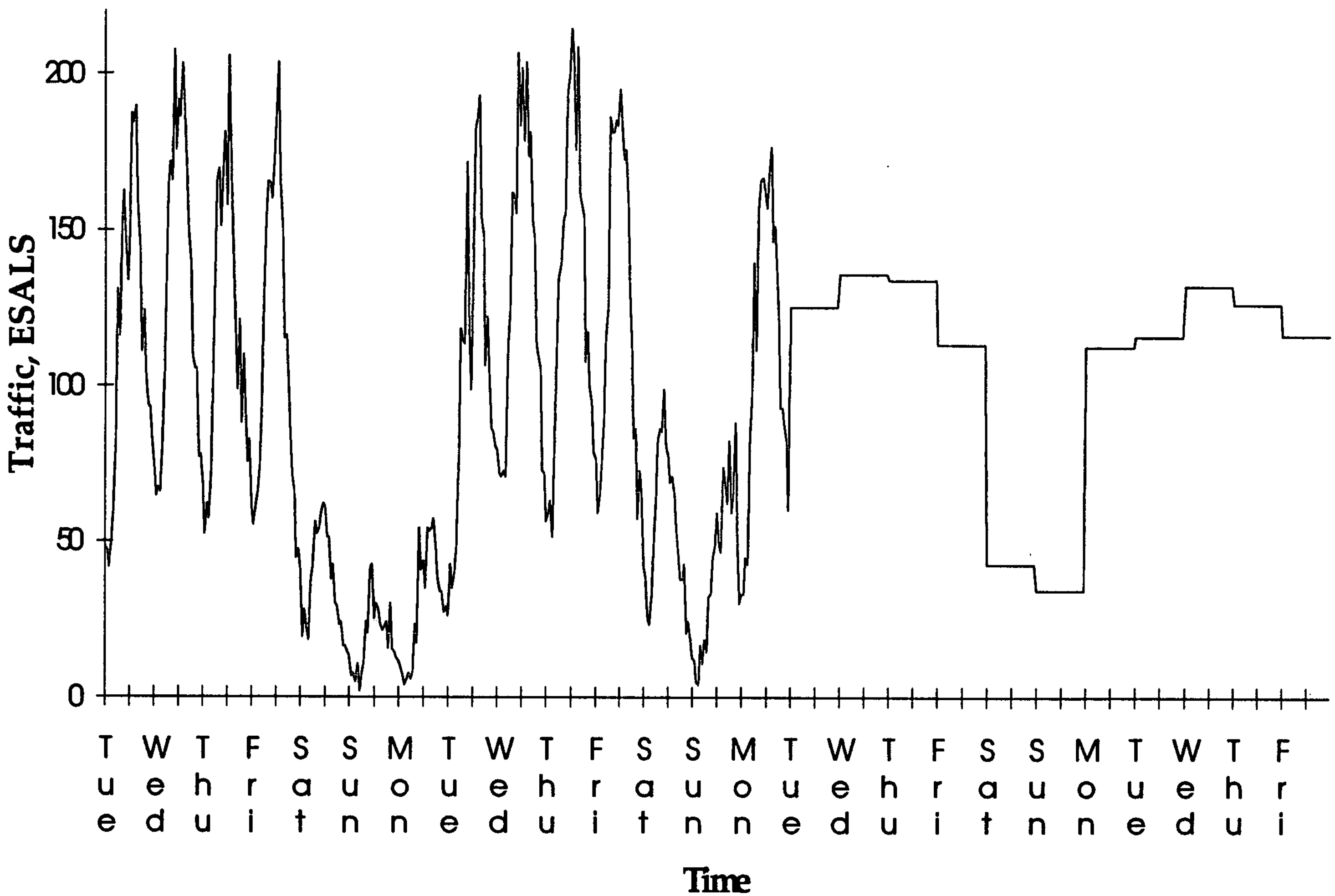

Figure B.8 Assumed one way traffic on SR2, Vermillion, OH (9/1/92 - 9/25/92).

Table B.8. Falling weight deflectometer testing results of Ohio sections.

Material	Repair No.	Air Temp oC	Load Transfer Efficiency Approach	Leave	Average	k, pci	Ec, psi
HES	A-6	9	74%	73%	74%	664	2.64E+06
	A-7	10	73%	77%	75%		
	A-8	10	74%	75%	75%		
	A-10	10	72%	75%	73%	177	1.76E+07
	Average		*73%*	*75%*	*74%*	*420*	*1.01E+07*
FT1	B-6	9	70%	63%	66%	177	1.76E+07
	B-7	8	71%	52%	62%		
	B-9	10	73%	63%	68%		
	B-10	8	68%	56%	62%	232	6.06E+06
	Average		*70%*	*59%*	*64%*	*204*	*1.18E+07*
PC 1	C-7	9	70%	75%	73%	359	2.56E+07
	C-8	10	76%	76%	76%		
	C-10	10	74%	76%	75%	561	8.33E+06
	Average		*73%*	*76%*	*74%*	*460*	*1.69E+07*
RSC 1	D-7	8	80%	79%	80%	313	4.02E+06
	D-8	11	81%	77%	79%		
	D-9	9	81%	79%	80%		
	D-10	9	65%	76%	70%	138	7.72E+06
	Average		*77%*	*78%*	*77%*	*226*	*5.87E+06*
VES	E-6	8	73%	73%	73%	260	2.93E+06
	E-7	7	82%	82%	82%		
	E-9	8	77%	80%	78%		
	E-10	7	83%	80%	81%	241	3.25E+06
	Average		*79%*	*79%*	*79%*	*251*	*3.09E+06*
PC 2	F-6	8	81%	82%	82%	153	1.09E+07
	F-7	8	82%	82%	82%		
	F-9	8	84%	82%	83%		
	F-10	8	81%	81%	81%	119	1.54E+07
	Average		*82%*	*82%*	*82%*	*136*	*1.31E+07*
RSC 2	G-6	9	82%	79%	81%	360	3.20E+06
	G-7	9	81%	79%	80%		
	G-9	9	84%	73%	79%		
	G-10	9	81%	80%	80%	222	3.11E+06
	Average		*82%*	*78%*	*80%*	*291*	*3.15E+06*
FS	H-6	10	72%	71%	72%		
	H-7	10	80%	75%	78%	263	4.63E+06
	H-9	9	75%	76%	75%	265	8.58E+06
	H-10	9	82%	71%	76%		
	Average		*77%*	*73%*	*75%*	*264*	*6.60E+06*

Note: oC = (oF - 32) / 1.8; 1pci = .2715 kPa/mm; 1 psi = 0.0069 MPa.

(54 kPa/mm) based on the results of FWD testing on larger slabs at this site. The load-transfer efficiency at the transverse joints ranged from 59% to 82%.

The visual distress survey showed that a significant number of the repairs were cracked longitudinally. All of the cracks occurred at the midslab, apparently soon after opening to traffic (within a few weeks). Figures B.9 and B.10 show typical cracked slabs. This type of distress was not expected for the Ohio repairs, because the load stresses that can cause such cracking are considerably lower than those that can cause transverse cracks. Most of the analysis conducted for this project focused on the assessment of fatigue damage due to early opening at the longitudinal edge, which is the critical location for fatigue damage. The longitudinal cracks were not observed during installation; however, it is possible that these cracks did occur within days of opening to traffic, but not observed. The results of the distress survey are summarized in Tables B.5 and B.6. The first repair in each section was not considered in the performance monitoring because so many cores were taken from them.

The exact cause of the longitudinal cracking in Ohio sections will be difficult to determine from the available data, but some of the likely causes have been identified. Table B.9 shows a summary of visual distress survey results along with factors that can affect performance of concrete repairs. It is suspected that cracking in Ohio sections initiated at the top of the repairs, caused by the combined effects of shrinkage, temperature stresses, and load stresses.

Table B.9 shows that, at least among Type III sections, the general trend appears to be that the higher the cement content, the higher the peak temperature in the repairs and the greater the chance of longitudinal cracking. The temperature in the ODOT section was higher than the VES section, because curing blankets were used in the ODOT section. With portland cement mixes, higher cement content will generally lead to higher shrinkage. All of the sections in Ohio were provided with dowels at the transverse joints. While these dowels allow free movement of repairs in the longitudinal direction, they would act to impede movement in the transverse direction. With movement in the transverse direction restricted, both shrinkage and volume change due to cooling would cause residual tensile stresses to develop in the repairs in the transverse direction. It is possible that these residual stresses, combined with night-time curling (high tensile stresses at the top of the slab) and load stresses, were high enough to cause longitudinal cracking.

Compressive force exerted on repairs by adjacent slabs are also known to cause longitudinalcracks in short repairs. Full-depth repairs placed in early morning can be subjected to high compressive stresses if a significant rise in the ambient temperature follows the hardening of the repair concrete. The expansion of adjacent slabs in response to a rise in temperature can exert a very high compressive force on the repairs, causing failures in some cases. This type of failure is usually a problem only for repairs shorter than 6 ft (1.8 m) on repairs made with conventional concrete mixes. Since early opening mixes typically set much faster than conventional mixes, the amount of adjacent slab movement the early opening repairs can absorb while the material is still plastic is much less than repairs made with conventional materials. It may be possible that the 6 ft (1.8 m) repair length is too short to avoid compressive failures caused by the expansion of adjacent pavement when early opening mixes are used.

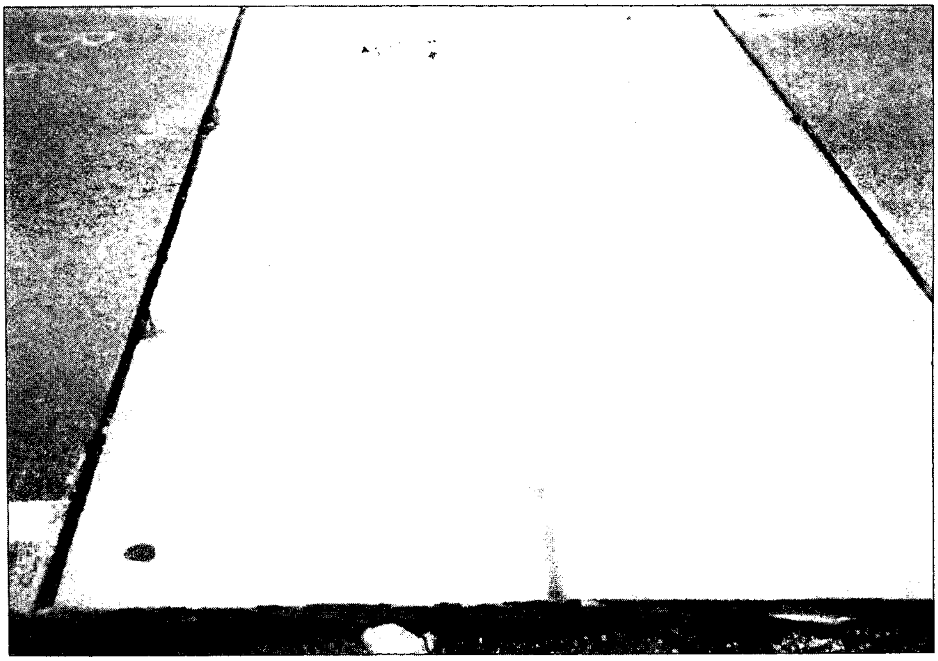

Figure B.9 Typical cracked repair on SR 2, Vermillion, OH

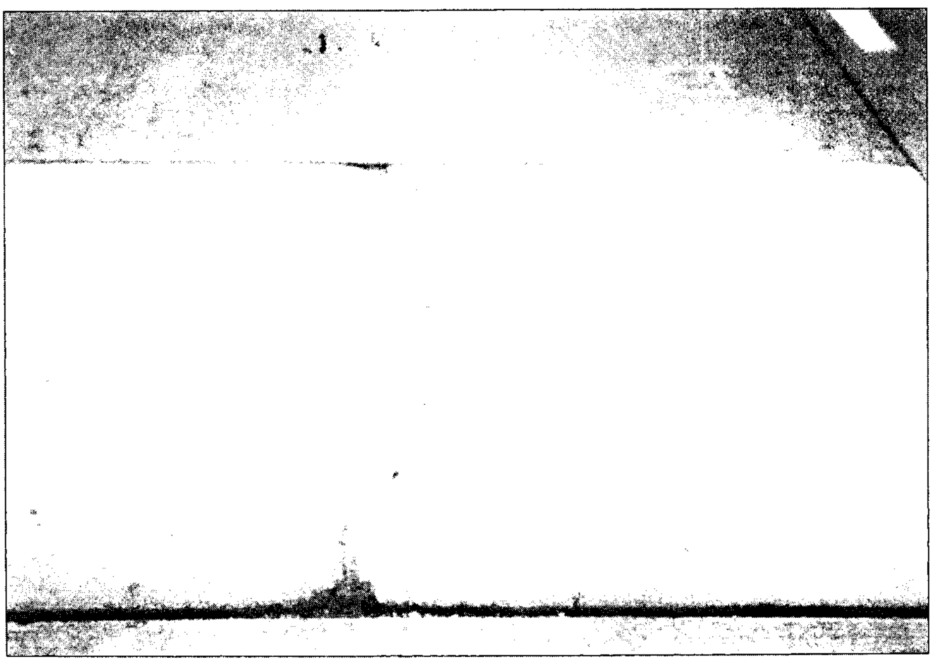

Figure B.10 Close-up of a typical crack on SR 2, Vermillion, OH

Table B.9. Summary of visual distress survey results of Ohio sections and factors affecting performance of concrete repairs.

Material Type	Maximum Slab Temperature (°F)	Maximum Temperature Difference (Top-Btm), (°F)	Cement Type	Cement Content (lb/cy)	w/c	Repairs Cracked (%)
ODOT	177	42	Type III	900	0.40	100
VES	157	24	Type III	915	0.45	56
HES	145	3	Type III	870	0.41	33
Fast Track	130	8	Type III	750	0.49	22
RSC 1	127	13	Rapid Set	750	0.40	89
RSC 2	129	23	Rapid Set	650	0.50	0
PC 1	108	13	Pyrament	900	0.27	11
PC 2	108	14	Pyrament	850	0.29	11

Note: °F = °C * 1.8 + 32; 1.8 °F = 1 °C; 1 lb/cy = .5932 kg/m³

This type of failure was not anticipated, so the type of measurements that would have been helpful in determining the cause of failure were not taken during installation. Further investigation and analysis are needed to determine the actual cause of the longitudinal cracks in the Ohio sections.

The longitudinal cracks observed in Ohio are not related to fatigue damage at the shoulder edge. They are caused by a different mechanism; thus, they should not affect fatigue studies of concrete repairs subjected to early opening.

Data Analysis

The assessment of the effects of early opening on the performance of concrete repairs was made based on fatigue analysis using the Miner's approach. The effect of concrete strength at opening on dowel bearing stress was also examined.

Fatigue Analysis

The objective of performing fatigue analysis in this project was to determine the strength needed at opening time to ensure adequate long-term performance. This can be achieved by determining:

- Fatigue damage attributable to early opening.
- Expected service life.
- Fatigue damage allowable for early opening.

The following describes how this analysis was performed and how the results may be used to determine the strength needed for early opening. A brief description of the models used in the analysis is given first, followed by a description of input data reduction, and then, finally, the analysis procedure is described.

Fatigue Model

Fatigue damage, according to Miner's theory, is calculated by summing the ratio of the number of applied loadings to the number of allowable loadings:

$$FD = \sum \frac{n}{N} \tag{B.1}$$

where:

FD = Fatigue Damage
n = number of applied 18-kip (80 kN) single axle loads
N = number of allowable 18-kip (80 kN) single axle loads

The allowable number of load repetitions (N) was determined using the following fatigue model (Salsilli 1991):

$$Log\ (N) = 2.81\ \left(\frac{M_R}{\sigma}\right)^{1.22} \tag{B.2}$$

where:

N = number of allowable ESAL applications
M_R = 28-day concrete modulus of rupture (third-point)
σ = maximum stress in the repair

This model was developed utilizing actual field performance data from Corps of Engineers test sections and the AASHO Road Test.

A transverse cracking model developed based on 52 in-service jointed concrete pavement sections is given below (Smith et al. 1990):

$$P = \frac{1}{0.01 + 0.03\ [20^{-\log\ (n/N)}]} \tag{B.3}$$

where:

P = Percent of slab cracked
n = Actual number of ESAL applications at slab edge
N = Allowable ESAL applications (from Eq. B.2)

According to this model, 25% slab cracking can be expected when n/N reaches 1.0.

Traffic Data Reduction

Figures B.4 and B.8 show the number of traffic passes (in ESALs) applied to the test sections at each test site during the consideration period for early fatigue damage assessment. In order to perform fatigue analysis, the number of traffic passes that actually loaded the repairs at the critical locations for fatigue has to be determined.

As a traffic wheel load passes over the pavement, it causes tensile stresses to develop at the bottom of the slab. At the edge locations, this stress is at a maximum directly under the load and quickly drops off to values that are insignificant for fatigue considerations at locations away from the loaded area. Figure B.11 shows the normalized stress distribution across the slab. Because of the high stresses under edge loading conditions, the longitudinal edge is typically the critical location for fatigue damage on concrete pavements.

In the fatigue analysis of pavement structures, pass to coverage (p/c) ratio is commonly used to model the number of fatigue load cycles caused by the applied traffic loads. The p/c is the number of ESAL passes that would cause one loading at the critical fatigue location. It represents the probability that the load will pass through the critical location. Brown and Thompson (Brown and Thompson, 1973) developed a statistical method for determining the p/c (originally for airfield pavements) that is now widely used.

$$C_x = \frac{1}{S_x \sqrt{2\pi}} e^{-0.5[(x-D)/S_x]^2} \qquad \text{(B.5)}$$

If the mean location of the wheel loads and the standard deviation of the wheel locations are known, the p/c can be expressed as:

$$p/c = 1 / (C_{Xc} * W_t) \qquad \text{(B.4)}$$

where:

- p/c = pass to coverage ratio
- C_{Xc} = maximum ordinate of the cumulative normal density function (C_x)
- W_t = tire contact width, in

and:

- C_x = normal density function
- x = mean location of the wheel load
- D = location in question
- S_x = standard deviation of the wheel location

Equation B.4 gives the number of ESAL loads passing within the tire contact width of the critical location for fatigue, assuming that these loads cause the same stress in the slab. This approach is valid on mature concrete, because the contribution to fatigue damage at the edge locations by the loads placed even a short distance away from the critical point is very small; however, on concrete pavements that are subjected to early loading, the contribution to fatigue damage due to the loads placed at adjacent locations can be significant. This is

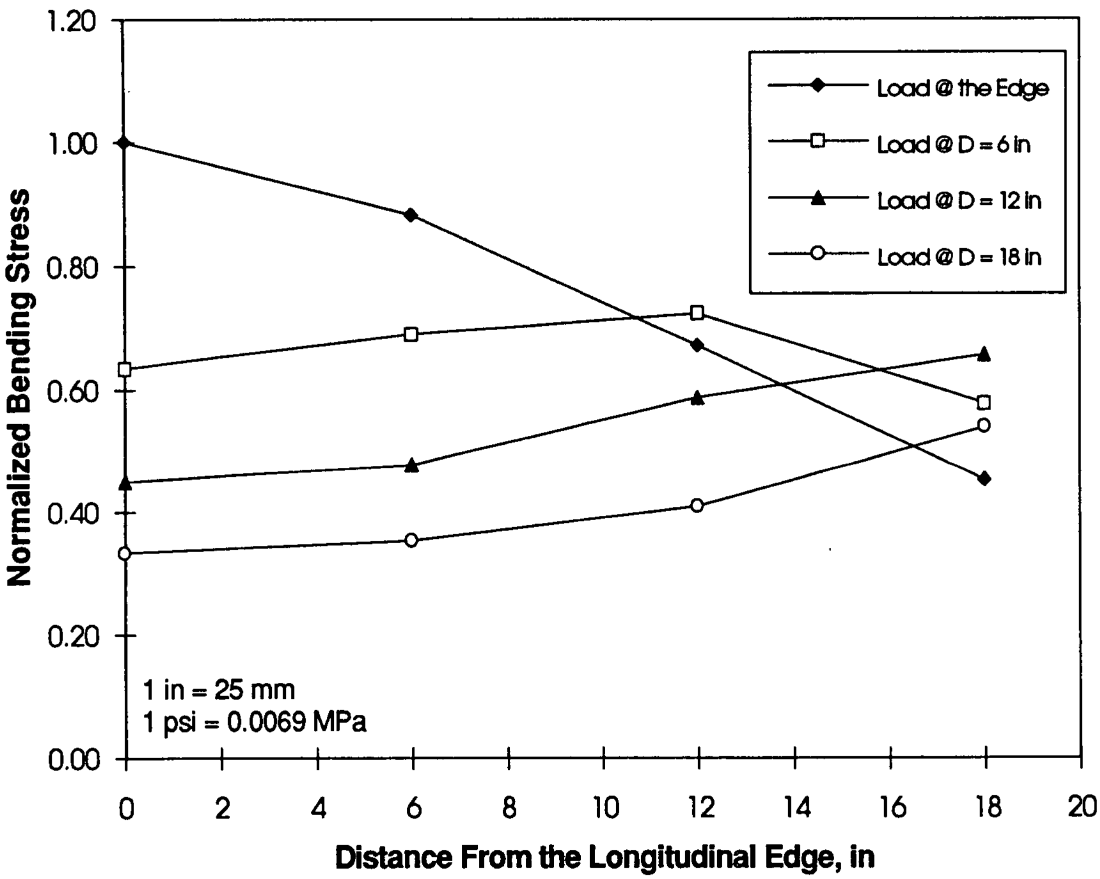

Figure B.11 Normalized edge stress distribution for various loading conditions, E = 3 MPsi (21 GPa).

because when the concrete strength is low, the stress to strength ratio can be relatively high even if the stress remains the same.

The concept of equivalent combined pass to coverage ratio (EPC), based on Brown and Thompson's p/c, was introduced to conveniently account for the effects of the loads placed at nearby locations on fatigue damage at a point. The p/c given by Equation B.4 approximates the probability that the wheel will pass through the critical location. In the EPC, the probability of the truck wheels passing through different locations across the pavement is multiplied by the appropriate weighting factor to obtain the equivalent p/c. The weighting factors are determined based on the stress levels at the critical location caused by the loads passing through different locations. The expression for EPC is given by Equation B.6, which reduces to Equation B.8.

$$\overline{p/c_{Di}} = \frac{FD_{Dii}}{\sum_j P(COV_{Dj}) * FD_{Dij}} \tag{B.6}$$

where:

FD_{Dii} = fatigue damage @ location D_i due to the load @ D_i
$P(COV_{Dj})$ = probability of coverage @ location D_j
FD_{Dij} = fatigue damage @ location D_i due to the load @ D_j

$$\overline{p/c_{Di}} = \frac{1/N_{Dii}}{\sum_j P(COV_{Dj}) * 1/N_{Dij}} \tag{B.7}$$

$$\overline{p/c_{Di}} = \frac{1}{\sum_j P(COV_{Dj}) * \frac{N_{Dii}}{N_{Dij}}} \tag{B.8}$$

where:

N_{Dii} = N allowable based on stress @ location D_i due to the load @ D_i
$P(COV_{Dj})$ = probability of coverage @ location D_j
N_{Dij} = N allowable based on stress @ location D_i due to the load @ D_j

Figure B.12 shows the effects of concrete strength on p/c at the two test sites. The large difference in the p/c at the test sites is due to the differences in the mean wheel locations (22.8 in. versus 18.9 in. [580 mm versus 480 mm]). The standard deviation of the lateral wheel location was assumed to be 10 in. (254 mm). This is the commonly accepted value for lateral traffic wander of highway truck traffic.

Although it was possible to determine the average wheel locations at both test sites based on field observations, the standard deviation of the wheel location could not be reliably determined because of the errors made during observation. In collecting the lateral wander data, the closest approach of any truck wheel to the edge was recorded rather than the lateral wander of a specific wheel (such as the first drive axle). While this gave a conservative estimate of the average wheel locations, it had the effect of reducing the standard deviation of the data collected.

Once the EPC is determined for different locations, the critical location for fatigue can be determined by examining the fatigue damage per load cycle at various locations. Figure B.13 shows the effect of concrete strength on fatigue damage per load cycle. As shown in the figure, at very low strengths, the fatigue damage at interior locations can be critical, but the fatigue damage per pass quickly drops off at the interior locations as the strength increases.

Material Data Reduction

The strength development curves for the materials evaluated in this experiment were constructed based on laboratory and field testing data. This information was needed to perform fatigue analysis. The strength development of the test sections during installation was monitored through both nondestructive (NDT) and destructive testing methods. The two NDT methods used were maturity and pulse-velocity. A number of cylinders and cores were also tested in the field to verify the results given by the NDT procedures; however, none of the destructive tests conducted in the field tested the modulus of rupture. Thus, when the different NDT methods predicted widely different strengths, there were no direct means available to determine what the true strength was. The strength predicted by different NDT methods in many cases were significantly different (by as much as a factor of 2), which created considerable problems in establishing the fatigue strength (modulus of rupture) development with time.

In establishing the strength development curve, it was assumed that the most reliable data available were the cylinder test data. Either insulated cylinder or temperature-matched curing cylinders gave good correlation to the in-place strengths (verified by the cores and NDT). Thus, it was possible to establish the compressive strength development vs. time relationship with a reasonable level of confidence.

Prior to field testing, each of the materials to be used were tested in the laboratory to determine the correlation relationships for the NDT. Using this information, it is possible to relate modulus of rupture to compressive strengths; however, one of the problems encountered in using the NDT correlation was that the correlation for the NDTs were based on a limited amount of test data. Some of the NDT relations were valid only for low strengths (compressive strength of 2,000 psi or less [13.8 MPa]) and others only for high strengths (3,000 psi or more [20.7]). This resulted in the NDT relations giving unreasonably high or low values of modulus of rupture in certain strength ranges.

To overcome this problem, linear regression was performed using the strength values that are in the valid range of the NDT correlation. Using the NDT correlation equations, squareroot of compressive strength values were correlated to the corresponding values of moduls of rupture. Figure B.14 shows the correlation for the Georgia materials and Figure B.15 shows

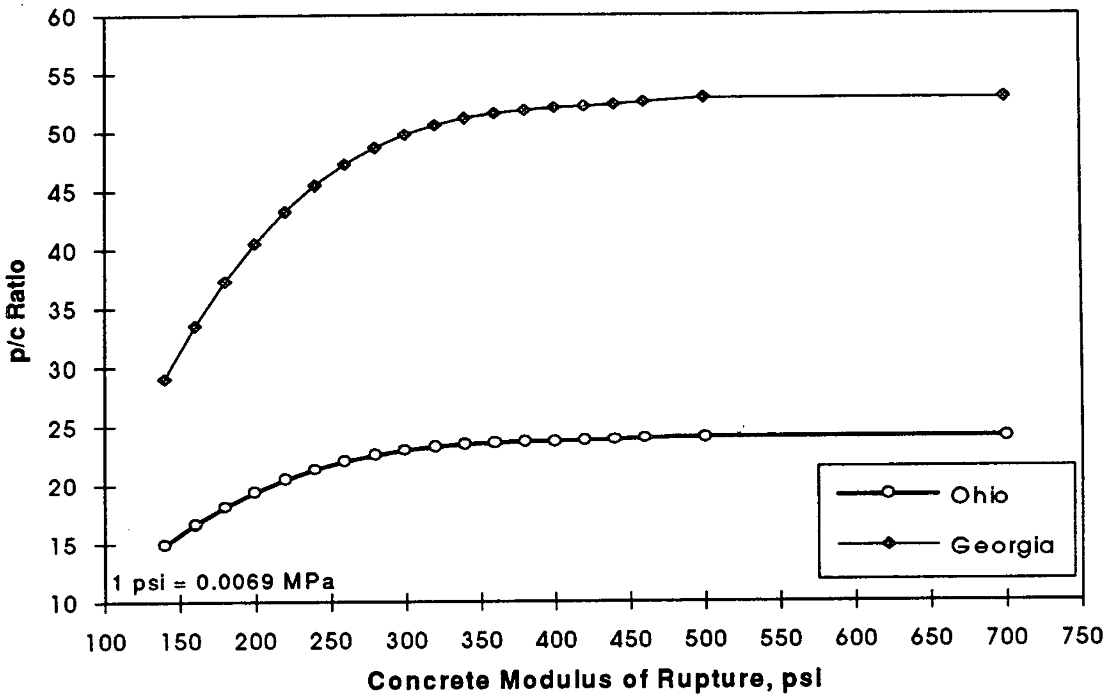

Figure B.12 Effects of concrete strength on p/c ratio.

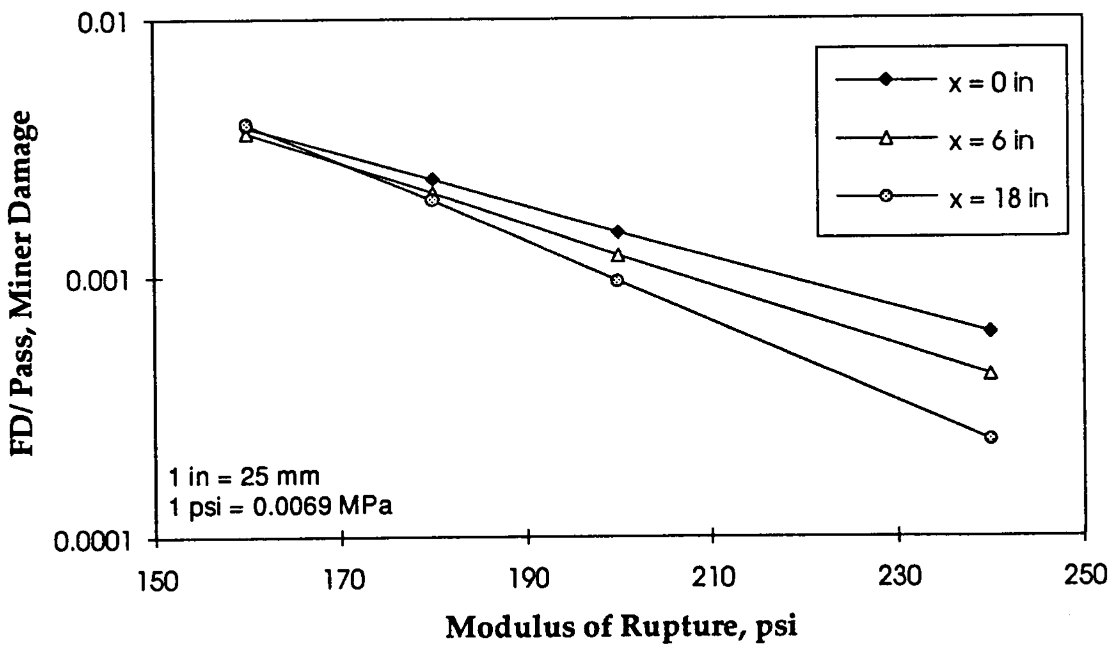

Figure B.13 Effect of concrete modulus of rupture on fatigue damage at various locations.

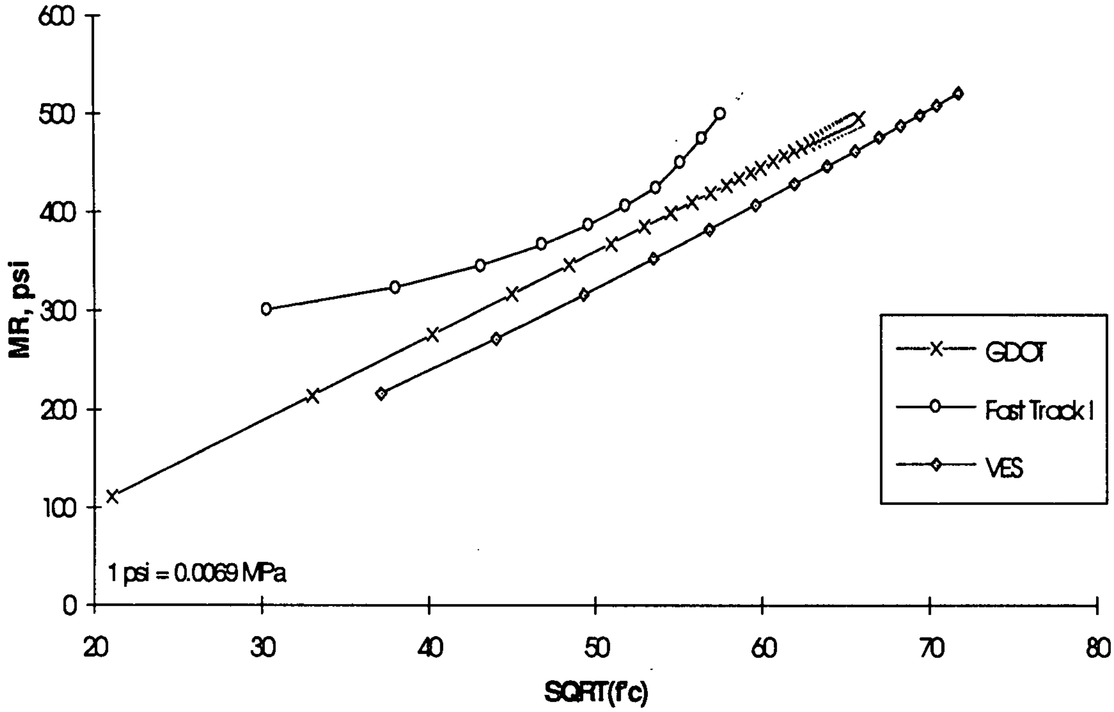

Figure B.14 F'c and modulus of rupture correlation for Georgia materials based on Arrhenius maturity.

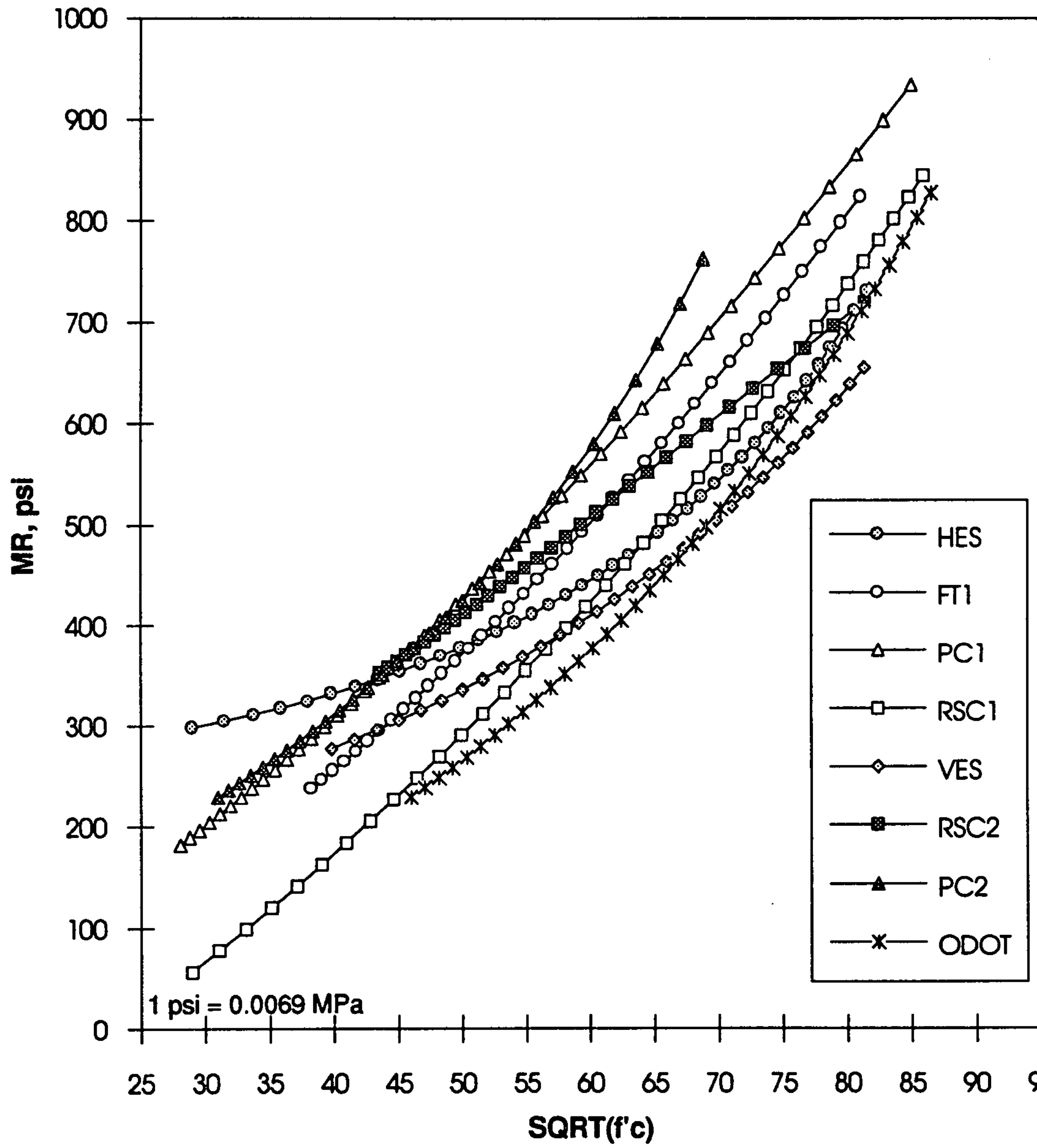

Figure B.15 F'c and modulus of rupture correlation for Ohio materials based on Pulse Velocity.

the relationships for the Ohio materials. Linear regression was performed on this data to establish the relationship between the modulus of rupture and compressive strength for the values outside the valid range of NDT correlation.

The modulus of rupture development curves for the Georgia materials are shown in Figures B.16 and B.17; those for the Ohio materials are shown in Figures B.18 through B.21. These curves were used in the fatigue analysis, and to estimate the strength of individual repairs at opening.

Climatic Data

The temperature gradients through the slab needed for fatigue analysis were generated using the Climatic-Materials-Structural (CMS) model developed for the FHWA (Dempsey 1983). This model is capable of producing hourly temperature and moisture gradients through the pavement structure, for any period, given the material properties and climatic data that can be obtained from the National Climatic Data Center.

Using this model, the temperature gradients through the slab during the period considered for early fatigue damage were obtained. The annual frequency histogram of hourly temperature differences through the slab was also obtained for both test sites using the CMS model. Figures B.22 and B.23 show the histograms. These values were used in the fatigue analysis to model curling stresses.

Fatigue Damage Due to Early Opening

Since typical curing periods for conventional concrete paving projects is 5 to 14 days, the fatigue damage accumulated during the first 14 days of service after opening to traffic was considered the fatigue damage attributable to early opening in this study. The fatigue damage accumulated during this period was determined on an hourly basis for the first 3 days, and then on a daily basis for the next 11 days.

Both the load and temperature stresses were considered in determining the fatigue damage. The combined load and curling stresses were determined using regression equations developed by Salsilli (Salsilli 1991). Salsili developed regression equations to give the maximum edge stresses in a concrete slab given the pavement structure, subgrade properties, applied load, and temperature gradient. These equations were developed using the results of extensive ILLI-SLAB finite element analysis runs; the equations very closely reproduce the results given by Illi-Slab.

Each period considered in the fatigue analysis presents a special case for stress analysis, since the elastic modulus of concrete and the temperature gradients are changing each hour. A large number of computer runs would have been required to determine the critical stresses in the slab, even if the number of cases considered were reduced by using average values. The use of Salsilli's equations greatly facilitates the analysis process. The regression equations allowed all of the calculations for the fatigue analysis to be carried out on a spreadsheet.

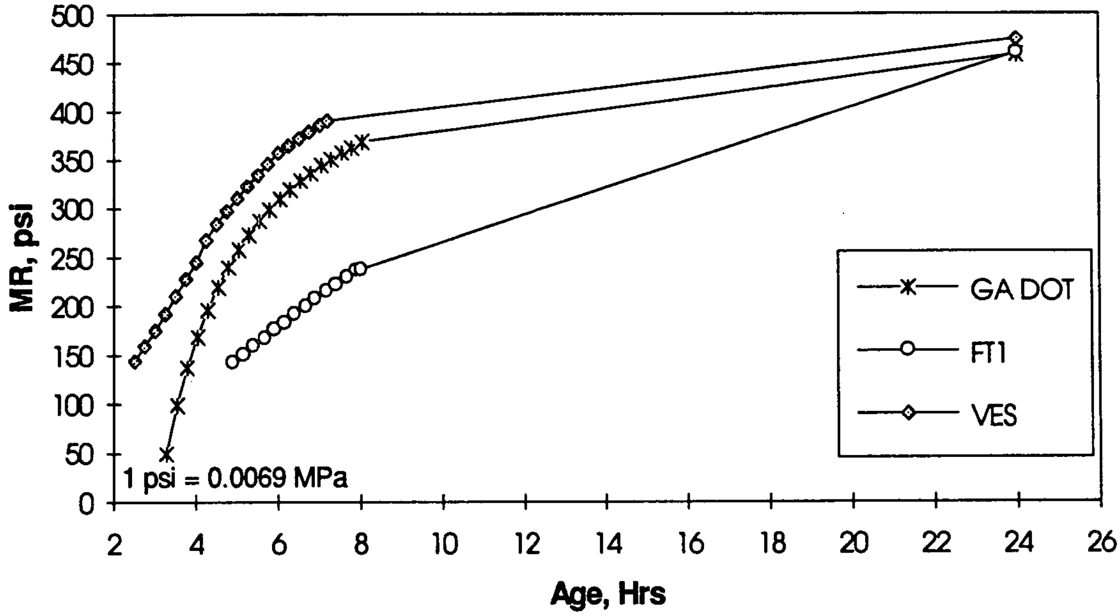

Figure B.16 Strength development of GA materials through 24 hours.

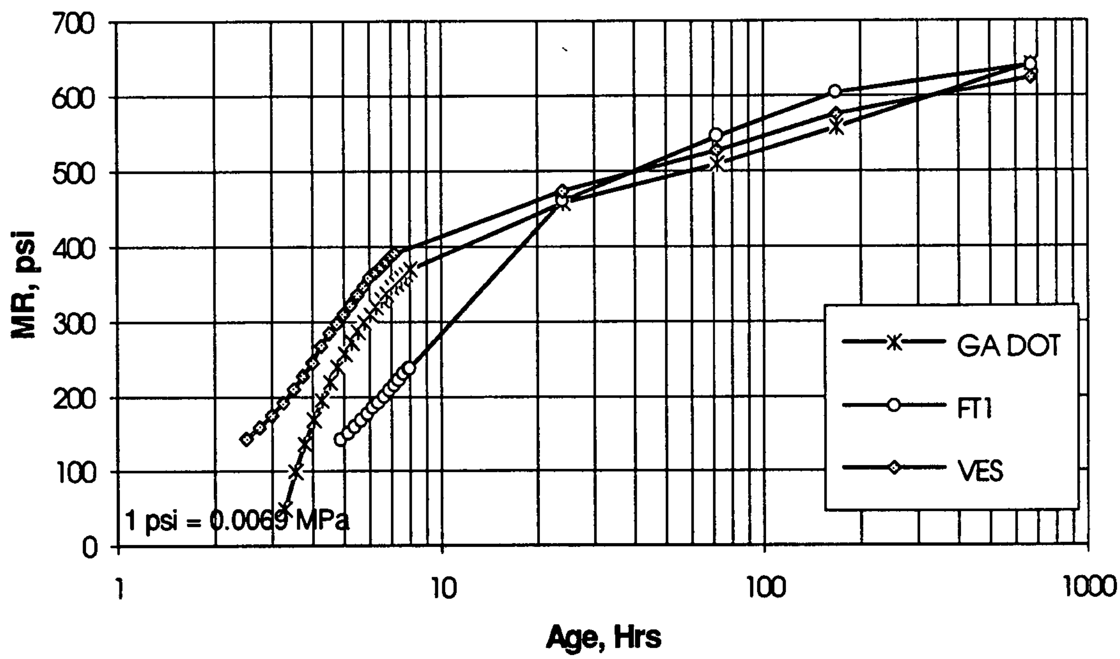

Figure B.17 Strength development of GA materials through 28 days.

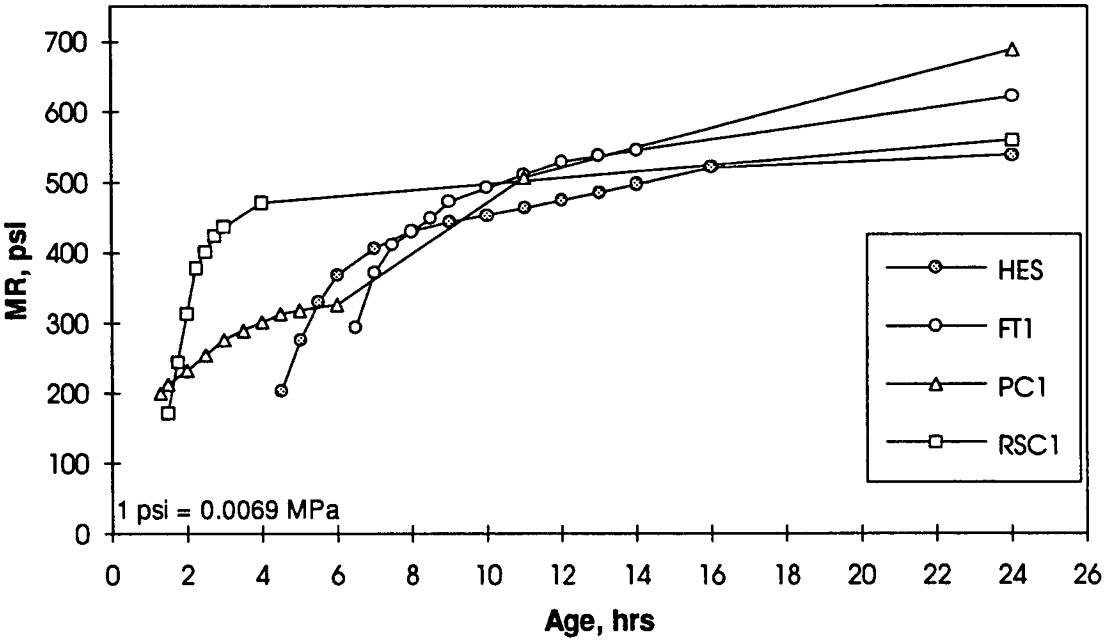

Figure B.18 Strength development of Ohio materials through 24 hours, eastbound sections.

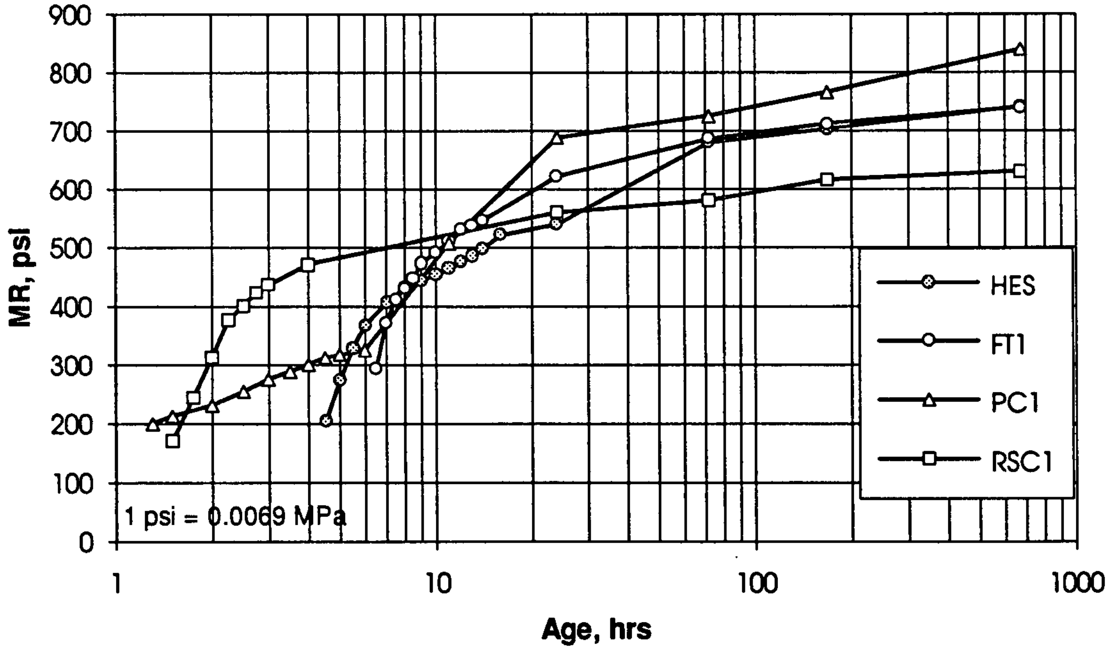

Figure B.19 Strength development of Ohio materials through 28 days, eastbound sections.

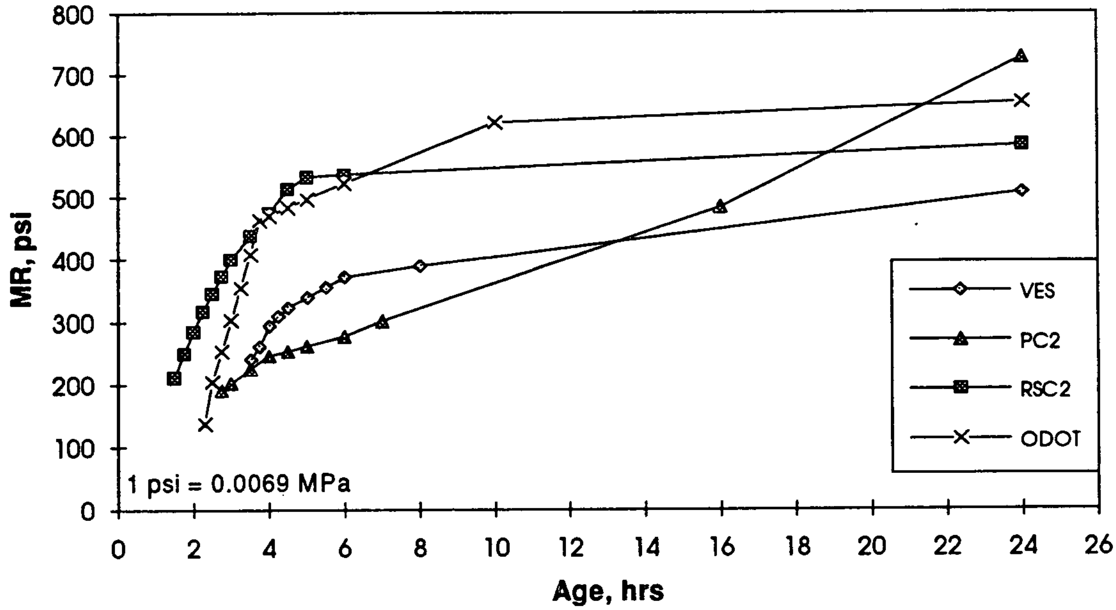

Figure B.20 Strength development of Ohio materials through 24 hours, westbound sections.

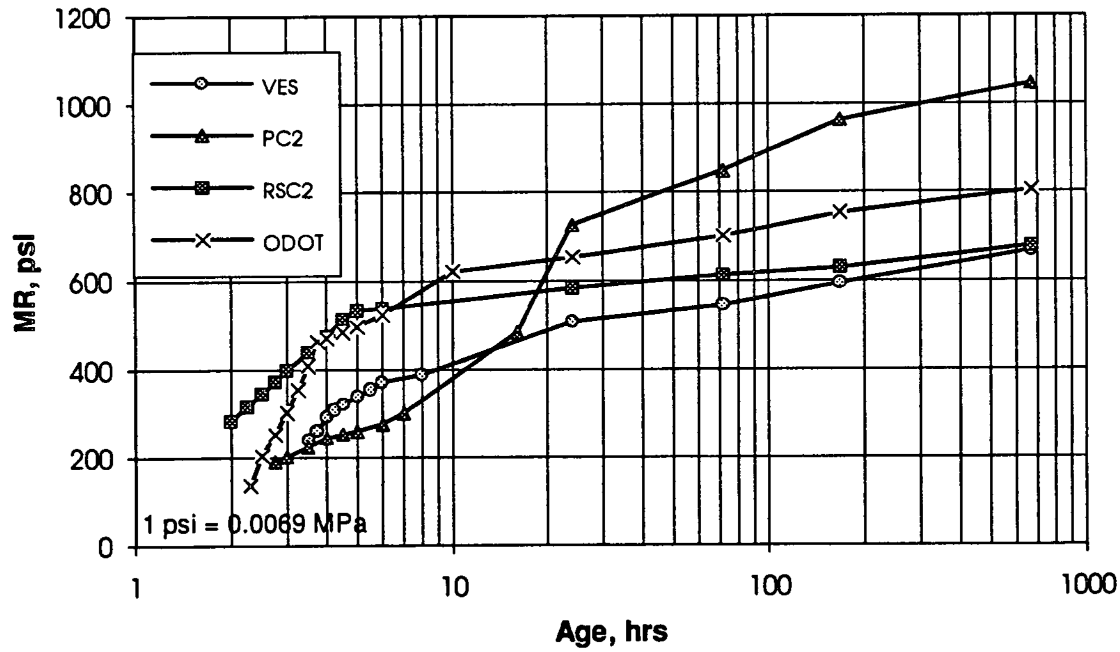

Figure B.21 Strength development of Ohio materials through 28 days, westbound sections.

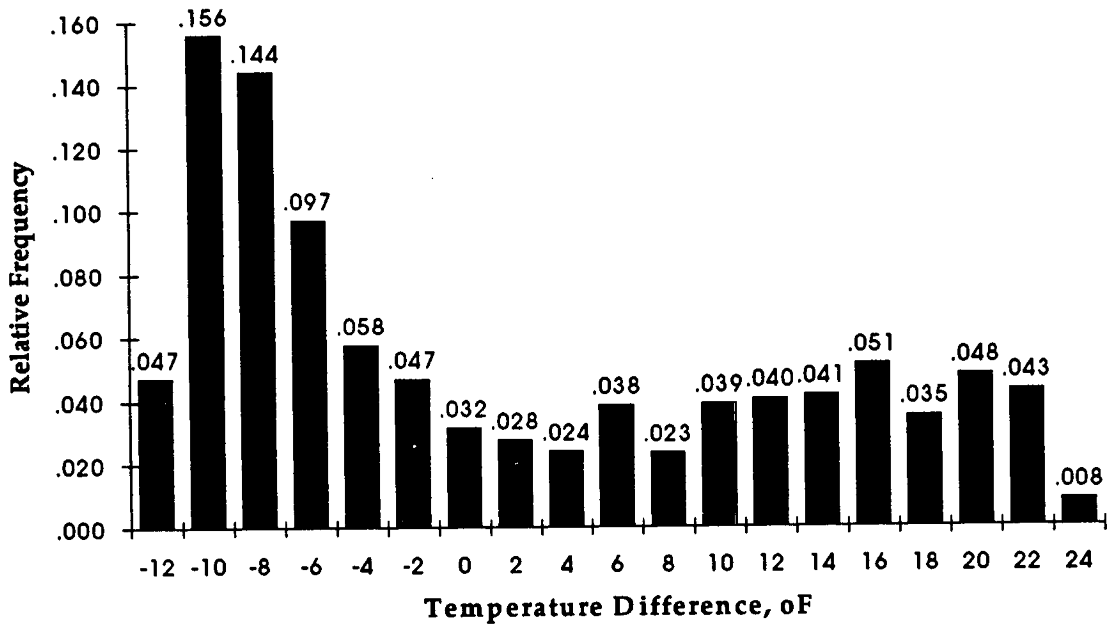

Figure B.22. Relative annual frequency histogram of hourly temperature difference through the slab for I-20, Augusta, GA, based on 30-year climatic data (°F = °C·1.8 + 32).

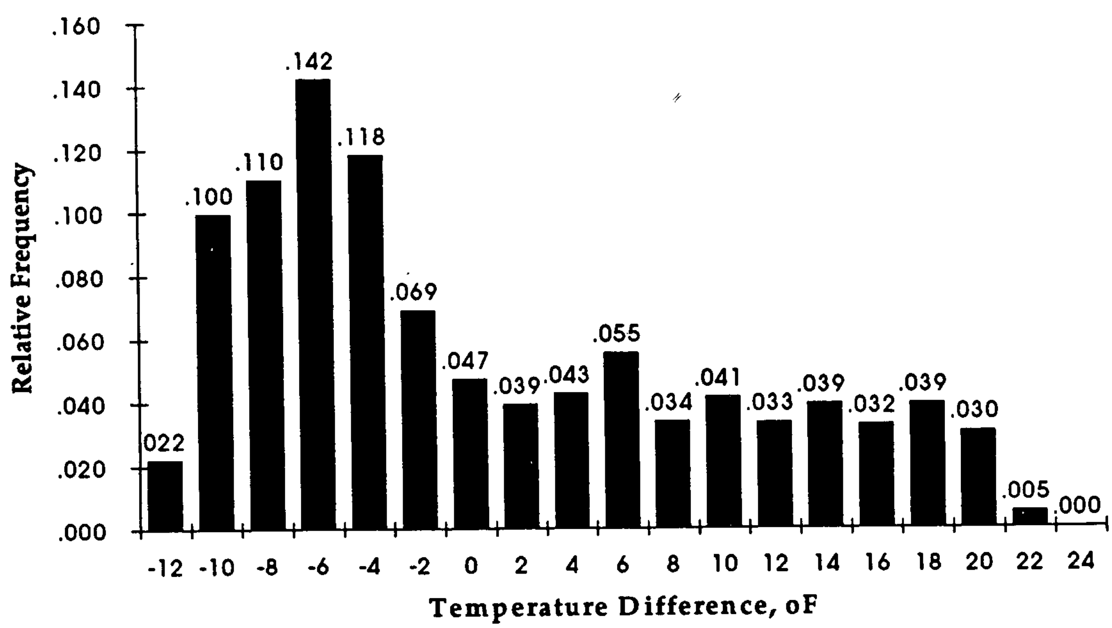

Figure B.23. Relative annual frequency histogram of hourly temperature difference through the slab for SR 2, Vermilion, Ohio, based on 30-year climatic data.

Once the critical edge stress during each period (hour/day) is determined, the fatigue damage incurred in that period can be easily determined. All of the other data needed for fatigue analysis have already been tabulated (the hourly material properties and traffic). First, the allowable number of load applications for the period, N, is calculated using Equation B.2. The fatigue damage incurred in that period is simply the number of loads applied during that period divided by the allowable number of load applications (n/N). The total fatigue damage attributable to early opening can then be determined by summing contribution to accumulated fatigue damage from each period in the period under consideration. The fatigue damage attributable to early opening for each of the repairs placed under this project is shown in Table B.2 for the Georgia repairs and Tables B.5 and B.6 for the Ohio repairs.

Figure B.24 illustrates the effect of the strength at opening and material type on fatigue damage due to early opening. Because of curling stresses, slab length is a significant factor in fatigue considerations. The 12-ft (3.7 m) repair length was used in Figure B.24 for all materials to show the effects of material type on early fatigue damages. As shown in Figure B.24, for the materials tested in Georgia, the fatigue damage due to early opening is negligible if the third-point modulus of rupture at opening is 250 psi (1.7 MPa) or more.

Expected Service Life

The expected service life was determined considering both load and temperature stresses. The annual frequency histogram of hourly temperature differences through the slab (obtained using CMS) was used in conjunction with Salsilli's regression equations to determine combined load and temperature stresses. The number of ESAL applications for each of the temperature ranges in the temperature gradient histogram was determined by multiplying the total annual ESAL applications for the design lane by the frequency of occurrence of the temperature range. This assumes that the truck traffic is evenly distributed throughout the year, but the error introduced by this assumption is not significant. The fatigue damage caused per year was then determined by summing fatigue damage given by the load applications in each temperature category. Figure B.25 shows the fatigue damage per year for Georgia materials. Again, the effect of slab length on fatigue damage is illustrated. Because of low curling stresses, the fatigue damage on small repairs is negligible.

The expected service life is simply the allowable level of fatigue damage, less any fatigue life consumed for early opening, divided by fatigue damage per year. The allowable level of fatigue damage can be established considering the cracking level given by Equation B.3. Once the allowable level of fatigue damage has been established, the expected service life can be determined using the following expression:

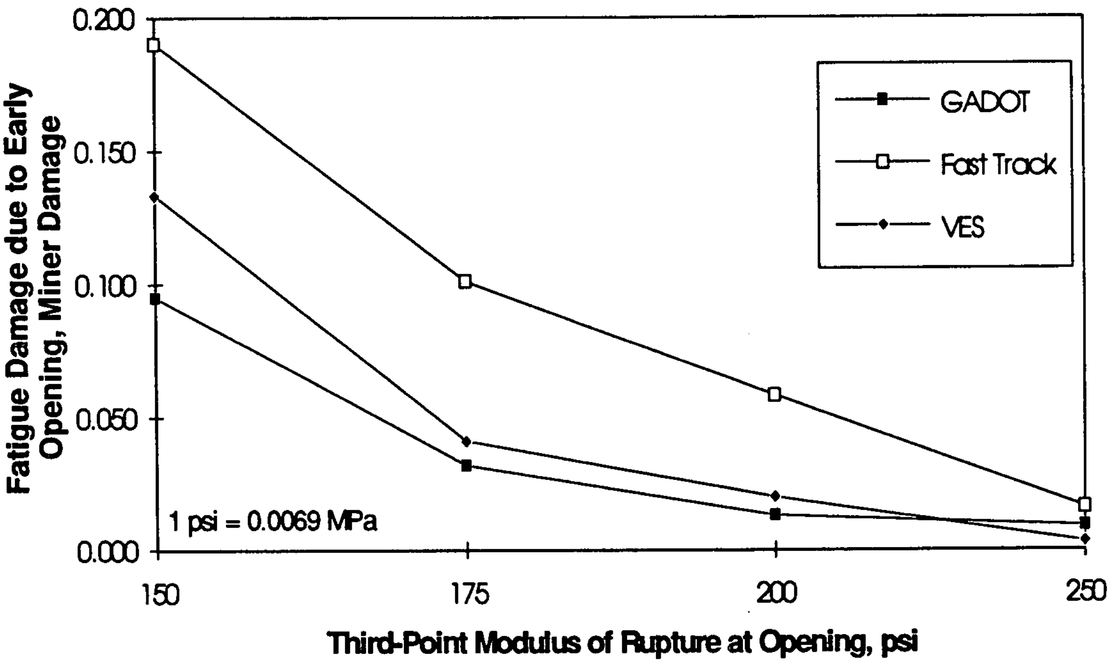

Figure B.24 Fatigue damage due to early opening for GA Materials on 12 ft (3.7 m) repairs.

$$ESL = \frac{FD - fd_{eo}}{fd_{yr}} \tag{B.9}$$

where:

- ESL = Expected Service Life
- FD = Allowable fatigue damage
- fd_{eo} = Fatigue damage due to early opening
- fd_{yr} = Fatigue damage per year

For example, according to Equation B.3, about 25% slab cracking could be expected at the accumulated fatigue damage level of 1.0. If this level of damage is selected as the allowable fatigue damage and if fd_{yr} were found to be 0.02, then the expected service life of a normal-opening concrete repair ($fd_{eo} = 0$) would be:

$$ESL = \frac{1}{.02} = 50 \ years$$

If this repair were opened early to traffic and the fatigue damage attributable to early opening were 0.20, then the expected service life would be:

$$ESL = \frac{1 - .20}{.02} = 40 \ years$$

Figure B.26 shows expected fatigue life for Georgia materials. The figure shows that if the repair length is 8 ft (2.4 m) or less, practically unlimited fatigue life can be expected.

Allowable Fatigue Damage for Early Opening

If the required service life, allowable fatigue damage (FD), and fatigue damage per year (fd_{yr}) are known, then the allowable fatigue damage for early opening (fd_{eo}) can be determined as follows:

$$fd_{eo} = FD - RSL \cdot fd_{yr} \tag{B.10}$$

where:

- fd_{eo} = Fatigue damage allowable for early opening
- FD = Allowable fatigue damage
- RSL = Required Service Life
- fd_{yr} = Fatigue damage per year

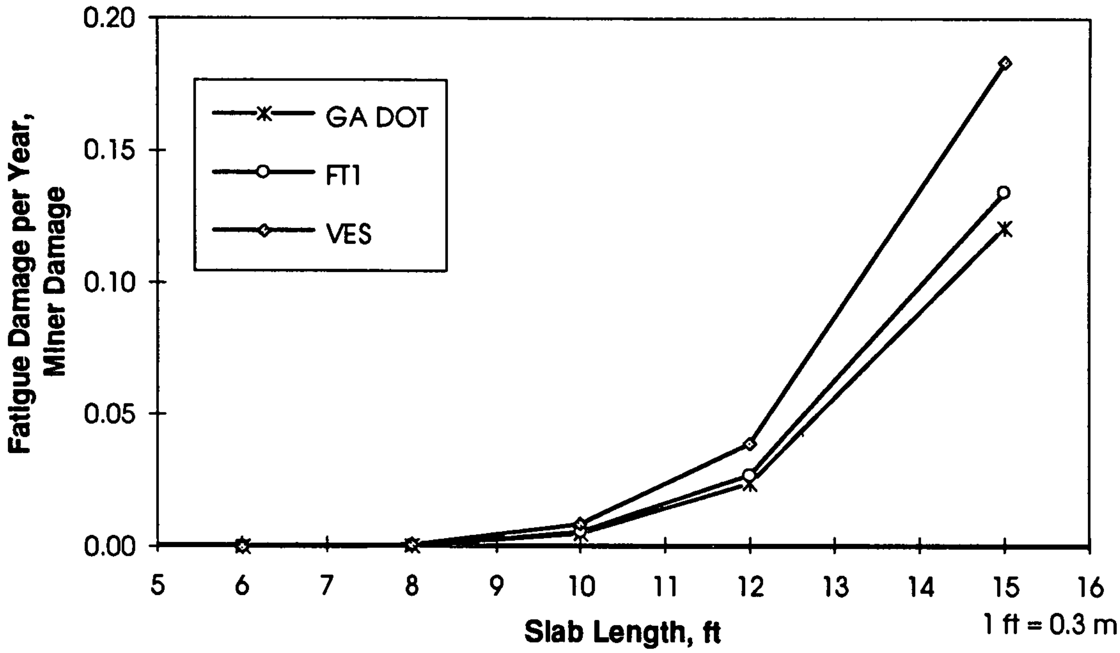

Figure B.25 Effect of slab length on fatigue damage.

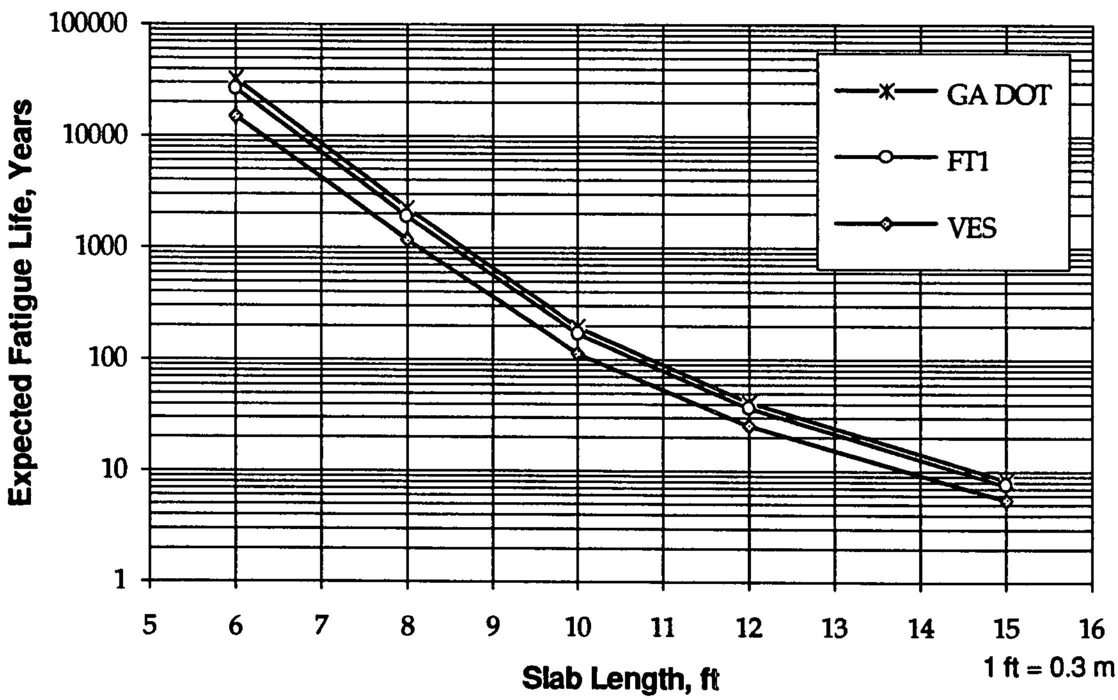

Figure B.26 Effect of slab length on expected service life.

For example, if the allowable fatigue damage is 1.0, fd_{yr} is 0.02, and the desired service life of the repair is 15 years, then fd_{eo} is:

$$fd_{eo} = 1.0 - 15 \cdot .02 = .70$$

For short repairs, the fd_{yr} is negligible. The fd_{yr} of the materials placed in the Ohio sections is 1.29×10^{-4} or less. This suggests that virtually all of the fatigue life may be used for early opening (i.e., fd_{eo} is nearly 1.0); however, it is important to recognize that Equations B.9 and B.10 are valid only if Miner's hypothesis is valid in the case of early opening projects.

While the linear fatigue accumulation model works reasonably well for mature concrete, the applicability of this model to concrete subjected to early loading has not been demonstrated. In an early opening project, the repairs are subjected to at least two conditions that are unique to this practice:

- The repairs subjected to early loading generally experience much greater amounts of fatigue damage over a short period. It is not known whether the damage incurred over such a short period (matter of days) has the same effect on fatigue life as the damage incurred over a much longer period (10 years or more).

- The material properties are changing rapidly at the same time that the repairs are being loaded. Also, because the hydration process is still active in the repair material for some time (that can range from hours to days) after the repairs are opened to traffic, it may be possible for some autogenous healing to take place.

Whether or not the fatigue models and expected service life predictions discussed in this report are valid can only be answered when the performance data become available.

Dowel Bearing Stress

For doweled concrete repairs subjected to early opening, the maximum dowel bearing stress can be the critical factor controlling the opening time, especially for short repairs. A modified Friberg analysis was used to determine the maximum dowel bearing stress. The dowel bearing stress is given by the formula (Heinrichs 1989):

$$\sigma_{\max} = G \bullet \delta_o \tag{4.11}$$

where:

G = Modulus of dowel support, psi/in.
δ_o = Deflection of the dowel at the face of the joint, in.
= $P_t (2 + \beta z) / 4\beta^3 E_s I$

in which

P_t = Shear force acting on dowel, lb
= Wheel load · percent load transferred · dowel distribution factor
z = Width of joint opening, in.
E_s = Modulus of elasticity of dowel bar, psi
I = Moment of inertia of dowel bar, in.^{4}
= $0.25 \cdot \pi \cdot (d/2)^4$ for a solid dowels with diameter d inches
β = Relative stiffness of dowel concrete system, 1/in.
= $(GD / 4E_sI)^{0.25}$

Okamoto (1992) investigated the modulus of dowel support, G, for concrete at early ages. Figure B.27 shows the values obtained from that investigation.

In order to determine the shear load acting on a dowel, P_t, the percent load transferred across the joint and dowel distribution factor are needed. The dowel distribution factor was determined using the finite element program ILLI-SLAB. Figure B.28 shows the dowel distribution factors for the maximum loading condition. The most critical dowel carries about 64% of the transferred load. The percent load transferred across the joint typically varies from 30 to 45%. This corresponds to deflection load transfer of about 80 to 95 percent, respectively.

The dowel bearing stresses for the Ohio and Georgia sections at different concrete strengths are shown in Figure B.29. This is assuming 45% of a 10,000 lb (4,500 kg) wheel load transferred across the joint. The concrete modulus at the typical opening times was about 2.0 million psi (13.8 GPa). Figure B.29 shows that the compressive strength needed to prevent excessive dowel bearing stresses is between 1,700 to 2,300 psi (11.7 to 15.9 MPa), depending on the dowel diameter.

The dowel bearing stress is fairly sensitive to joint load-transfer efficiency. Figure B.30 shows the effects of joint load-transfer efficiency on dowel bearing stress. The figure shows that when the deflection load-transfer efficiency drops to 70 percent, the dowel bearing stresses are about 900 psi (6.2 MPa) for 1.5 in. (38 mm) dowels and 1,200 psi (8.3 MPa) for 1.25 in (32 mm) dowels. The measured deflection load-transfer efficiencies during the follow-up testing ranged from 59 to 89%. The average value at both test sites was 76%.

The data from the field experiment do not show any correlation between the compressive strength at opening and the load-transfer efficiency; however, this may only be an indication that construction factors were more significant to load-transfer efficiency in this case than the concrete strength. It is reasonable to expect the excessive dowel bearing stresses to cause looseness around the dowels. Since any looseness around the dowels will significantly reduce load-transfer efficiency, excessive dowel bearing stresses can be expected to have an adverse effect on load-transfer efficiency. This may be a more noticeable problem at higher levels of load-transfer efficiency.

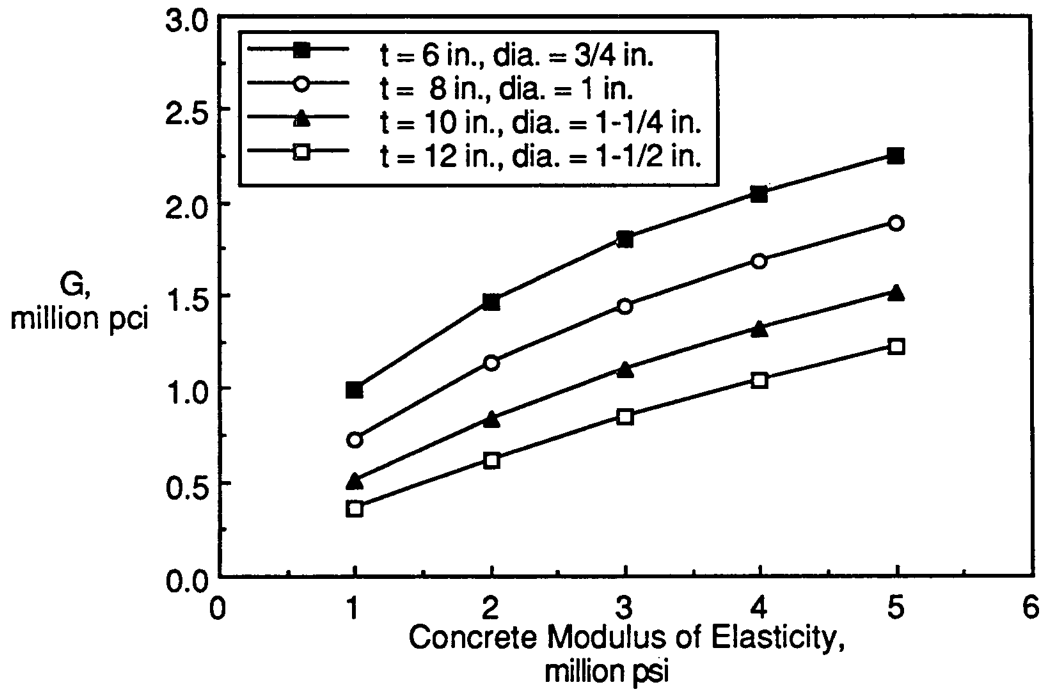

Figure B.27 Modulus of dowel support.

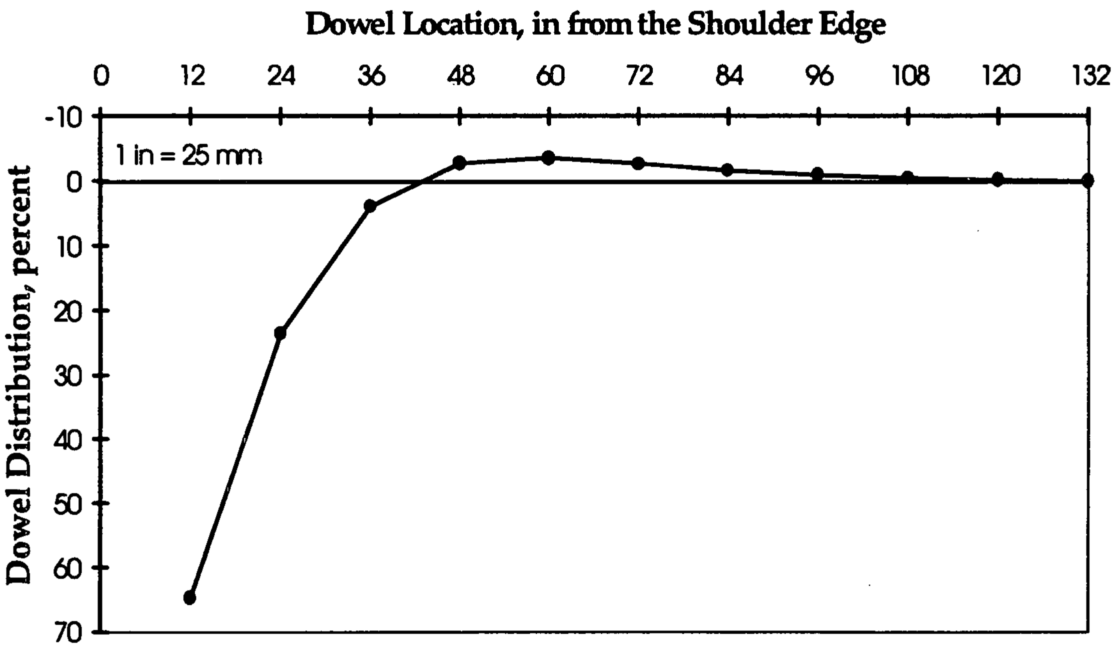

Figure B.28 Dowel distribution factors.

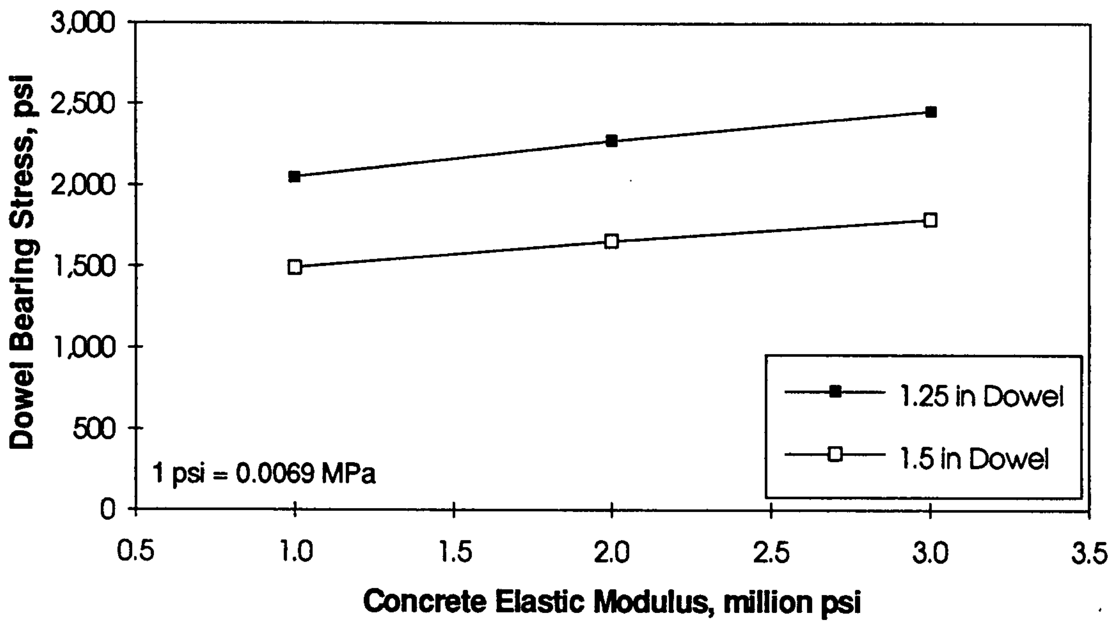

Figure B.29 Effects of concrete modulus of rupture and dowel diameter on dowel bearing stress.

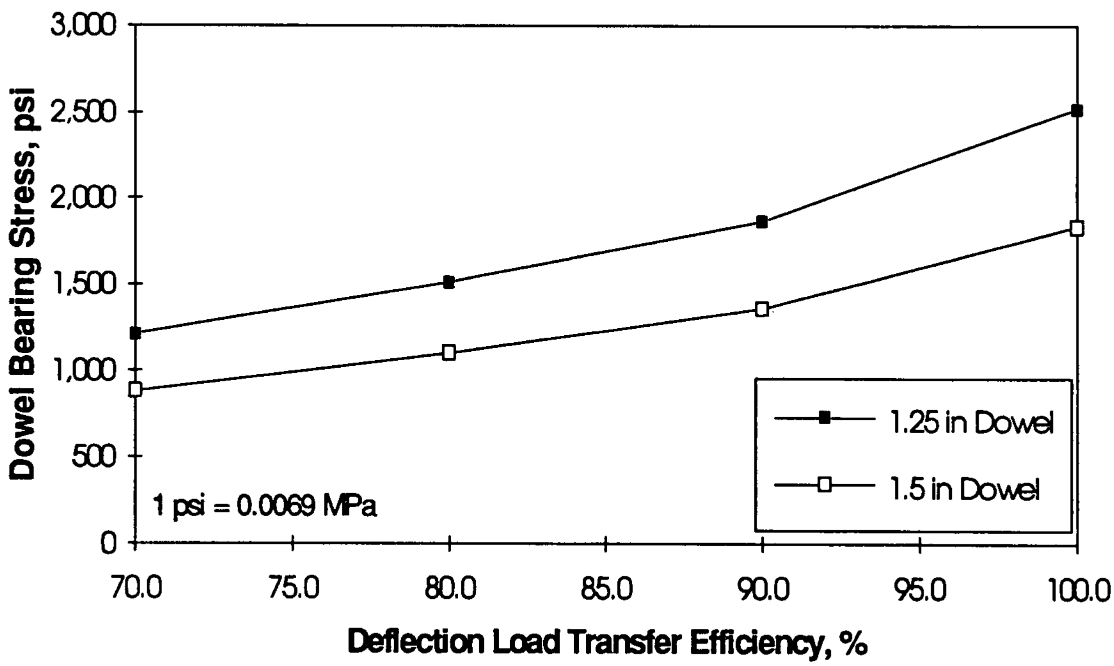

Figure B.30 Effects of load-transfer efficiency on dowel bearing stress.

Considerations for Early Opening of Concrete Repairs

Material Selection

Material selection is a very important consideration for an early opening project. It affects the ability to open the repairs to traffic within the desired time frame and to obtain the desired long-term performance. A material that gains strength very rapidly may allow opening to traffic earlier and at a lower strength than the one that gains strength more slowly. The long-term fatigue life of concrete repairs is determined by the ultimate, long-term fatigue strength (modulus of rupture) achieved by the material. If the material is capable of achieving a very high modulus of rupture, a grater proportion of its fatigue life could be used for early opening without compromising its long-term performance. Thus, the strength at opening, rate of strength gain, and the ultimate, long-term strength achieved by the material all affect the fatigue life of concrete repairs subjected to early opening, and these are all material-related factors.

Strength at Opening

The strength required for opening concrete repairs to traffic depends on both the material used and repair length. The repair length affects the strength required for opening because it influences the critical stresses in the repairs. As the repair size increases, the stresses due to temperature curling increase rapidly. There is also a slight increase in load stress as the repair size is increased. The effects of slab length on service life is shown in Figure B.26.

The amount of fatigue damage that can be used for early opening can be determined using Equation B.10. Once this is known, the strength required for opening can be determined by developing a chart similar to Figure B.24. For very short repairs (6 ft [1.8 m]), dowel bearing stress is more critical than modulus of rupture for early opening considerations. Even on longer repairs, the dowel bearing stress can often be more critical than the modulus of rupture. Hence, the opening to traffic should be based on both modulus of rupture and compressive strength.

Modulus of Rupture

For concrete repairs up to 12 ft (3.7 m) long, the opening criteria used by many states—center-point modulus of rupture of 300 psi (2.1 MPa)—is reasonable for fatigue considerations. The center-point modulus of rupture of 300 psi (2.1 MPa) corresponds to third-point modulus of rupture of about 250 psi (1.7 MPa). For longer repairs, the required strength should be established based on the results of analyses that consider the specific conditions of the project.

Compressive Strength

Doweled repairs should have adequate compressive strength at the time of opening to traffic to resist the dowel bearing stresses. The minimum compressive strengths needed are:

- 2,000 psi (13.8 MPa) for 1.5-in. (38-mm) diameter dowels
- 2,500 psi (17.2 MPa) for 1.25-in. (32-mm) diameter dowels

Conclusions and Recommendations

Based on the analyses performed for this project, the following conclusions are made:

1. The strength required for opening concrete repairs to traffic depends on both material type and repair length. In general, more rapid-setting mixes can be opened to traffic at a lower strength than slower mixes, and longer repairs require a higher strength for opening to traffic.

2. Dowel bearing stress can be a critical factor controlling the opening of concrete doweled repairs to traffic. For very short doweled repairs (6 ft [1.8 m]), dowel bearing stresses are more critical than concrete modulus of rupture for early opening considerations.

3. Opening to traffic should be based on both modulus of rupture and compressive strength.

   For concrete repairs up to 12 ft (3.7 m) long, the opening criteria used by many states—third-point modulus of rupture of 250 psi (1.7 MPa)—is reasonable. For longer repairs, the modulus of rupture required for early opening should be determined based on the results of an analysis that considers the specific local conditions, including the strength gain properties of the repair material, traffic, and temperature gradient.

   The compressive strengths needed for opening on doweled repairs are:

   - 2,000 psi (13.8 MPa) for 1.5-in. (38 mm) diameter dowels

   - 2,500 psi (17.2 MPa) for 1.25-in. (32 mm) diameter dowels

6. Continued monitoring of the test sections constructed for this study is very important. While the linear fatigue accumulation model works reasonably well for mature concrete, the applicability of this model to concrete subjected to early loading has not been demonstrated. Whether or not the fatigue models and expected service life predictions discussed in this report are valid can only be answered when the performance data become available.

   The monitoring should consist of:

   - Distress surveys—these should be conducted annually, until failure, to monitor:

     — Cracking
     — Faulting
     — Spalling

- Traffic—A record of ADT, truck factor, and the percent of traffic that was heavy trucks should be maintained.

7. Upon completion of data collection, or when sufficient data are available, the data should be analyzed to develop guidelines for early opening based on performance.

Appendix C

Proposed Standard Method of Test for Water Content of Freshly Mixed Concrete Using Microwave Oven Drying

Proposed Standard Method of Test
for
Water Content of Freshly Mixed Concrete Using Microwave Oven Drying

AASHTO DESIGNATION: TP 23
SHRP DESIGNATION: 2027

1. SCOPE

1.1 This method covers the measurement of total water content of fresh concrete. It consists of drying a defined sample of fresh concrete (1500 grams) using a relatively high power microwave oven within a period of time not to exceed 14 minutes. The difference in weight between the fresh and dry concrete sample will be the water content of that sample. Water content per unit volume of concrete can be determined by knowing the unit weight of that concrete.

1.2 The method is applicable to both laboratory and field tests as long as a power source is available.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever used this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. REFERENCED DOCUMENTS

2.1 AASHTO *Standards*

T 141	Sampling Fresh Concrete
T 121	Weight Per Cubic Foot, Yield, and Air Content (Gravimetric) of Concrete
T 231	Weights and Balances Used in the Testing of Highway Materials
T 84	Specific Gravity and Absorption of Fine Aggregate
T 85	Specific Gravity and Absorption of Coarse Aggregate

2.2 ASTM *Standards*

C 670	Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials

3. SIGNIFICANCE AND USE

3.1 This method covers the laboratory and field measurements of water content in fresh concrete. The test results have shown good correlation with the actual water contents in concrete.

3.2 The method can be used as a quality control tool to check the water content of as-delivered concrete which is usually subjected to change, and to calculate the actual water-to-cement ratio by knowing the cement content of the tested concrete.

3.3 If one desires to calculate the net water content of the concrete from results of this test, the absorption of coarse and fine aggregates used in the test concrete must be accurately known.

3.4 Drying time and precision statements are based on the specified weight of sample and a particular microwave oven. If larger samples or other types of microwave ovens are used, the necessary drying time may be longer.

4. APPARATUS AND MATERIALS

4.1 *Microwave oven*— A microwave oven with a power setting of 900 watts. A commercial microwave oven (SHARP Model R-9H93) with a capacity of 1.5 ft^3 (0.043 m^3) and turntable system has been found to be suitable.

4.2 *Glass tray*— Heat-resistant glass tray with dimensions of 9 × 9 × 2 inches (230 × 230 × 50 mm).

4.3 *Fiber glass cloth*— Plain weave fiber glass with 10 oz/yd^2 (0.34 kg/m^2) weight and thickness of 14 mils (0.35 mm).

4.4 *Scale*— A scale accurate to within 0.1% of the weight of the sample and having a capacity of 5000 gm.

4.5 *Small metal scraper*— Metal scraper having a sharp edge with 1 in. (25 mm) width blade.

4.6 *Grinding pestle*— A pestle having a porcelain grinding head with a diameter of 2 in. (50 mm).

5. TEST SPECIMENS

5.1 Obtain the sample of freshly mixed concrete in accordance with T 141.

5.2 Sample should weigh approximately 1500 grams (3.3 lb).

6. PROCEDURE

6.1 Cut a piece of fiber glass cloth large enough to wrap a 1500 gram sample of fresh concrete (20 in × 20 in. (510 × 510 mm) piece or larger).

6.2 Place the cloth on the tray having the cloth uniformly overhanging the edges of the tray.

6.2 Weigh the tray and cloth together. Record this and all subsequent weight to the nearest 1.0 gram or 0.05% of the sample weight, whichever is greater.

6.3 Leave the tray and cloth on the scale and add approximately 1500 grams of the test concrete and wrap it completely with the cloth.

6.4 Weigh the tray and cloth and fresh sample together.

6.5 Put the tray in the microwave, set it to the highest power (900 watts) and dry for 5 minutes.

6.6 At the end of 5 minutes of drying, take the tray out and quickly unwrap the sample. With the edge of the scraper, break the mass of concrete until coarse aggregates is separated from the mortar. Then grind the mortar with the pestle to expose more surface.

Note 1— This grinding process should be done carefully to avoid losing any material and should not take more than 1 minute.

6.7 Wrap the sample again and put it in the microwave oven for another 5 minutes.

6.8 At the end of the second 5 minute period take the tray out and unwrap the sample. Stir it with the scraper and then weigh it.

6.9 Wrap the sample again and put it in the microwave for another 2 minutes. Then take it out and weigh it. If the change in weight is less than 1 gram, stop drying; otherwise, dry for another 2 minutes.

Note 2— For most concretes, the required drying time is 12 to 14 minutes. If a low capacity microwave oven is used, longer times may be needed for complete drying.

7. CALCULATIONS

7.1 Water Content Percentage— Calculate the water content percentage as follows:

$$WC\ (\%) = 100 \times (WF - WD) / WS$$

Where:

WC = water content percentage
WD = weight of tray + cloth + dry sample
WF = weight of tray + cloth + fresh sample
WS = weight of fresh sample

7.2 Total Water Content— Calculate the total water content as follows:

$$WT = 27 \times WC \times UW / 100$$

Where:

WT = total water content, lb/yd^3 (kg/m^3)
UW = unit weight of fresh concrete lb/ft^3 (kg/m^3)
27 = a constant (not used for SI calculation).

8. PRECISION—The single operator within-lab standard deviation has been found to be 2.7 lb/yd^3 (1.6 kg/m^3) (1s). Therefore, results of two properly conducted tests by the same operator on the same material should not differ by more than 7.6 lb/yd^3 (4.5 kg/m^3) (d2s).

Appendix D

Proposed Standard Method of Test for Density of Freshly Mixed Concrete In Place by a Twin-Probe Nuclear Method

Proposed Standard Method of Test
for
Density of Freshly Mixed Concrete In Place By a Twin-Probe Nuclear Method

AASHTO DESIGNATION: TP 24
SHRP DESIGNATION: 2028

1. SCOPE

1.1 This method covers the determination of in-place density of unhardened concrete by gamma radiation. Density of horizontal strata is evaluated using a commercially available twin probe nuclear density gage. Freshly mixed concrete density is nondestructively determined through calibration of nuclear count rate with several materials at different and known densities. For notes on the nuclear test see appendix to this test method.

1.2 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. REFERENCED DOCUMENTS

2.1 AASHTO *Standards*

T 121 Weight Per Cubic Foot, Yield, and Air Content (Gravimetric) of Concrete

2.2 ASTM *Standards*

C 670 Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials

3. SIGNIFICANCE AND USE

3.1 This test method is useful as a rapid nondestructive testing technique for in-place determination of the density of horizontal strata in unhardened concrete. The fundamental assumptions inherent in the test method is that Compton scattering is the dominant interaction and that the material under test is homogeneous.

3.2 This method can be used as a quality control tool and for assisting acceptance testing during construction.

3.3 The method presented is used to nondestructively determine concrete density. Density, determined by any means, is a function of mix constituents and proportions. It should be recognized that normal variability of these components will affect in-place density. When used to evaluate the degree of concrete consolidation, ratio of actual to design mix density, and variability of concrete density should be recognized.

3.4 Nuclear density count rate of unhardened concrete is affected by proximity to steel, chemical composition of concrete constituents, and concrete heterogeneity. Precision statements are based on a particular gage. Variations can be minimized by user's compliance with appropriate sections of the test procedure and manufacturer's maintenance and operation procedures.

4. APPARATUS

4.1 Exact details of equipment may vary, but should meet or exceed the system precision stated in the test procedure. The system shall consist of the following:

4.1.1 Gamma Source Probe— An encapsulated and sealed radio-isotopic source such as cesium-137 (see Appendix to this test procedure). The encapsulated radioactive source shall be detachable for safe storage from the probe inserted into guide tubes.

4.1.2 Detector Probe— Gamma radiation detector such as a Geiger-Muller tube or scintillation crystal mounted on probe inserted into guide tubes.

4.1.3 Guide Tubes— Metal tubes with solid points to aid in plastic concrete insertion. Caps on guide tubes shall be so made mechanically, that when probes are manually moved to the desired depth, it will be held securely in position.

4.1.4 Readout Instrument— A direct readout meter.

4.1.5 Gage Housing— The readout instrument, electronics, and power supplies shall be in housings of rugged construction that are moisture, shock, and dust proof.

4.1.6 Detector Cable— Cable shielding wires connecting detector to gage housing.

4.1.7 Reference Standards— Blocks of uniform density provided for checking equipment operation, repeatability, and density calibration.

4.1.8 Guide Tube Frame Assembly and Base Plate— A rigid frame mounted to a base plate to maintain constant distance between guide tubes inserted into unhardened concrete.

4.2 Guide tubes should be marked in increments of at least 1/2 in. (13 mm) to aid in driving tubes to equal elevations. Probe rods should be marked in increments of at least 1/4 in. (6 mm) to establish source and detector elevations.

5. CALIBRATION

5.1 A calibration curve for each gage is established by determining the nuclear count rate of each of several materials at different and known densities, plotting the count ratio versus each known density, and plotting a curve through the resulting points. Materials should be selected over a range of densities, both greater and less than the anticipated density during construction.

5.2 In addition to manufacturer-supplied reference standards, calibration may be determined on unhardened or hardened concrete.

5.2.1 When concrete is used to calibrate density with nuclear count rate, concrete specimens should be mixed, placed, and consolidated to ensure a relatively homogeneous material. Density of unhardened concrete should be measured in general accordance with T 121. Concrete should be consolidated in the same manner as concrete specimens fabricated for nuclear density testing.

5.2.2 When permanent, uniform, hardened concrete blocks are used to calibrate the gage, guide tubes should be cast-in-place. Concrete density should be determined from measured block or companion cylinder specific gravity.

5.3 Orientation of the detector and source should be recorded and consistently be maintained during calibration and field testing. Orientation effects can result in different nuclear count rates. In addition to consistent orientation, distance between source and detector must be constant.

5.4 A minimum of four density readings should be made for each calibration point. A second set of four readings should be made by reversing the source and detector positions.

5.5 The average of the eight nuclear count rates for each material density should be divided by the average count on the manufacturer -recommended reference standard. Density as a function of the natural logarithm of count ratio is then established.

6. STANDARDIZATION

6.1 Calibration should be checked by measuring nuclear count rates of manufacturer supplied reference standards. Deviations in predicted and actual density requires gage recalibration.

6.2 Standardization is also required at the start of each testing day and whenever test measurements are suspect.

6.3 Warm-up time shall be in accordance with manufacturer's recommendations. Take at least six readings on the reference standard, or more if recommended by the manufacturer. Source and detector position are reversed after each test.

6.4 If more than one of the individual readings is outside the 5 percent level of significance, set by the following equation, repeat the standardization.

$$(N_c - N_o) < 1.96 \text{ sqrt } (N_o)$$

where:

N_c = count currently measured in checking the instrument

N_o = original count established during calibration

6.5 If the second attempt does not satisfy the criteria in 6.4 check the system for a malfunction. If no malfunction is found, or if criterion in 6.4 is satisfied, establish the reference count by taking the average of a minimum of 10 count rates on the reference standard. Source and detector positions should be reversed after each test.

7. PROCEDURE

7.1 Warm-up time shall be in accordance with manufacturer's recommendations.

7.2 The reference count should be established.

7.3 Position the guide frame and base plate on unhardened concrete surface where density is to be determined.

7.4 Position guide tubes in frame and gently drive tubes into plastic concrete by manually pushing them or gently tapping with small mallet. Care should be exercised to ensure both tubes are set at equal elevations.

7.5 Insert source and detector probes into guide tubes and position at equal elevations.

7.6 Take automatically timed readings for a minimum time in accordance with manufacturer's recommendations. Reverse source and detector rods and repeat reading.

8. CALCULATIONS

8.1 Average the two density readings at each elevation tested.

8.2 Divide average nuclear count rate by reference standard count rate.

8.3 Unhardened concrete density is determined using the count ratio and correlation established during calibration.

9. PRECISION

9.1 The precision of the instrument is determined from the slope of the calibration curve and the statistical standard deviation of the count rate, as follows:

$$P = s/S$$

where

P = precision, lb/ft^3 (kg/m)^3
s = standard deviation, cpm (counts/minute), and
S = slope, cpm/lb/ft^3 (cpm/kg/m^3).

9.1.1 The slope of the calibration curve is determined at the 140 lb/ft^3 (2240 kg/m^3) point and the standard deviation is calculated from ten individual automatically-timed readings taken on 140 ± 5 lb/ft^3 (2240 ± 80 kg/m^3) material. The gage is not moved after seating for the first count. For a direct reading gage, the precision, P, in lb/ft^3 (kg/m^3), is the standard deviation of 10 individual automatically-timed density readings.

9.1.2 The single operator equipment precision had been found to be 0.7 lb/ft^3 (11.2 kg/m^3) (1s). Therefore, results of two properly conducted tests by the same operator with the same equipment should not differ by more than 2.0 lb/ft^3 (31.7 kg/m^3) (d2s).

APPENDIX

NOTES ON THE NUCLEAR DENSITY TEST

It should be noted that the volume of concrete represented in the measurements is indeterminate and will vary with (1) the source-detector geometry of the equipment used, and (2) with the characteristics of the material tested. In general, and with all other conditions constant, the denser the material, the lower the nuclear count rate. The density so determined is not necessarily the average density within the volume involved in the measurement. Where these materials are of uniform density, this characteristics of this test method is of no effect. However, reinforcing steel and underlying concrete are both often within the volume in which they may influence gage readings. Also, the extent of the influence of density variations, such as those caused by reinforcing steel, depends on the proximity of source and detector to steel.

The use of cesium-137 and other radiosotope sources in density gages is regulated and licensed by the U.S. Nuclear Regulatory Commission or, in agreement states, by state regulatory agencies. The primary objective of these regulations is the use of these materials in a manner safe to the operator, to other workers, and to the general public. In order to meet regulatory agency requirements, gage owners must establish effective operator instruction and use procedures, together with routine safety procedures, such as source leak tests, recording and evaluation of film badge data, the use of survey meters, etc., in connection with the operation of this type of equipment. Likewise, individual users of the equipment must complete a formal training program and must be familiar with possible safety hazards.

The determination of density in these test methods by nuclear means is indirect. The relationship between nuclear gage count rate and density necessarily is determined by correlation tests on materials at known densities.

Concrete and Structures Advisory Committee

Chairman
James J. Murphy
New York Department of Transportation (retired)

Vice Chairman
Howard H. Newlon, Jr.
Virginia Transportation Research Council (retired)

Members

Charles J. Arnold
Michigan Department of Transportation

Donald E. Beuerlein
Koss Construction Co.

Bernard C. Brown
Iowa Department of Transportation

Richard D. Gaynor
National Aggregates Association/National Ready Mixed Concrete Association

Robert J. Girard
Missouri Highway and Transportation Department

David L. Gress
University of New Hampshire

Gary Lee Hoffman
Pennsylvania Department of Transportation

Brian B. Hope
Queens University

Carl E. Locke, Jr.
University of Kansas

Clellon L. Loveall
Tennessee Department of Transportation

David G. Manning
Ontario Ministry of Transportation

Robert G. Packard
Portland Cement Association

James E. Roberts
California Department of Transportation

John M. Scanlon, Jr.
Wiss Janney Elstner Associates

Charles F. Scholer
Purdue University

Lawrence L. Smith
Florida Department of Transportation

John R. Strada
Washington Department of Transportation (retired)

Liaisons

Theodore R. Ferragut
Federal Highway Administration

Crawford F. Jencks
Transportation Research Board

Bryant Mather
USAE Waterways Experiment Station

Thomas J. Pasko, Jr.
Federal Highway Administration

John L. Rice
Federal Aviation Administration

Suneel Vanikar
Federal Highway Administration

Expert Task Group

Bernard C. Brown
Iowa Department of Transportation

Stephen Forster
Federal Highway Administration

David L. Gress
University of New Hampshire

Crawford Jencks
Transportation Research Board

Celik Ozyildirim
Virginia Transportation Research Council

Terry Mitchell
Federal Highway Administration

Robert J. Perry
New York State Department of Transportation

Charles F. Scholer
Purdue University

Lawrence L. Smith
Florida Department of Transportation